FLOOD ANALYSES REPORT

Flood Analyses Report

Submitted to:
California State Coastal Conservancy
U.S. Fish & Wildlife Service
California Department of Fish and Game

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ENCLOSURE (Available upon request)

DVD containing the original UNET and the HEC-RAS models received from the District. The DVD also contains the HEC-RAS models prepared for this project analysis. The data output generated by the HEC-RAS program is also available on the DVD. Use the *Readme* file located on the DVD for additional information about the specific content of the DVD.

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ABBREVIATIONS & ACRONYMS

1D	One-dimensional
2D	Two-dimensional
3D	Three-dimensional
BFE	Base Flood Elevation
BM	Bench Mark
CCS	California Coordinate System
CDFG	California Department of Fish and Game
CEQA	California Environmental Quality Act
cms	Cubic Meters per Second
cu-m	Cubic Meters
EIR	Environmental Impact Report
EIS	Environmental Impact Statement
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
HEC-2	Hydraulic Engineering Center 2 Model
HEC-RAS	Hydraulic Engineering Center – River Analysis System
ISP	Initial Stewardship Plan
LGR	Lower Guadalupe River
LGRP	Lower Guadalupe River Flood Protection Project
LiDAR	Light Detection and Ranging
MHHW	Mean Higher High Water
MHW	Mean High Water
m	Meters
NAVD88	North American Vertical Datum of 1988
NEPA	National Environmental Policy Act
NGVD29	National Geodetic Vertical Datum of 1929
QA	Quality Assurance
RDSS	Read Decision Support System
RS	River Station
Shoreline Study	South San Francisco Bay Shoreline Study
SBSP	South Bay Salt Pond
SCVWD	Santa Clara Valley Water District
SLR	Sea Level Rise
UNET	Unsteady NETwork model
UPRR	Union Pacific Railroad
USACE	United States Army Corps of Engineers
USF&WS	United States Fish and Wildlife Service
USGS	United States Geological Survey
WSEL	Water Surface Elevation

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1. EXECUTIVE SUMMARY

This report documents the coastal and fluvial flood technical analyses conducted in support of long term planning for the South San Francisco Bay Salt Pond (SBSP) Restoration Project.[1] The report provides a more detailed description of the proposed flood management program presented in the Final Alternatives Report (PWA et al. 2006) and presents modeling analyses demonstrating the potential benefits of the project to reducing flood hazards at the mouths of creeks flowing through the project area. The analyses documented in this report will be used to assess potential project-related changes in flood protection at the programmatic level in the SBSP Restoration Project EIS/R.

The report is organized into two main sections:

- *Coastal flood analyses.* The purpose of the coastal analyses is to inform planning and design for the proposed SBSP coastal flood protection program, consisting of a system of shoreline levees to provide flood protection from high bay water levels and wind-waves. These coastal flood levees will connect with the levee system providing flood management along each of the fluvial channels. The coastal analyses consist of defining the proposed flood protection levee alignment and providing an initial levee design (levee cross-sections, including crest elevation) expected to be necessary to provide 100-year coastal flood protection that could be certified by the Federal Emergency Management Agency (FEMA). The coastal analyses include an evaluation of the South San Francisco Bay extreme water levels and waves.

- *Fluvial flood analyses*. The fluvial analyses focus on demonstrating "proof of concept" for the project approach to fluvial flood risk reduction, modeling one case study in detail. The project proposes to reduce flood hazards at the mouths of creeks flowing through the SBSP complex by (1) constructing a larger flow area (removing confining levees through the ponds) and (2) using future tidal scour in the channels to maintain flood benefits over time, even as the floodplain/marshplain fills with sediments. A hydraulic model of the lower Guadalupe River (Alviso Slough) was developed to quantitatively demonstrate the effectiveness of this approach. Comparable or greater flood reduction benefits are expected in other creeks.

The flood analyses presented in this report support long-term planning, which is being conducted at a conceptual level of detail, to be followed by more detailed planning as individual phases of the plan proceed to implementation. The analyses are intended to demonstrate general feasibility and provide input to preliminary cost estimates. They will be used for *program*-level evaluation of the long-term plan in the Environmental Impact Statement/Environmental Impact Report (EIS/R). Subsequent, *project*-level analyses will be conducted for each phase of implementation. These will be accomplished, in part, by the

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[1] This report was prepared in July 2006, with minor corrections and revisions for clarity in November 2007. Due to refinements to the project description between July 2006 and release of the Final EIS/R, the information in the Final EIS/R supersedes the information in this report.

US Army Corps of Engineers (USACE) South San Francisco Bay Shoreline Study (Shoreline Study) and the FEMA Re-Study of Coastal Flood Hazards in South San Francisco Bay.

The project includes adaptive management as an integral part of the planning and implementation process, including planning and implementation for flood management. The adaptive management process will consist of monitoring, implementing experiments, actively learning, and adjusting actions as the project proceeds. Project implementation will be phased over many years; learning from the performance of early phases will guide implementation of later phases. For the flood management elements, adaptive management is expected to inform refined predictions of channel and floodplain geomorphic evolution (scour and sedimentation) and will be used to regularly assess flood performance as new water level and other data become available. Any actions needed to improve levels of flood protection can be identified and implemented though ongoing adaptive management.

Coastal Flood Analyses

One of the goals of the SBSP Restoration Project is to maintain or improve flood protection in the project areas and for developed areas landward of the project area. The SBSP Restoration Project is committed to ensuring that future flood protection with the Project is equal to, or better than, existing conditions. Beyond this, it is desirable by all entities to develop a flood management program around the SBSP Restoration Project area that would provide a consistent level of flood hazard management with flood protection measures (levees, high ground) meeting both FEMA and Corps criteria. The Project expects to be able to achieve this objective. However, the actual level of protection over and above existing would depend on a number of considerations, but most important is funding.

In many locations, the perimeter levee will follow the alignment of the existing inboard pond levees. The alignment of the proposed perimeter levees is shown in Figure 8 and Figure 9. The location shown represents the current preferred alignment, based on input from landowners, stakeholders, and local flood protection agencies. However, it is subject to refinement during subsequent detailed design studies.

The coastal analysis resulted in a range of conceptual design levee crest elevations and typical cross-sections for each pond complex. Typical cross sections are shown in Figures 10, 11 and 12. Crest elevation ranges for the three pond complexes are:

Table 1. Approximate Levee Crest Elevations for the Preliminary Coastal Levees

Pond Complex	Approximate Required Crest Elevation Range (MLLW)	
Eden Landing, Alameda County	4.3 to 5.8 meters	14 to 19 feet
Ravenswood, San Mateo County	4.6 to 6.1 meters	15 to 20 feet
Alviso, Santa Clara County	4.9 to 6.7 meters	16 to 22 feet

The above elevations are based on available data and prior studies and are subject to confirmation and refinement in future studies. The low estimate is based on the 100-year still water levels at each location and is similar to existing FEMA flood levels, plus wind setup, 0.15 m (0.5 ft) of future sea level rise, and 0.6 m (2 ft) of levee freeboard. The high estimates include wind setup, wind wave setup and runup, 0.15 m (0.5 ft) of future relative sea level rise, and 0.3 m (1 ft) of levee freeboard. Land subsidence and local settlement are not included, so any initial over-build would need to be added to the crest elevations. The sea level rise value of 0.5 ft and the 50-year planning horizon are minimums that may be increased based on further consideration of risk.

The coastal flooding analysis used a combination of engineering methods to quantify future coastal flood hazards. The analysis approach included methods for estimating extreme water levels, wind-wave development, and wave runup elevations. An approach was developed that incorporated these main components in a series of lookup tables to expedite programmatic-level planning and provide a conservative set of maximum flood elevation estimates. For each pond complex, the conceptual design cross-section varies by location, depending on level of anticipated exposure to wind-waves from the Bay and levee fronting conditions as defined by the final alternatives. Section 4 provides additional information on methods and results for the coastal flood analysis.

Fluvial Flood Analyses

The fluvial assessment addresses potential flooding issues with the major drainages that convey rainfall runoff from upland watersheds through the salt ponds to the San Francisco Bay. Flooding in the downstream reaches of these drainages occurs during periods of high river flow rate and high Bay water levels. The conceptual approach to managing fluvial flood hazards in the SBSP project is to identify opportunities to lower flood water surface elevations in the lower creeks and channels and concurrently improve flood levee protection in the areas immediately upstream of the salt ponds. Lowering the water levels in the creek channels in the SBSP complex will be accomplished by constructing a larger flow area (by removing or setting back the confining levees) and by using increased tidal flows from the restored marshes to scour the existing channel over time. We demonstrate the effectiveness of this approach using hydraulic model studies on the lower Guadalupe River (Alviso Slough). This is one of the most severe test cases because of the critical nature of flood hazards in this area, and the current use of adjacent salt ponds (A8, A7, A6, & A5) for flood flow storage. Model results demonstrate that the loss of flood storage can be offset by the increased channel conveyance. Comparable and greater benefits are expected in other fluvial creeks that do not use adjacent ponds for storage. Comparable hydraulic model testing will be conducted on other SBSP streams as part of the Shoreline Study and implementation of future phases of the project.

The fluvial analysis was conducted using the Hydrologic Engineering Center – River Analysis System (HEC-RAS) hydrodynamic model. It is based on previous Santa Clara Valley Water District analysis in the lower Guadalupe River / Alviso Slough system updated to current and future conditions. We updated the Santa Clara Valley Water District (The District) hydraulic models with current information to develop our existing conditions model. Our modeling approach uses a combination of hydraulic modeling for Alviso Slough and the adjacent ponds and “hydraulic geometry” relationships to predict future channel conditions for the three alternatives. This approach allows analysis and comparison of the various restoration and flood management measures (e.g., pond breaches, levee setbacks, levee lowering, pond storage, and channel scour associated with tidal restoration) that will affect upstream water levels. This modeling approach can be refined as project planning proceeds. The current fluvial analyses are appropriate at the programmatic-planning level, and contain a series of assumptions regarding detailed-level project design decisions, such as the extent of levee removal, and levee breach sizes and locations.

The alternatives evaluated at Alviso Slough are:

- Alternative A: No Action
- Alternative B: Managed Pond Emphasis (50:50 Tidal Habitat to Managed Pond)
- Alternative C: Tidal Habitat Emphasis (90:10 Tidal Habitat to Managed Pond)

Modeling for each alternative included evaluation of short term (immediately after construction) and long-term (after 50-years) conditions. Results were then used to compare flood performance. This approach will support refinement of restoration alternatives, selection of a preferred restoration plan, and will support the National Environmental Policy Act / California Environmental Quality Act (NEPA/CEQA) environmental impact analysis of the SBSP Restoration Project.

The No Action Alternative modeled in this analysis assumes that the west bank slough levees will fail in the next 50 years and ponds A5, A6 and A7 will become tidal while sea-level rise will exacerbate coastal flood conditions (see *Final Alternatives Report* (PWA et al. 2006)). West bank levee failures were modeled as unplanned breaches located where the existing levee crosses a historic slough channel. This condition requires that the east bank levees protecting ponds A9, A10, A11, and A12 are maintained to preserve the current level of flood protection for the community of Alviso. SBSP restoration alternatives B and C propose to breach the slough levees at historic slough channel locations in the short term. In the long-term the levees were modeled as lowered to future marshplain elevation resulting in a widened cross-section and a reduction of flood levels upstream of the Alviso pond complex. The linear extent of levee lowering or removal has not been defined but since the effective flow area of the levee is small in comparison to the overall flow path in alternatives B and C, the levees were removed in the analyses. Comparable types of flood hazard reduction opportunities are available in other fluvial systems in the project area.

The results of the fluvial analysis show that by lowering or eliminating the constraining slough levees through the ponds and reconnecting the ponds (future marsh plain) to the slough, the flow area is increased, resulting in lower fluvial flood levels in the lower Guadalupe River / Alviso Slough. Tidal scour is expected to improve on these benefits in the future.

The results of the fluvial analysis show that by lowering or eliminating the constraining slough levees through the ponds and reconnecting the ponds (future marsh plain) to the slough, the flow area is increased, resulting in lower fluvial flood levels in the lower Guadalupe River / Alviso Slough. When compared to the Baseline Conditions Model the short term results for Alternative B and C show a 0.11 m (0.4 ft) reduction in flood levels at the Gold Street Bridge. Long-term project alternatives were modeled with sea level rise. Alternative A results show no improvement to the water surface elevation at the Gold Street Bridge when compared to the Baseline Conditions Model despite the downstream increase of 0.15

m (0.5 ft). The resulting water surface elevation at Gold Street Bridge for Alternatives B and C long-term show a reduction in water levels of 0.18 m (0.59 ft) and 0.31 m (1.02 ft), respectively when compared to the Baseline Conditions Model.

The technical approach to the coastal and fluvial analysis recognizes the opportunity for adaptive management to refine the analysis / design and inform decision-making during later stages of the project. In the future, the analysis may be refined based on preliminary results and input from the Science Team, Project Management Team (PMT), and independent external technical reviewers.

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2. INTRODUCTION

This report summarizes coastal and fluvial analysis conducted for long-term planning at a programmatic level for the SBSP restoration project to provide an integrated system of both coastal and fluvial flood elements.[2] The coastal flood protection program identifies a system of shoreline levees to provide flood management to coastal floods resulting from high bay water levels and wind-waves. These coastal flood levees will connect with the levee system providing flood management along each of the fluvial channels. This report also provides a summary of the fluvial hydraulic model for Alviso Slough to quantify the potential opportunities for reducing flood hazards as part of the SBSP Restoration Project. The SBSP restoration project has identified two broad restoration alternatives and a No Action alternative. Each restoration alternative integrates flood protection, habitat restoration, and public access at a programmatic (general) level of detail (PWA et al. 2006). This analysis examines the potential flood hazard impacts of all three alternatives and their proposed project changes to existing infrastructure to describe the fluvial flood reduction benefits and potential coastal and fluvial improvements resulting from the SBSP project.

Coastal Analysis

This document provides a summary of the programmatic evaluation of future coastal flooding risks in each of the three pond complexes shown on the project vicinity map (Figure 1). Extreme Bay water levels were estimated for the Eden Landing, Alviso, and Ravenswood area. Incorporating FEMA criteria for levee certification, “still water” and total water surface elevations for each pond complex were used to establish a range of levee crest elevations corresponding to conditions without and with wind-wave action, respectively.

Coastal flood hazards result from extreme tides, with water levels further raised by storm surge and waves. Planning for coastal floods must take into account existing flood hazards, but also recognize evolving conditions including sea level rise and local subsidence. The South Bay has elevated tides relative to the ocean and the rest of the Bay. The maximum tide levels generally increase with distance southward, although the tidal levels in the numerous tributary sloughs are not well quantified. Prior studies have estimated the high South Bay water levels by assuming a linear relationship with the heights at the Bay mouth (San Francisco). This approach was largely taken due to the long data record at the Presidio, San Francisco tide gauge, very limited records available in the south bay, and the ability to approximately scale tide range with linear multipliers. This approach has not been evaluated for decades and detailed hydrodynamic modeling has been recommended to develop a better understanding of surge response (including other high water components) in the South Bay. Recent El Nino events (1983-4 and 1997-8) have caused an increase of Bay water levels averaging about 0.3 m (1 ft) over the entire winter, with peak increases on the order of 0.6 to 0.9 m (2 to 3 ft) during storms. Wind waves can exceed 1.5 m

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(5 ft) in exposed areas of the Bay during extreme wind events with recurrence on the order of 100 years. This level of wind wave action can erode and overtop most of the existing salt pond levees.

Nevertheless, since construction, the salt ponds have been very effective dissipaters of incident wind wave action and act as large reservoirs to store overtopped waters. With frequent maintenance of the non-engineered levee systems, the salt ponds have historically formed an effective ad-hoc flood protection system. However, there is no formal assessment of the flood management effectiveness of the existing system and hence the actual performance under design conditions is uncertain.

Fluvial Analysis

This document provides a summary of the lower Guadalupe River and Alviso Slough hydraulic models that quantify the potential opportunities for reducing flood hazards as part of the SBSP restoration project. Alviso Slough is a major tidally influenced waterway through the restoration area. It was selected as a demonstration site to evaluate the potential fluvial flood hazard reduction benefits and potential impacts of the SBSP Restoration Project. This analysis provides an opportunity to evaluate how slough channels in the project area will function under the proposed restoration project alternatives. Alviso Slough includes one relatively unique feature: the channel currently benefits from offline storage facilities (former salt ponds A5, A6, A7, & A8). In the SBSP restoration alternatives B & C, the pond levees are removed or relocated with a large setback allowing the ponds to become fully tidal, and eventually become tidal marsh plain that fluvial flood flows overtop to improve flow conveyance to the Bay. Alviso Slough was selected as a demonstration because it can demonstrate the restoration benefits of a system that currently uses offline storage to manage floodwaters. Other fluvial systems within the SBSP project will have a greater flood reduction benefit as a result of the restoration alternatives because they do not rely on offline storage to reduce flood water levels. This case study will assist the SBSP project team to quantify water level reduction benefits at other streams in the overall SBSP project site. This analysis can be extended to study the existing flood management needs in these other systems.

The Alviso Slough project reach extends 6.5 km from the UPRR Bridge, in the community of Alviso through the Alviso Pond Complex to the mouth of Alviso Slough at Coyote Creek. There are 24 salt ponds in the complex totaling 30.4 square-kilometers [km^2] (7,500 acres). This analysis included 10 ponds associated with Alviso Slough flooding. Guadalupe River and Alviso Slough are located within the District's Central Flood Control Zone. Alviso Slough receives runoff from the 440.3 km^2 (170 square miles) Guadalupe River Watershed. The flow regime reach in the project site is a combination of tidal and fluvial processes.

Fluvial flood hazards will be reduced where levees are removed or lowered, and increased tidal flows from adjacent salt pond restoration scours the lower reaches of flood control channels, resulting in increased flow conveyance and a lower water surface elevation In locations subject to both fluvial and coastal flooding, levee elevations will be designed to accommodate the appropriate risk of individual (*i.e.* fluvial or coastal) as well as simultaneous high tide and high river flow flood occurrences. The resulting flood management program will provide a more consistent and higher level of flood protection compared to existing conditions.

2.1 Purpose

This document presents the model set-up, methodologies and results of the coastal and fluvial technical analyses that were developed to support formulation of SBSP restoration alternatives, describe the expected system response to be used in the selection of a preferred restoration plan, and support the National Environmental Policy Act / California Environmental Quality Act (NEPA/CEQA) environmental impact analyses phase of the SBSP Restoration Project. The associated environmental impacts of the proposed restoration and management actions will be described in the Environmental Impact Statement / Environmental Impact Report (EIS/R).

NEPA/CEQA does not require that technical analyses be performed, but analyses are often advantageous to a project in order to exclude potential impacts. For example, technical analyses can assist in avoiding impacts and/or demonstrating that impacts are negligible. A programmatic EIS/R addresses broad policy issues, and can be followed by other site- or project-specific EIS/Rs that "tier off" the programmatic EIS/R. The project-level EIS/Rs will not need to reevaluate the broad policy matters, but instead will refer to the programmatic-level EIS/R and focus on more detailed project-specific impact assessments. Project-specific analysis may reveal impacts of greater magnitude than those anticipated in the programmatic EIS/R. The programmatic-level EIS/R need only provide a general level of detail regarding the potential impacts and general mitigation measures that can be applied, as well as an overview of the regional impacts and general site impacts of each final alternative. The analysis presented here are in support of the programmatic EIS/R assessment.

The analysis of impacts requires a comparison of post-project conditions with a "baseline" condition. Under CEQA, the baseline condition is the existing, on-the-ground conditions at the time that the draft EIR is prepared. NEPA allows the setting to be either existing on-the-ground conditions or some future without project conditions. Because of CEQA's stricter definition, the combined EIS/R uses the existing conditions as the baseline. The baseline conditions for the SBSP restoration project will be defined in the EIS/EIR as "Fall 2006". Most project-related impacts will change over time, therefore future conditions must also be considered under NEPA/CEQA. The analysis presented in this report will inform and evaluate the most-likely future conditions, although there are no sure predictors of future conditions.

The coastal analysis evaluated the existing flood potential using existing flood plain maps and prior studies (see also Existing Conditions Report, 2005 and Section 2.2 of this report). The coastal flood analysis results in estimated locations and basic dimensions for new coastal flood control levees to be included in the SBSP Alternatives, for program-level environmental analysis. By definition, this component will improve flood protection from the coastal flood source and reassessment of the existing coastal flood potential was not required. The presumed required level of flood protection is the 100-year coastal flood elevation as defined by the FEMA (FEMA, 2005; FEMA, 1988). The coastal analysis considered primarily coastal hydraulic criteria, assuming typical earth-levee geometries.

For the fluvial flood source, post-project (short-term, immediately after implementation, and long-term, year 50) conditions are compared to baseline conditions. The long-term with- and without-project are also compared. Environmental changes that may result under the alternatives are considered as either adverse or beneficial impacts.

The analysis of Alviso Slough was conducted as an example project to evaluate the potential flood hazard reduction benefits of the SBSP Restoration Project. This modeling effort demonstrates that the restoration project may reduce fluvial flood hazards at other riverine outlets to the Bay. Impacts for fluvial systems other than Alviso Slough will be characterized using qualitative analysis supplemented with this analysis in EIS/EIR phase. In general, it is expected that flood benefits in other channels within the SBSP alternatives will be greater, as these systems do not actively rely on adjacent salt ponds for flood storage.

2.2 Prior Studies

Several prior studies have been conducted to describe coastal and fluvial flood hazards and support the development of flood management strategies in the south bay. The following sections provide a brief review of two coastal studies of extreme water levels estimated for San Francisco and the South Bay, a levee assessment report for the SBSP, and a series of riverine studies that provided the initial framework for the Alviso Slough hydraulic model.

2.2.1 Coastal Studies

Key prior coastal studies in the South Bay, called the Shoreline Studies (U.S. Army Corps of Engineers 1988b; 1989) concluded that the coastal flood risk depends largely on the level of levee damages during a flood event. The range of coastal flooding predicted varies by location, with major flooding predicted in the developed areas inland of the Alviso Ponds, less but locally significant flooding in the vicinity of the Ravenswood Ponds and areas to the south, and minimal coastal flooding near the Eden Landing Ponds and areas to the south. This analysis relied on the high water levels developed by the Corps in their 1984 study (see next paragraph). The effects of coincident wind waves were considered in terms of levee erosion, runup and overtopping. It was assumed that the 1984 levee conditions would be maintained and provide some protection during the flood events analyzed. Updates to these studies are anticipated.

In 1984, the USACE published a water level analysis based on a tide stage versus frequency curve. The curve was developed from 129 years of annual maximum water level data at the San Francisco-Presidio tide gauge (1855-1983) (U.S. Army Corps of Engineers 1984a). Due to disagreement in the recurrence interval for water levels measured in 1983 between the frequency curve and plotted data, the USACE adjusted the frequency curve to reflect what was felt to be a more appropriate recurrence interval for the 1983 water levels. To do so, they looked at the difference between the mean annual maximum tide for the 20-year interval prior to 1984 and that for the 129 year record. This is shown in Figure 2. The mean annual maximum tide for 1963 to 1983 is 0.16 m (0.53 ft) higher than that for 129 years of record as a whole. To account for this trend, the USACE adjusted the mean of the computed tidal stage versus frequency curve upwards by this amount. The USACE then developed tide stage versus frequency curves for other stations around the Bay by assuming datum frequencies similar to those at San Francisco. The profile of these computed 100-year water levels for the Bay south of San Francisco was then smoothed to yield the adopted 100-year water level profile which resulted in slightly higher values than computed.

Figure 3 shows the adopted USACE (1984a) 100-year water level profile for the South Bay. This is a key report that influenced both the prior Shoreline Study by the Corps (described above) and the existing FEMA flood mapping presently in effect (see next paragraph). In addition to this water level analysis, Knuuti (1995) conducted an extreme value analysis on a detrended time series of annual maximum water levels from the San Francisco-Presidio tide gauge between 1897 and 1995. Knuuti's 100-year water level estimate for the San Francisco-Presidio station is also depicted in Figure 3.

FEMA has published flood limits for the periphery of San Francisco Bay (Federal Emergency Management Agency 1981; Federal Emergency Management Agency 1997, 1998, 1999 1999 #1820; Federal Emergency Management Agency 1998a; Federal Emergency Management Agency 1998b; Federal Emergency Management Agency 1999a; Federal Emergency Management Agency 1999b; Federal Emergency Management Agency 2000; PWA 2005; PWA et al. 2006)) (also see the Existing Conditions Report (PWA et al. 2005)). These flood limits extend as far as several miles inland of the salt ponds, and are therefore much more extensive than estimated by the Corps Shoreline Studies accomplished in the late 1980's (described above). The existing FEMA coastal flood plain is based on the 100-year high water levels estimated by the Corps in the 1980's, as described above. Since the existing salt pond levees are not certified, the mapping was based on the theoretical condition of "failed levees" resulting in free propagation of Bay waters and inundation of the surrounding areas. However, the effects of wave action were not added to the water levels – and hence these flood limits are associated with "inland projection of the 100-year still water level." Updates to these studies according to new guidelines are anticipated (FEMA, 2005).

The general condition of the Alviso and Ravenswood levee networks were assessed by Moffatt and Nichol (Moffatt & Nichol Engineers 2005b) and existing levee physical parameters (such as length, slope, width, vegetation) were recorded. Levee construction methods, levee materials, and subsurface conditions are further detailed in the report by Moffatt & Nichol (Moffatt & Nichol Engineers 2005b). A potential "perimeter levee" was identified for urban flood protection and both design and cost for the levee were evaluated (Moffatt & Nichol Engineers 2005b). In addition to the salt pond levees, there are a number of engineered flood management levees and areas of high ground.

2.2.2 Fluvial Studies

The Lower Guadalupe River Sedimentation Study, Santa Clara Valley Water District Final Report (Northwest Hydraulic Consultants (NHC) 2000) described the Lower Guadalupe River hydraulics, flow conveyance and sediment issues. Both long-term trends in river degradation and aggradation as well as episodic single flood events were evaluated for existing conditions and for two alternative channel designs. This study also described the estimated future channel condition of the no-project alternative.

The Lower Guadalupe River Planning Study: Engineer's Report by NHC described the flood-related problems in the lower river reaches of the Guadalupe River Watershed. The report recommended a capital improvement project on the lower Guadalupe River, between Union Pacific Rail Road (UPRR) and Interstate 880, with a variety of alternatives. The recommended project provided 100-year design flood protection from overbank flooding along the river reach. The project included channel improvements and a combination of floodwalls, levees, and channel improvements along a 10 km corridor (Santa Clara Valley Water District 2001). Other project goals were to protect endangered species, preserve fish and migratory bird habitat, and to minimize long-term maintenance costs. Construction of the channel improvements project was completed in December 2005. Jones & Stokes Associates (Jones & Stokes 2001) prepared the draft Environmental Impact Report: Lower Guadalupe River Planning Study for the District's project. The EIR described the project as proposed, a no-project alternative, a channel bank modification alternative, and a channel bypass alternative. The EIR and the Engineer's Report address the reach immediately upstream of the SBSP Project site, but provided data for development of the Alviso Slough hydraulic model.

The Final Reconnaissance Report by NHC extended the analysis of the lower Guadalupe River Flood Protection Planning Process Reports to examine how the effects of the flood protection project on the Lower Guadalupe River (LGR) and Baylands would respond to a design flood event on the LGR. The Final Reconnaissance Report of June 2002 provided Technical Memorandums that documented the results of the studies (Northwest Hydraulic Consultants 2002). The memorandums described modeling parameters that we reviewed and used to setup the Alviso Slough fluvial model for the SBSP project.

The Alviso Slough Tidal Enhancement Project: Draft Technical Report (Schaaf & Wheeler 2004) evaluated a proposed project to increase the tidal prism (the volume of water exchanged between the Bay and the marsh through the channel during an average channel cycle) in Alviso Slough by creating a hydraulic connection from Alviso Slough to Pond A8 and removing approximately seven acres of channel vegetation. The project objectives were to increase salinity in the slough, reduce freshwater vegetation, increase tidal flow velocities, and decrease sedimentation in the channel. The report indicated that the average water surface elevation in Pond A8 would increase from elevation -1.0 (feet NAVD88) to elevation 0.0 if the pond were to be managed in a muted tidal regime. The increased water surface would reduce the amount of flood storage available and increase the potential for overflows into other adjacent ponds. The Alviso Slough Tidal Enhancement Project has not been implemented but is being considered within the context of the SBSP Restoration Project. As part of the SBSP Phase 1, the impacts of flooding effects from Pond A8 becoming tidal will be assessed.

2.3 Report Organization

This report is divided into two major parts: coastal flood analyses and fluvial flood analyses. Each section includes a description of the analysis tool and physical processes modeled, a description of the model set-up and methodology, and the proposed model strategy for assessing environmental benefits and impacts:

- **Section 4. Coastal Flooding Analyses.** This section presents the analysis for estimating extreme water levels, wind-wave estimates, wave runup elevations, and provides an estimate of the location/alignment and size of coastal flood levees needed to provide flood protection.
- **Section 5. Fluvial Flooding Analyses.** The fluvial section presents the simulation results and implications of the hydraulic analyses (of the final alternatives) for program-level planning, and fluvial impact determination. This section describes the flow regime characteristics of Alviso

Slough and describes the hydraulic modeling approach and water surface elevations from the mouth of Alviso Slough, at Coyote Creek, up to Highway 237 near the community of Alviso.

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3. OVERVIEW OF FLOOD PROCESSES

Flooding in near-shore areas adjacent to the SBSP project sites results from a combination of fluvial (rainfall-runoff) discharges and coastal flooding (U.S. Army Corps of Engineers 1988a; U.S. Army Corps of Engineers 1989). Fluvial discharges include the contribution from primary drainages (rivers and major streams) and secondary drainages (including small creeks, culverts, and pump station or gravity storm water outfalls). Coastal flooding results from exceptionally high astronomical tides, increased by storm surge and wind wave action. Storm surge refers to the increased elevation of water levels due to meteorological conditions, such as the elevation of water surface elevations due to low barometric pressure and the "setup" of the water surface due to on-shore winds. Overtopping of levees resulting from wind-wave induced erosion and wave runup (the maximum vertical elevation above still water) can exacerbate coastal flooding. Near-shore flooding often occurs when coastal flooding conditions and large rainstorm events coincide. These two effects are often correlated, since large winter rainstorms may also cause conditions producing storm surge. During these combination events, the elevation of the tide may inundate upland zones directly, or may prevent rainfall runoff from draining to the Bay, resulting in localized inland flooding.

A discussion of the various vertical land and tidal datums reported herein and the values used to convert between them is provided in Section 4.1.

3.1 Coastal Flood Processes

In combining the water level fluctuations from the astronomical tides and storm surge with the increases in water level elevation due to wave setup and wave runup, and with estimates for future sea level rise, a total increase in the water level elevation relative to the still water level can be defined. For coastal flooding, maximum total water surface elevations depend on local site characteristics as well as regional differences in the flood-generating processes. For example, the slope of the nearshore profile, the type of fronting marsh, roughness coefficients corresponding to levee armoring, and permeability are all important site characteristics that affect wave runup and, in turn, maximum water surface elevations. An important regional consideration is that the maximum tidal elevations increase with distance going south in the Bay, producing higher tides in the Alviso ponds than in the Eden Landing and Ravenswood complexes. This phenomenon results from the shape of the South Bay in conjunction with the tidal characteristics. While storm surge has been considered as a uniform increase above the astronomic tide, surge elevations are likely to vary regionally as well as locally. Thus, the total water level (tide, storm surge and wave runup) may be expected to vary appreciably between each pond complex.

Astronomical tides in the South Bay are mixed semidiurnal consisting of two tides of unequal range that occur each day. On an annual basis, the tides in the South Bay show strong spring-neap variability with the greatest spring tides typically occurring in July and December and the smallest neap tides occurring in April and October. As the tides propagate from the Pacific Ocean into the San Francisco Bay, in the form of shallow water waves, the tide amplitudes and phases are modified by bathymetry, reflections from the shores, the earth's rotation and bottom friction. The enclosed nature of the South Bay creates a mix of progressive wave and standing wave behavior, wherein the wave is reflected back upon itself (Walters et al. 1985). The addition of the reflected wave to the original wave increases the tidal amplitude. Amplification causes the tidal range in the South Bay to increase southward, from approximately 1.77 m (5.8 ft) at the Presidio, to 2.6 m (8.5 ft) at the Dumbarton Bridge, to 2.74 m (9.0 ft) at Coyote Creek, Alviso Slough (at the mouth) (National Oceanic & Atmospheric Administration)(NOAA). As tides propagate up sloughs, tidal range may either be amplified or dampened, depending on the combined effects of friction and momentum. This is exemplified by tides in Alviso Slough, which when monitored at Gold Street Bridge between 1974 and 1976 (National Oceanic & Atmospheric Administration) show tidal wave amplification (tidal range = 2.83 m (9.28 ft)). More recent monitoring for three months in 2004 at the community of Alviso (Moffatt & Nichol Engineers 2005c), following years of sedimentation within the slough, implies that tides are dampened as they propagate up-slough (mean tidal range = 2.06 m (6.75 ft)), and therefore that friction has increased.

Storm surges result from atmospheric disturbances characterized by low pressures and high winds and produce a short-term rise in water elevation. The timing of storm events with respect to the phase of the astronomical tides is critical in defining the water surface elevation. When a storm coincides with a spring high tide, the resulting increase in water elevation can be significantly larger than just the storm surge alone. In addition to storm surge, an El Niño event can produce a substantial difference in still water level along the Pacific Coast and within estuaries. The El Niños of 1982-83 and 1997-98 raised water levels along the Pacific Coast by 0.3 to 0.6 m (1 to 2 ft) in some areas and persisted for several months (Komar and Allan 2004). A broader definition for "storm surge" equal to the difference between measured and predicted astronomic tides is used at times in this report. This definition includes the contribution of climatic conditions such as El Nino and is consistent with use of tide gauge data in engineering practice (see also USACE, (1984a) and FEMA, 2005 for further discussion).

Most of the waves in the South San Francisco Bay are locally generated wind-waves as opposed to swell propagating from the open ocean. The wind direction over the South Bay is typically from the west to northwest in the late spring, summer, and early fall with more variable conditions in winter (Cheng and Gartner 1985). Extreme high winds can arrive from other directions, in particular southerly winds that precede frontal passage and north and northeast winds that can occur during strong thermal gradients in the fall and winter. Due to local winds, the local water level at the shoreline is elevated due to wind setup, wave setup, and wave runup. Wind setup is due to the onshore component of wind stress across the water surface, wave setup is primarily a function of the height of breaking waves, profile slope and wave approach angle, and wave runup mostly depends on wave height, wavelength, and the slope of the levee or embankment.

In addition to tidal and wave processes, sea level rise will also impact flood elevations in the South Bay. The rise in sea level relative to land depends on global eustatic sea level rise and vertical land movements. If the land experiences uplift, the relative rate of sea level rise will be less than the eustatic sea level rise; if the land subsides, the relative sea level rise will be greater than eustatic sea level rise. The historic mean sea level trend at the Presidio, San Francisco between 1906 and 1999 is 2.13 mm/yr (0.70 ft /century). Global estimates by the Intergovernmental Panel on Climate Change (IPCC) average 1.8 mm/yr for the last century but increase to close to 3 mm/yr (0.15 m/50-years) for projections over the next 50 years (IPCC 2001) due to accelerating sea level rise.

3.2 Fluvial Flood Processes

The watersheds bordering the SBSP convey stormwater to the Bay through a network of rivers, creeks, flood control channels and culverts. Flooding has been documented in the lower reaches of virtually every watershed draining to the Bay. Flooding is typically caused by the inadequate stormwater capacity of the receiving waterway. Channels not meeting the capacity of the expected runoff eventually allow levee overtopping or channel capacity limitations resulting in water back-up throughout the storm drainage system. Excessive ponding may occur in topographic depressions due to inadequate or compromised drainage facilities, when subjected to severe storm conditions (Tudor Engineering Company 1973; U.S. Army Corps of Engineers 1988a).

Fluvial discharges result from rainfall runoff conveyed to the Bay by natural or constructed channels. In the South Bay, fluvial flooding often results from the constriction of flows to a relatively narrow corridor, bordered by levees that protect the adjacent developed areas (upstream) or the salt ponds (downstream). During large rainstorms, high flows are constricted by the channel levees, resulting in higher water surface elevations which may overtop levees and inundate the near-channel areas.

From a flood management perspective, potential approaches to reduce fluvial flooding may include increasing channel flow conveyance and/or reducing flood flows (by increasing flood storage capacity /detention). The SBSP restoration project utilizes both approaches to reduce the impacts of fluvial flooding. The benefits and impacts of these approaches are evaluated using fluvial hydraulic modeling. One of the SBSP Restoration Project's objectives is to maintain and/or improve flood protection within the project area.

Increased conveyance results from channel modifications to accommodate a higher flow rate within the channel corridor. This can be achieved by increasing the width or depth of the channel (thereby providing additional cross-sectional area for flow), or in some cases, reducing the channel roughness to increase flow velocity and conveyance. Channel width is usually constrained by adjacent development. In some cases increased depth can be obtained by excavation, or by raising the height of the channel levees.

Modifications to the channel cross-section may change over time due to either erosion (increases conveyance but destabilizes the channel) or sedimentation (decreases conveyance). The cross-sectional area of a stable channel is generally in equilibrium with the amount of water and sediment conveyed on a regular basis. While channel dredging may temporarily provide additional flow area, subsequent sediment deposition will gradually reduce the channel conveyance back to an equilibrium configuration. This is a common problem for most of the fluvial channels in the SBSP area. As a result of the low channel slope in the baylands (resulting in low flow velocities and the potential for sediment deposition) sedimentation has reduced the channel depth and width over time, resulting in reduced conveyance and increased flood hazards. One approach to permanently increasing channel cross-section for these tidal-channels is to increase the amount of daily tidal flow (referred to as tidal prism) in the channels by connecting adjacent or restored tidal wetlands to the channel. The increased tidal flow can provide ongoing scour of existing channels and result in augmented channel conveyance without repeated dredging costs and impacts.

Providing temporary detention storage of floodwater can also reduce flooding impacts by reducing the flow rate in the channels further downstream. Although this has typically been accomplished with reservoirs or basins in the upper watershed, it is a viable approach in the baylands area as well. Off channel detention storage can reduce in-channel water surface elevations, which is an important consideration during very high tides. One approach to providing off channel storage would be to route channel discharge through the restored salt ponds. This could result in a decrease to downstream water levels and reduce upstream water levels and flood hazards.

In the analyses, culverts or weirs between the channel and the ponds would be created to divert flood events into the salt ponds for flood control purposes. The alternatives being considered have the potential to increase the tidal prism resulting in scour of the channel and increased conveyance of flood flows. An expected result is a decrease in downstream water levels within the channel. The lower water surface (and associated flood reduction benefits) would extend for some distance upstream from the Bay to reduce the flood hazards along the drainage-way corridor.

4. COASTAL FLOOD ANALYSES

This coastal flood analysis provides a preliminary flood protection levee alignment and cross-sections for each pond complex.   To provide preliminary estimates of required levee heights, an assessment of potential 100-year water levels was developed. Using available tide and storm data, an extreme value analysis has been performed and 100-year water levels calculated that include the effects of both astronomical tides and storm surge. Lookup tables previously developed by PWA (2005) were used to estimate wave heights and wave runup elevations dependent on fetch lengths and depths for each of the final alternatives. The lookup tables provide planning-level estimates consistent with conceptual analyses. A description of coastal flood protection levee alignment is provided for each pond complex and preliminary levee cross-sections were developed that take into account the predicted total flood water levels and FEMA freeboard requirements.

For a coastal flood protection levee to be recognized by the National Flood Insurance Program (NFIP) and be incorporated into flood hazard maps, the levees must be designed, constructed and maintained to prevent flooding landward of the levee crest during 100-year flood conditions (Federal Emergency Management Agency 2005). To be certified, the coastal flood protection levee must meet the requirements set forth in Title 44 of the Code of Federal Regulations Section 65.10 (Federal Emergency Management Agency 1988). FEMA freeboard requirements for a coastal levee to be certified as providing protection against flooding are defined for 100-year water levels with wind-wave action and without wind-wave action. For the case with no wind-wave action, which defines minimum conditions, the freeboard must be 2 ft above the 100-year water level. For the case with wind-wave action, the freeboard must be 1 ft above the one-percent annual chance wave height or the maximum wave runup elevation (whichever is greater) that is associated with the 100-year water level.

4.1 Datums

Elevations cited within this report are referenced to either the North American Vertical Datum of 1988 (NAVD88) or the tidal datum of Mean Lower Low Water (MLLW). Whereas the National Geodetic Vertical Datum of 1929 (NGVD29) geoid was created using fixed mean sea level (MSL) at 26 tide stations in the US and Canada, the NAVD88 geoid was created by fixing MSL in 1985 at the primary tidal bench mark at Father Point/Rimouski, Quebec, Canada and referencing elevations elsewhere to this primary benchmark (National Oceanic & Atmospheric Administration). The NAVD88 geoid takes into consideration the fact that mean sea level is not the same equipotential surface at all tidal bench marks. Mean sea level elevations relative to NAVD88 elevations are therefore, not consistent.

Due to changes in sea level and vertical land motions, conversions between tidal datums and land-based datums are time-dependent. To account for relative changes, reference benchmarks are routinely re-leveled while tidal datums are updated every epoch.

Water surface elevation data for the extreme water level analysis were obtained and analyzed relative to Mean Lower Low Water (MLLW). Results of selected analyses were converted from MLLW to

NAVD88 using benchmark conversions recently determined from a USGS benchmark survey conducted as part of the San Francisco Bay Bathymetry Study, as yet unpublished by NOAA. These were obtained unofficially from NOAA on January 19, 2006. Table 2a lists the conversions between MLLW and NAVD88 used in this report. Table 2b summarizes the tidal datums for the tide stations discussed within this report in meters relative to both MLLW and NAVD88. The elevations provided herein are believed to be accurate and adequate for planning purposes for this project only and should not be considered generally applicable for design or construction of other projects.

Data used in the HEC-RAS flood model were received relative to NAVD88. Alviso Slough cross-sections used in the UNET and SCVWD HEC-RAS models were originally surveyed in meters relative to NGVD29 and were later converted by the District using a NGVD29-NAVD88 conversion of between 0.835 and 0.839 m (2.74-2.75 ft). This conversion is consistent with the most recent SCVWD NGVD29-NAVD88 conversion for the Alviso area of 0.835 m (2.74 ft).

Table 2a. Conversions between MLLW and NAVD88[1]

Tide Station	Pond Complex	MLLW to NAVD88 conversion (m)[1]
San Francisco/Presidio (#9414290)		0.02
Alameda (#9414750)		-0.07
San Mateo Bridge, West (#9414458)	Eden Landing	-0.23
Dumbarton Bridge (#9414509)	Ravenswood	-0.38
Coyote Creek, Alviso Slough (#9414575)	Alviso	-0.46
Gold Street Bridge, Alviso Slough (#9414551)		-0.6

1- The conversion values listed should be added to elevations in MLLW to convert the MLLW elevations to NAVD88 elevations. Conversely, these values should be subtracted from elevations in NAVD88 to convert the NAVD88 elevations to MLLW elevations.

Table 2b. Tidal Datums in and near the Project Area

Tide	San Francisco/Presidio (#9414290)	Alameda (#9414750)	San Mateo Bridge, West (#9414458)	Dumbarton Bridge (#9414509)	Coyote Creek, Alviso Slough (#9414575)	Gold Street Bridge, Alviso Slough (#9414551)
m MLLW						
Mean Higher High Water	1.78	2.01	2.35	2.59	2.74	2.83
Mean High Water	1.59	1.82	2.16	2.40	2.57	2.65
Mean Tide Level	0.97	1.08	1.26	1.38	1.47	1.49
Mean Sea Level	0.95	1.05	1.25	1.39	1.50	1.52
Mean Low Water	0.35	0.34	0.36	0.37	0.38	0.33
NAVD88	-0.02	0.07	0.23	0.38	0.46	0.60
Mean Lower Low Water	0.00	0.00	0.00	0.00	0.00	0.00
m NAVD88						
Mean Higher High Water	1.80	1.94	2.12	2.22	2.28	2.23
Mean High Water	1.61	1.75	1.93	2.02	2.10	2.05
Mean Tide Level	0.99	1.01	1.03	1.01	1.01	0.89
Mean Sea Level	0.97	0.98	1.02	1.01	1.04	0.92
Mean Low Water	0.36	0.27	0.13	-0.01	-0.09	-0.27
NAVD88	0.00	0.00	0.00	0.00	0.00	0.00
Mean Lower Low Water	0.02	-0.07	-0.23	-0.38	-0.46	-0.60

4.2 Extreme Water Levels

Extreme, or low frequency, high Bay water levels result from the coincidence of storm surge conditions and high astronomical tides. Storm surges are fluctuations in the water level resulting from atmospheric weather forcing (Murty 1984). Low atmospheric pressure systems, high intensity rainfall runoff events, and persistent high winds can cause water to "build up" at the coast. These surges of water produce waves in the period range of a few minutes to a few days and can produce local water levels that vary significantly.

Storm surges on the West Coast of the United States are smaller than those caused by tropical depressions on the East and Gulf Coasts of the United States (Murty 1984). Storm surge research has therefore focused on the East Coast (Gjecik et al. 2004; Peng et al. 2004) and detailed characterizations of storm surges in the Bay have not been undertaken. The following section summarizes the information available on extreme water levels in the South Bay.

4.2.1 Storm Surge & Tidal Dynamics

This section combines knowledge from previous research on general estuarine storm surge dynamics with San Francisco Bay tidal hydrodynamics. A brief analysis of the propagation of storm surges into South San Francisco Bay from tide gauge data is presented. A broader definition for "storm surge" equal to the difference between measured and predicted astronomic tides is used at times in this report. This definition includes the contribution of climatic conditions such as El Nino and is consistent with use of tide gauge data in engineering practice (see also (U.S. Army Corps of Engineers 1984a) and FEMA, 2005 for further discussion). It is recognized, however, that the Bay response to different "storm surge" components may vary with location.

Proudman (1955) modeled the dynamic interaction of storm surges and tides in estuaries. His results showed that, for a progressive surge wave in an estuary of short length, variable width amplifies the tidal range, friction dissipates the tidal wave and reduces the tidal range (high water lower and low water higher than would occur under open ocean conditions) and shallow water accelerates high water and decelerates the low water. For estuaries of greater length, surge amplitudes are greater at the time of low water for progressive waves and greater at the time of high water for standing waves.

In San Francisco Bay, progressive waves moving up-estuary are modified by the bottom bathymetry, shoreline shape, and the Earth's rotation. Figure 4 compares the propagation of the predicted higher high water with the adopted 100-year water levels from the USACE San Francisco Bay: Tide Stage vs. Frequency Study (1984a) (originating from January and December 1983 water level observations). Predicted higher high water levels were estimated by NOAA for primary, secondary and tertiary stations (see Table 1). For primary stations, tidal constituent frequencies are calculated from harmonic analysis of several years of observed data. These frequencies are then used to predict water levels based on the astronomical cycles forcing tides by the motions of the earth, sun and moon (National Oceanic and Atmospheric Administration, http://tidesandcurrents.noaa.gov/). For secondary and tertiary stations, shorter periods of measured data are compared to measured data at primary stations for the same time period and adjustments to times and heights are calculated. These adjustments are applied to the predicted tides at primary stations to then predict tides at secondary and tertiary stations.

Figure 4 compares a simplified profile of high water levels observed in 1983 with predicted astronomic tides. It is unclear whether there is a consistent relationship between the predicted and observed values and therefore, the residual (or storm surge height) does not necessarily propagate uniformly up-estuary. The data indicate a non-linear amplification of water levels in San Francisco Bay may exist for one or more components; however it is unclear how this relationship changes as the combined surge-tidal wave systems moves up-estuary.

To explore storm surge conditions in South San Francisco Bay, differences between San Francisco and South Bay water levels during storms were analyzed in consideration of meteorological conditions. The data records for San Francisco, San Mateo Bridge-West, and Dumbarton Bridge stations capture both seasonal and cyclical climate fluctuations and are of high enough frequency (e.g. hourly, 6-minute) to capture the highest water levels. The tidal hydrodynamics near the tide gauge at Redwood City are not representative of the open South Bay and it was therefore excluded (pers. comm. NOAA staff). Pressure, wind speed, and wind direction (collected from the NCDC San Francisco Buoy and Redwood City for years between 1982 and 2004) were sorted to extract the dates of large storms (short periods of low pressure and high wind speed). Large storms occurring during the period of record for which tidal data were available were categorized based on pressure and wind speed thresholds. The water levels occurring during the periods of low pressure and high wind speed were extracted and compared to the predicted high tide for that day. The differences (residuals) between the observed and predicted high waters were calculated in order to obtain heights reflective of the storm surge. These residuals from San Mateo Bridge- West and Dumbarton Bridge were then compared to those at San Francisco in order to estimate relationships between storm surge behavior in the South Bay, storm surge behavior at San Francisco, and meteorological conditions.

Storm surge heights at San Mateo Bridge-West and San Francisco revealed a consistent relationship and shown graphically in Figure 5. The results show that there is a consistent correlation between storm surge heights at San Mateo Bridge-West and San Francisco (storm surge height at SMB = 1.1*storm surge height at San Francisco) for a variety of wind directions. For northwest winds in particular, the data show that surges at San Mateo Bridge-West are slightly higher relative to San Francisco than under other wind conditions. A strong relationship between surge heights at Dumbarton Bridge and San Francisco is not observed as also shown in Figure 5, possibly due to a limited data set. There does, however seem to be a correlation between wind direction and water level between these two stations. Southerly winds enhance storm surge heights at Dumbarton Bridge while easterly winds enhance storm surge heights at San Francisco. The highest storm surges at both Dumbarton Bridge and San Francisco result during south-southeasterly winds.

4.2.2 Available Data

Tide gauge data for the San Francisco Bay Area is available from the NOAA NOS website (National Oceanic & Atmospheric Administration). In addition, water level data were collected in 2004 as part of the Interim Monitoring Project of the South Bay Salt Pond Restoration Project. Additional tide gauge data were collected in the South Bay by the USGS, PWA, and NOAA.

NOAA Data

NOAA's NOS Center maintains a network of tide gauges. There are 26 NOAA tide gauge stations located near or south of San Francisco in the San Francisco Bay at which data has been collected for at least one month. The locations of these stations are shown in Figure 6 and data inventories are summarized in Table 3. The type and duration of data collected at each station is determined by its designation as primary, secondary or tertiary. Primary tide stations provide a coarsely distributed, continuously operating nationwide network which is supplemented by denser networks of shorter-term operating secondary and tertiary networks.

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Table 3. NOAA NOS COOP South San Francisco Bay Station Inventory

Station	COOP Identifier	Control Station	Record Lengths in Years (Duration) MONTHLY MEANS	MONTHLY EXTREMES	HOURLY/SIX-MINUTE	Notes/Additional Data
PRIMARY STATIONS						
San Francisco-Presidio	9414290	NA	107 (1897 - Feb-05)	107 (1897 - Feb-05)	104 (1901 - Dec-05)	
Alameda	9414750	NA	66 (Apr-39 - Feb-05)	35 (Nov-62 - Feb-05)		Data Gap (extremes): 12/1962 - 12/1969
SECONDARY STATIONS						
San Mateo Bridge-West	9414458	Alameda	15 (May-74 - Feb-05)	15 (May-74 - Feb-05)	0.33 (Jan-05 - Apr-05)	Data Gap: 9/1988 - 12/2004
Redwood City-Wharf 5	9414523	Alameda	9 (Sep-83 - Feb-05)	9 (Sep-83 - Feb-05)	15 (Oct-74 - Dec-05)	Data Gap: 5/1984 - 10/1997
Coyote Creek-Alviso Slough	9414575	Alameda	3 (May-77 - Mar-85)		0.5 (Mar-05)	Data Gaps: 10/1977 - 12/1977; 2/1978 - 1/1979; 3/1979 - 12/1982
Dumbarton Bridge	9414509	Alameda	1.4 (May-77 - Jun-05)	1.4 (Apr-96 - Jul-97)	1.4 (3/1/1996 - Apr-05)	Data Gaps: 6/1977-3/1996; 8/1997-4/2005
Oakland/Alameda Park Street Bridge	9414746	Alameda	1.4 (Apr-77 - Mar-81)	1.25 (Jan-80 - Mar-81)		Data Gap: 6/1977 - 12/1979
TERTIARY STATIONS						
Yerba Buena Island	9414782	Alameda	1.3 (Jun-77 - Aug-93)	0.5 (May-93 - Aug-93)		Data Gap: 11/1977 - 2/1993
Oakland, Matson Wharf	9414779	Alameda	0.75 (Feb-77 - Nov-77)			no current datum, scheduled for Benchmark Update
Palo Alto Yacht Harbor	9414525	Alameda	0.5 (Jun-84 - Dec-84)			
Hunters Point	9414358	Alameda	0.25 (Feb-79 - Aug-79)			Data Gaps: 3/1979; 5/1979-6/1979; scheduled for Benchmark Update
Gold Street Bridge	9414551	Alameda	0.25 (Jan-76 - Mar-76)			
Alameda Creek	9414632	San Mateo Bridge, West	0.25 (Dec-76 - Feb-77)	0.25 (Dec-76 - Feb-77)		
Oyster Point Marina	9414392	Alameda	0.25 (May-79 - Jul-79)			
Mowry Slough	9414519	Alameda	0.17 (Jun-77 - Dec-84)			Data Gap: 7/1977 - 11/1984
Oakland Inner Harbor	9414764	Alameda	0.08 (Aug-94 - May-79)		0.67 (Jan-77 - Jul-77)	8 months hourly data (1/1977-7/1977)
San Leandro Marina	9414688	Alameda	0.08 (May-79 - May-79)	0.08 (Jan-05 - Feb-05)	0.16 (Jan-06 - Feb-05)	
Alameda NAS-Navy Fuel Pier	9414767	Alameda	0.08 (Aug-94 - Aug-94)	0.25 (Jul-94 - Sep-94)	0.08 (Aug-94)	3 months hourly and 6-minute data (7/1994-9/1994)
Oakland Middle Harbor	9414777	Alameda	0.08 (Aug-94 - Aug-94)			
Coyote Hills Slough	9414621	San Mateo Bridge, West			0.33 (Dec-76 - Mar-77)	5 months hourly data (1/1977-4/1977)
San Mateo Bridge, East	9414637	San Mateo Bridge, West			0.33 (Jan-77 - Apr-77)	5 months hourly data (1/1977-4/1977)
STATIONS WITH NO DATUM						
Dumbarton Railroad Bridge	9414510		0.75 (Jun-83 - Mar-85)	0.75 (Jun-83 - Mar-85)	2 (May-83 - May-85)	no current datum, Data Gap: 1/1984-12/1984
Palo Alto Channel Marker 8	9414537		1..25 (Jan-76 - May-77)	1..25 (Jan-76 - May-77)		no datum
Redwood Creek-Channel Marker 8	9414501		1.92 (Apr-75 - Mar-77)			no datum
Newark Slough	9414506		0.33 (Apr-77 - Oct-84)			no datum, Data Gap: 6/1977-7/1984
GPS Buoy, SF Bay	9414796		4.92 (May-95 - Apr-00)	0.92 (May-99 - Apr-00)	1 (5-99 - 5-00)	no datum; hourly data for same period; 3 months 6 minute data

Note: Instrumentation and accuracy information can be found at: NOAA NOS website, http://www.tidesandcurrents.noaa.gov/

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Interim Monitoring Project

A short-term monitoring effort was undertaken as part of the initial planning phase of the South Bay Salt Pond Restoration Project. The purpose of the monitoring effort was to collect information to help characterize physical and chemical processes during a period associated with runoff. Water levels were collected at nine locations between February and May, 2004. The stations and their dates of coverage are summarized in Table 4. Water level data were collected at intervals of 12 minutes. These data were not used in this flood analysis due to limited length of record. These data are useful for understanding the way tides vary by location and up tidal sloughs, and are provided here to facilitate future use.

Table 4. Interim Monitoring Project Data Collection Stations

Station Location	Deployment Period	
	From	To
Alviso Slough	2/7/2004	4/29/2004
Coyote Hills Slough	2/7/2004	4/27/2004
Dumbarton Bridge	2/6/2004	4/1/2004
Guadalupe Slough	2/6/2004	4/29/2004
Power Tower	1/31/2004	4/29/2004
Railroad Bridge	1/24/2004	4/29/2004
Ravenswood Slough	1/24/2004	4/29/2004
Stevens Creek—Mouth	2/18/2004	4/29/2004
Stevens Creek—Upstream	1/24/2004	4/29/2004

Note: Instrumentation and accuracy information can be found at: NOAA NOS website, http://www.tidesandcurrents.noaa.gov/

4.2.3 Water Level Analysis

PWA conducted an extreme value analysis of water levels in and around the South San Francisco Bay project area to confirm the accuracy of 100-year water levels to be used in other South Bay Salt Ponds Restoration Project analyses. Ten years of additional tide gauge monitoring data have been collected and made available since the last 100-year water level analysis (Knuuti 1995). Even including the additional data, this extreme value analysis was still limited by data availability at most locations. For a Flood Insurance Study extreme value analysis, FEMA recommends fitting 30 years or more of observed annual maxima to the appropriate distribution using the method of maximum likelihood (Federal Emergency Management Agency 2005). Hydrodynamic modeling is recommended when sufficient data do not exist for a location.

Of the stations listed in Table 3, an extreme value analysis could only be conducted on these at San Francisco and Alameda tide gauge stations to obtain 100-year water levels. Baseline time series of monthly maximum water levels were obtained from NOS (http://tidesandcurrents.noaa.gov/) for the duration of record for each station. Water level data from previous years were adjusted to reflect sea-level rise to the present. Detrended time series of maximum water levels for each water year were fit to the Gumbel probability distribution. The Gumbel distribution was chosen for use in the maximum likelihood analysis because it provides a good fit at the tails (i.e., where extreme water levels are extracted) (Knuuti 1995).

The method of maximum likelihood was used to estimate the location and scale parameters of the Gumbel distribution for both the San Francisco and Alameda time series. These parameters were applied to the Gumbel cumulative distribution function to estimate the 100-year water levels for each station.

4.2.4 Results

The results of the PWA extreme water level analysis are compared with those of the USACE (1984a) and Knuuti (1995) analyses in Table 5. It should be noted that the USACE (1984a) and Knuuti (1995) results have not been adjusted to account for sea level changes occurring between the date of the study and the present and that Knuuti's results indicate the range of potential water levels within the 95% confidence interval.

The PWA 100-year water levels summarized in Table 5 for stations other than San Francisco and Alameda have been estimated based on measured data from the 1983 storm surge events. The ratio of measured 1983 high water levels at South Bay stations to the San Francisco-Presidio station was used to prorate the Presidio 100-year water level to the local stations. This approach uses a simple, linear amplification factor to provide a very approximate estimate subject to refinement in more detailed studies at the project level. Methods used by USACE (1984a) and Knuuti (1995) are discussed in Section 2.2.1.

Table 5. Summary of Extreme Water Level Study Results

Station Name		USACE (1984)	Knuuti (1995)	PWA 10-Yr	PWA 100-Yr
		m, NAVD88			
San Francisco	1984	2.65			
	1995		2.71 ± 0.05		
	2005		2.72 ± 0.05	2.56	2.66
Alameda		2.90		2.65	2.77
San Mateo Bridge, West (Eden Landing Complex)		3.00		2.89	3.01
Dumbarton Bridge (Ravenswood Complex)		3.10		3.00	3.11
Coyote Creek, Alviso Slough (Alviso Complex)		3.35		3.24	3.36

Note: Most analyses performed in feet. Precision of values reported in feet maintained in conversion to meters.

Variation in calculated 100-year water levels summarized in Table 5 stem from differences in methodology, data type and length of data record. USACE (1984a) employed a tide stage vs. frequency analysis on 129 years (1855 – 1983) of annual maximum tide observations. In contrast, Knuuti (1995) and PWA employed the method of maximum likelihood – extreme value distribution to obtain recurrence probabilities on 98 years (1897 – 1995) and 107 years (1897 – 2004), respectively, of de-trended annual maximum tide observations. Due to evidence of the rise in monthly mean sea levels at San Francisco – Presidio station (2.13 mm/yr, NOAA), earlier analyses would presumably yield lower estimates of 100-year water levels because higher water levels would be captured and averaged into the analyses by later studies. A number of reasons explain why this is not more obvious in Table 5 results. These reasons are:

- USACE (1984a) adjusted the mean of the San Francisco/Presidio tide stage vs. frequency plot to account for the changes in the 20-yr mean annual maximum tides. It is stated in the report that, without adjustment, the 1983 water level plots graphically as a 200-year event and that the frequency curve implies it would be exceeded or equaled once every 4000 years. To account for the trend in mean annual maximum tides and to better capture what they felt to be the more probable frequency of the 1983 water levels, USACE adjusted the mean of the tide stage v frequency plot by 0.16 m (0.53 ft). This is the difference between the 20-year mean of the mean annual maximum tide and the mean of the entire 129 year record of mean annual maximum water levels (see Figure 2). Knuuti (1995) and PWA chose to account for trends in the data differently. In both Knuuti (1995) and PWA's analysis, monthly maximum water levels were de-trended relative to mean sea level at the Year 2005 (PWA) and Year 2000 (Knuuti, 1995). The 100-year water levels cited by Knuuti dated after the reporting date were estimated by applying a sea level rise rate of 1.8 mm/yr to calculated 100-year water levels at San Francisco-Presidio. It appears that the adjustment used by USACE (1984a) was conservative for the time due to the abnormally high water levels of 1983 which pull up the 20-yr mean and misrepresent the average trend over the period of record (see Figure 2).

- The USACE (1984a) computed 100-year water levels for all stations in the South Bay (see section 2.1.1 for methods of extrapolation) were adjusted further to obtain the adopted 100-year water levels (see Figure 7 in USACE report). Computed values did not yield a smooth profile of 100-year water levels in the South Bay and therefore, USACE (1984a) smoothed out computed values to obtain adopted values which generally meant that values were bumped up more to be conservative.

Estimated 100-year water levels for Alameda in particular differ substantially between USACE (1984a) and PWA's estimate. This is primarily due to the methodology employed to calculate the extreme water level. To obtain 100-year water levels at Alameda, USACE (1984a) assumed datum frequencies similar to those at San Francisco, and used MHHW and HET to adjust the tide stage vs. frequency curve created for San Francisco. PWA statistically computed the 100-year water level by performing an extreme value analysis on the data available. This yields a substantially lower 100-year water level for Alameda than previously estimated by USACE (1984a) because tidal hydrodynamics at Alameda, like Redwood City, are not representative of those at open Bay stations. Because Alameda station is within a harbor, the tidal hydrodynamics are modified from those in the Bay, and therefore the tide stage vs. frequency curve at

San Francisco would not appropriately represent tides at Alameda. It is interesting to note that the Alameda and Redwood City tide gauges have been used by NOAA NOS as reference stations for their south Bay tide gauges, possibly affecting tidal datum information for the south Bay. The mean sea level rise data for south Bay tide gauges was apparently affected by use of the Alameda gauge as a reference station (Moffatt and Nichol Engineers 1988), leading to questions about the accuracy of the tidal datums for these gauges.

For planning purposes, it is necessary to take into account historic and future trends in extreme water levels resulting from storm surges. Storm surges are dynamically tied to meteorological conditions, and therefore, long-term climate changes will impact the height and frequency of extreme water levels. Application of the extreme water levels in Table 5 should take into consideration the following:

Joint Occurrence of High Water Levels, Winds and Wind Waves: A 100-year coastal flood level was selected for levee sizing based on FEMA Guidelines (FEMA, 2005). The 100-year coastal flood level is associated with the 100-year still water elevation in the absence of wave action. Where wave action is present, a total water level is estimated that includes wave setup and runup on the levee, called the "total water level." South San Francisco Bay is subject to locally generated wind waves. Hence, the primary forcing parameters affecting coastal flood potential are Bay water levels and winds, and the governing response parameter is either "still water level" or "total water level," depending on the fetch available for wind wave generation.

The actual joint probability of occurrence of wind speed and water levels is not defined for south San Francisco Bay. However, prior studies indicate that a combination of water level and wind speed with a joint recurrence interval of much more than 100 years (typically 200 to 500 years) is required to result in the 100-year wave runup (Garrity et al. 2007; PWA 2004a; Wallingford 1998). Therefore, methods using long data time series (either real or synthetic) and or probability methods are required to calculate the flood response (Federal Emergency Management Agency 2005; U.S. Army Corps of Engineers 1996). These analyses are beyond the scope of this study. Therefore an "event selection" method was used where the forcing parameters are selected to result in an extreme response that is assumed to be approximately equal to the 100-year runup. Usually, multiple "events" with similar joint probabilities are selected and the worst-case runup is used (Federal Emergency Management Agency 2005; Garrity et al. 2007).

The implicit assumption is that high winds and high water levels are partly but not completely dependent. To clarify, complete dependency would require a 100-year wind speed concurrent with a 100-year water level. In contrast, complete independence would result in the selection of a 1-year or lower speed. Other factors such as timing and duration are important elements of the analysis.

The 100-year recurrence interval water level was calculated owing to its importance in flood mapping efforts in South San Francisco Bay. In order to estimate the 100-year recurrence wave runup elevations, a concurrent wind condition was needed (the wind speed affects water levels and wave conditions). A 10-year recurrence wind speed was selected based on judgment and prior studies (Coulton et al. 2002; PWA 2002a; PWA 2002b). Only one event was selected in this study, and consequently, neutral to conservative assumptions are required with this approach. For example, the 10-year wind speed is applied along the “worst-case” direction and during the tidal stage that results in the maximum wave height.

Sea-Level Rise: Mean sea level has been rising at a rate of 2.13 mm/yr (0.70 ft/century) at the San Francisco-Presidio tide gauge since 1906 (National Oceanic & Atmospheric Administration). Global data suggest that rates of sea-level rise did not accelerate during the last century. However, projections suggest acceleration of sea-level rise throughout the present century dependent upon which greenhouse gas emissions scenario is used to force the model (IPCC 2001). The mean sea level rise projection for the next 50 years, based on the average greenhouse gas emissions scenario is 0.15 m (0.5 ft) (IPCC 2001). The projected value of 0.15 m (0.5 ft) of sea level rise over the next 50 years was used for this analysis. The California Climate Change Center recommends using a mean sea level rate of rise of 3.3 mm/yr for planning purposes to 2100 (Cayan et al. 2006), which amounts to 0.165 m (0.54 ft) in 50 years. It is important to note that the IPCC projections do not include the contribution of changes in ice sheet melting to sea level rise due to significant difficulties in predicting these contributions. The state of the science of sea level rise has been changing very rapidly recently. Therefore, a greater sea level rise rate may be appropriate when assessing future flood risks.

Future extreme water levels, however, may also be affected by modifications to the tidal hydrodynamics which result in changes to the tidal range. Tide gauge data from San Francisco shows an upward trend in both diurnal and mean tide range of 64 and 60 mm/century, respectively, since 1900. MHHW at San Francisco between 1855 and 1999 rose at a rate of 258 mm/century, which is 16% faster than the rate of mean sea-level rise (Flick et al. 2003).

Storm Frequency: While recent studies (Bijl et al. 1999; Pugh and Maul 1999; Woodworth 1999; Zhang et al. 2000) have concluded that there has been no discernable global trend in storm activity for the last century, it is difficult to project future changes in storm activity and resulting extreme water levels. Woodworth and Blackman (2004) concluded from analysis of 141 tide gauge records that there is evidence for a global increase in extreme high waters since 1975, but that the changes are similar to those of mean sea level and are most likely the result of the same type of atmospheric/oceanic forcing.

Wind Setup: Increases in water levels due to storm surge are impacted by the stress of the wind acting over the water surface (wind setup) and therefore, changes to prevailing wind patterns may affect extreme water levels. To estimate amplification due to wind set-up, FEMA (Federal Emergency Management Agency 2005) recommends a simplified 1-dimensional wind surge model. A brief analysis of wind setup using this model was conducted for the South San Francisco Bay. Model input consists of both a 10-year wind equal to approximately 40 mph (for San Francisco Airport between 1948 and 1995), bathymetric data normal to the locations analyzed, and tide levels corresponding to the 100-year predicted still water elevations. Using the PWA 100-year water levels from Table 5 and a constant 10-year wind speed, wind setup results from the 1-dimensional model for each SBSP pond complex are shown in Table 6.

As an alternative approach, wind setup was also calculated at each pond complex by applying the equation of motion in the two dimensional vertical plane directed toward the shore. This equation was simplified by retaining only the sea surface slope and wind stress terms resulting in the following equation:

$$s = -d \pm (d^2 + 2(\tau_{sx} / \rho g)L)^{1/2}$$

where; d = water depth in meters, L = length in meters over which the wind blows, s = wind setup in meters, τ_{sx} = wind stress in kilograms per meter squared second, ρ = density of water in kilograms per cubic meter, g = gravitational acceleration in meters per second squared, and $\tau_{sx} = \rho_a CW^2$ where ρ_a = density of air in kilograms per cubic meter, C = $2.28*10^{-3}$ and W = wind speed in meters per second.

This equation was applied over shallower depths closer to shore and yields slightly higher results for wind setup. These estimates can be used to more conservatively approximate potential wind setup.

Table 6. Wind Setup Values for Onshore Winds

Pond Complex	BATHYS Wind Setup	2D Equation of Motion
Eden Landing	0.03 m (0.1 ft)	0.28 m (0.9 ft)
Ravenswood	0.06 m (0.2 ft)	0.22 m (0.7 ft)
Alviso	0.12 m (0.4 ft)	0.27 m (0.9 ft)

The purpose of this calculation is to approximately estimate the additional increase in water level due to local wind setup, to be added to the 100-year water level estimated for the vicinity based on tide gauge data. The values used are consistent with the event (100-year water level and 10-year onshore wind speed and wind waves) selected to approximate a 100-year coastal flood event. However, actual wind setup may be much larger, especially during lower water levels. Wind setup magnitudes during strong winds in south San Francisco Bay can be expected to be on the order of 0.3 meters (1 foot). The higher of the two sets of wind setup values in Table 6 were used for subsequent calculations in this analysis.

Overall, further research is necessary in the process of analyzing extreme water levels in the South Bay. There are not sufficient data in both duration and quantity to calculate extreme water levels at stations other than San Francisco and Alameda. It may be possible to calculate extreme water levels in the South Bay in the future with the collection of more data however more accurate results may be obtained from storm surge models that replicate the geometry of the estuary and meteorological dynamics.

4.3 Coastal Flood Hazard Management

To estimate preliminary crest elevations and other geometry characteristics for the coastal flood protection levees surrounding the SBSP project area, total water levels are estimated for two different cases: (1) with wind-wave action across the project area and (2) with no wind-wave action. Per FEMA requirements, the total water level with no wind-wave action (also called the "still water level") defines a minimum condition for protection against coastal flooding and the total water level with wind-wave action defines the required condition for protection where waves are present. For each pond complex in the SBSP project area, the flood protection levees have been categorized into three different wind-wave exposure levels: (A) flood protection levees with an outboard marsh, (B) flood protection levees without an outboard marsh, and (C) flood protection levees with an outboard managed pond. In addition to providing preliminary flood protection levee cross-sections, levee alignment for each pond complex and each levee category is also described.

4.3.1 Flood Water Levels

“Lookup” tables were constructed to help quantify potential coastal flood hazards and flood water levels due to local wind-wave action. Approximate parametric wind wave hindcasting equations from the USACE Coastal Engineering Manual (U.S. Army Corps of Engineers 2002a) were used. These equations largely neglect shallow water effects and therefore may overestimate the wind wave heights in south San Francisco Bay. A comparison was made between parametric equations in the CEM and in the older USACE Shore Protection Manual (U.S. Army Corps of Engineers 1984b) which do include the effects of shallow water (unpublished project technical memorandum, (PWA 2004b)). In deep water, the wave heights calculated using the SPM method were about 25% higher than the CEM hindcast wave heights. The difference decreases with decreasing depth to wave length ratio (d/L ratio) and when the d/L ratio is close to 0.2, the SPM and the CEM calculated wave heights are approximately equal. When the d/L ratio decreases further (less than d/L~ 0.2), the CEM derived waves are higher than the SPM derived heights. Wave periods predicted by the SPM methods were higher for most of the cases tested (deep and shallow) except for two of the intermediate water conditions. More detailed studies should employ two-dimensional wind wave models (U.S. Army Corps of Engineers 2002a).

In general, simplified, engineering methods were used to approximately calculate extreme high water levels, waves and wave runup. Deterministic equations based on simplified solutions and empirical data were used, consistent with the US Army Corps of Engineers Coastal Engineering Manual (U.S. Army Corps of Engineers 2002a) and the Final Draft Guidelines for Flood Hazard Analysis and Mapping for the Pacific Coast of the United States (Federal Emergency Management Agency 2005). This approach allowed an efficient analysis using a range of parameters in the form of spreadsheet-based look-up tables. The results are approximate but adequate to define a range of levee crest elevations for program-level planning and evaluation. Actual values developed in a project-level study could be substantially different owing to the complexity of the natural processes, methods used, etc. Consequently, a range of values was produced. The low end of the range was based on the still water level and is generally similar to the approach used by the USACE to previously estimate the 100-year water levels in the shoreline study. The 100-year still water levels estimated by the USACE were subsequently adopted as the Base Flood Elevations by FEMA. While it is possible that new studies would result in lower flood elevations, we believe this to be unlikely. Consequently, the calculations were focused on estimating the high end of the likely range. "Conservative" assumptions were used in some steps to err on the high elevation side, so that the range of estimated levee crest elevations would "bracket" actual elevations to be calculated subsequently at the project level. Engineering judgment was used to accomplish the work within the available time and budget constraints without use of the results of the more detailed USACE and FEMA studies that were anticipated but not initiated in time to be useful to this Flood Analyses Report.

To establish flood water levels, wave runup elevations have been estimated for each SBSP complex and each different levee exposure level. Input parameters are wind conditions, levee geometry, roughness factors due to fronting marsh and reduction factors due to berms. For this programmatic level of analysis, wave runup is predicted based on the assumption that the joint occurrence of a 10-year wind event and a 100-year extreme water level produce the one percent annual chance wave height or maximum wave runup elevations on the flood protection levee slopes. The lookup tables allow for the evaluation of multiple wind-wave fetches, but only the worst case fetches have been used to estimate wave heights and maximum wave runup conditions for preliminary levee cross-sections.

The analysis indicates that a 10-year wind speed on the order of 40 mph could potentially create waves as high as 6 foot with periods of 4 seconds along the outboard edges of each pond complex. However, once these waves begin to propagate across the project area during a flood event, they will be attenuated due to shallow water conditions. A method proposed by Camfield (Camfield 1977) to compute wave attenuation across shallow flooded areas, where bottom characteristics include vegetation and frictional effects, has been applied to each pond complex. However, predicted values using the Camfield method were approximately equal to depth-limited conditions. Thus, a simple approximation of wave height equal to 0.6 times the depth, as recommended by the Coastal Engineering Manual (U.S. Army Corps of Engineers 1984b), was used to predict wave heights adjacent to the flood protection levees. For each levee category and pond complex, wave heights and wave runup heights, including wave setup, for a 10-year wind and 100-year water levels from Table 5 are shown in Table 7. In Table 7, H_{mo} is the significant wave height and $R_{2\%}$ is the value for wave runup height (above the water level) that is exceeded 2% of the time during the extreme event. Wave setup of about 0.3 m (1 ft) was added to the $R_{2\%}$ values reported in Table 7.

Table 7. Wave Heights & Runup ElevationsRunup Heights[1]

	Eden Landing		Alviso		Ravenswood	
Levee Category	**H_{mo} (m)**	**$R_{2\%}$ (m)**	**H_{mo} (m)**	**$R_{2\%}$ (m)**	**H_{mo} (m)**	**$R_{2\%}$ (m)**
Without outboard marsh (Tidal Habitat)	0.8	1.8	0.9	2.0	0.8	1.8
With outboard marsh (Upland Transition Area)	0.8	1.2	0.9	1.3	0.8	1.2
With outboard managed pond (Managed Pond)	0.5	0.8	0.8	1.2	0.4	0.7

1- Wave setup of about 0.6 m (1 ft) is included in the wave runup height ($R_{2\%}$) values.

Table 8a shows the components used to estimate minimum and maximum levee crest elevations for the "without outboard marsh (tidal habitat)" levee condition. For the minimum elevation, the total water level is estimated by adding 100-year extreme water levels from Table 5 with wind setup from Table 6, 0.6 m (2 ft) of freeboard, and an allowance of 0.15 m (0.5 ft) for sea level rise. This approach is similar to

FEMA's minimum condition. However, the estimated minimum levee crest elevations in Tables 8a and 8b are higher than the existing FEMA flood levels due to the inclusion of wind setup and sea level rise. For FEMA's maximum condition, the total water level is estimated by adding 100-year extreme water levels from Table 5 with the following estimated values: wind setup, wave setup and runup (or the one-percent annual wave height, if it is greater than wave runup), and 0.3 m (1 ft) for freeboard. Even though FEMA does not require consideration of future sea level rise, a value of 0.15 m (0.5 ft) was added to the total water level (maximum) estimates. Land subsidence and settlement are not included and should be added or otherwise addressed as appropriate. Minimum and maximum condition levee crest elevations for each pond complex and each levee category are shown in Table 8b.

The wind setup in Tables 8a and 8b conservatively account for the higher of the two estimates for wind set-up (see Table 6). Note that levee crest elevations in Tables 8a and 8b were calculated in feet and rounded up to the nearest foot and then converted to meters.

Table 8a. Components of Minimum and Maximum Levee Crest Elevations

	Eden Landing		**Alviso**		**Ravenswood**	
	Minimum	**Maximum**	**Minimum**	**Maximum**	**Minimum**	**Maximum**
Component	***m, MLLW***					
100-year SWL (m NAVD) [1]	3.01	3.01	3.36	3.36	3.11	3.11
100-year SWL (m MLLW) [2]	3.24	3.24	3.82	3.82	3.49	3.49
Wind setup (m) [3]	0.28	0.28	0.27	0.27	0.22	0.22
SLR (m)	0.15	0.15	0.15	0.15	0.15	0.15
Wave setup and runup (m) [4]		1.8		2.0		1.8
Freeboard (m)	0.6	0.3	0.6	0.3	0.6	0.3
Levee crest elevation (m MLLW)	4.3	5.8	4.9	6.7	4.6	6.1
Levee crest elevation (ft MLLW)	14	19	16	22	15	20

1- From Table 5

2- From Tables 2a and 5

3- From the higher of the two estimates for wind set-up in Table 6

4- From Table 7

Table 8b. Miniumum and Maximum Levee Crest Elevations

	Eden Landing		Alviso		Ravenswood	
	Minimum	**Maximum**	**Minimum**	**Maximum**	**Minimum**	**Maximum**
Levee Category	***m, MLLW***					
Without outboard marsh (Tidal Habitat)	4.3	5.8	4.9	6.7	4.6	6.1
With outboard marsh (Upland Transition Area)		5.2		6.1		5.5
With outboard managed pond (Managed Pond)		4.9		5.8		4.9

4.3.2 Levee Alignment

The South Bay is currently protected from coastal flood hazards by an ad hoc combination of salt pond levees, some areas of high ground, and some engineered flood levees. Under the No Action alternative, portions of the existing flood defense system will most likely be abandoned because the former salt ponds do not presently require the protection they once did under salt production operation. To maintain flood protection for those areas behind the salt ponds, portions of the former salt pond levees will be maintained/improved for flood protection. The mix of coastal flood levees providing present and future flood protection under the No Action alternative in each of the three project areas is shown in Figure 7. Because of the non-engineered construction of the salt pond levees, these do not meet FEMA criteria for flood levees. Thus, many coastal areas behind the salt pond levees are currently included in the areas of mapped 100-yr floodplains by FEMA. As such, there are development restrictions in these areas, and owners are encouraged to purchase flood insurance.

One of the goals of the SBSP Restoration Project is to maintain or improve flood protection in the project areas and for developed areas landward of the project area. The SBSP Restoration Project is committed to ensuring that future flood protection with the Project is equal to, or better than, existing conditions. Beyond this, it is desirable by all entities to develop a flood management program around the SBSP Restoration Project area that would provide a consistent level of flood hazard management with flood protection measures (levees, high ground) meeting both FEMA and Corps criteria. The Project expects to be able to achieve this objective. However, the actual level of protection over and above existing would depend on a number of considerations, but most important is funding.

In many locations, the perimeter levee will follow the alignment of the existing inboard salt pond levee. The alignment of the proposed perimeter flood protection levees is shown in Figure 8 and Figure 9 and described in subsequent sections. The proposed alignments are identical for both Alternatives B and C. The location shown represents the current preferred alignment, based on input from landowners, stakeholders, and local flood protection agencies. However, it is subject to refinement during subsequent detailed design studies.

For cases in which the proposed alignment follows the alignment of existing inboard levees, the existing levee may require improvement including base preparation (keyway, compaction etc), and subsequent expansion in basewidth and height to comply with FEMA criteria. It will tie into existing flood control levees or high ground to provide a continuous system of engineered flood management. It should be noted that in Figure 7, Figure 8, and Figure 9, levees shown as "existing flood levee" may still require some improvement to comply with FEMA standards. However, the extent of alteration to bring the levees into FEMA compliance, and associated impacts should be less than that required to upgrade existing salt pond levees. At each of the major fluvial/creek channels, the coastal levee will connect with the fluvial channel levee. At these confluence locations, the required levee crest elevation will be estimated by the maximum design water elevation, which may be coastal, fluvial or a combination of a joint fluvial-coastal flood event.

The improved levee will cross a number of utility corridors, including pipelines, power transmission lines, access roadways, etc. Protection of and continued access to these facilities will be required. In addition, the alignment will intersect the rail line in one or two locations, requiring design consistent with rail operation. Storm drains and surface runoff will likely be affected and considered in the levee designs.

Eden Landing

Beginning at the northern end of the Eden Landing pond complex, the levee will intersect with Highway 92 at the San Mateo Bridge. It will provide protection to the bridge areas, and allow roadway drainage as required. It will extend westward, tying into the existing high ground along the bridge. Extending eastward, the levee will be constructed along the new trail/levee being constructed as part of the Eden Landing Restoration project. It will then tie into the levee/high ground around the recently constructed Eden Shores development in Hayward, which extends to the engineered flood control levee on the north side of Old Alameda Creek (OAC).

Continuing along the South bank levee along OAC, the levee will be constructed on the rear levee of ponds E6 and E5 and then tie into the existing landfill. It will continue south behind E4C and E3C, then extend eastward to connect with the existing engineered levee along the Alameda Ck Flood Control Channel.

Alviso

The northeastern extent of new levee work in the Alviso complex will begin around the backside of Pond A22 and A23. These will tie into the fluvial levees along Laguna Creek. South of here, the flood control limit will extend on the backside of the Coyote Creek Lagoon (a.k.a. Warm Springs Marsh), and tie into the flood control channel under I-880, and the high ground of the landfill. From here, it will link into the extensive levees and flood control elements of the Coyote Creek Flood Control system, constructed over the past decade by the District. On the rear side of Pond A18, the flood control limit will follow the existing higher ground of the Sewage Treatment Plant ponds and facilities, tying into Artesian Slough. On the west side of Artesian Slough, a major levee improvement will be constructed on the north border of the New Chicago Marsh, providing flood management for the town of Alviso. This will link into the existing improved levees along Alviso Slough/lower Guadalupe River.

On the north side of Alviso Slough, levees will link into the high ground of the landfill, before continuing west behind Pond A8 and A4. The levee will extend bayward around the bayside of the Sunnyvale Treatment ponds, and then continue along the backside of Pond A3W to protect the Lockheed property. It will continue west around Moffett Field behind pond A2E, then connect with the Stevens Creek fluvial levees. West of Stevens Creek, the high ground of Mountain View Shoreline Park provides flood protection west to approximately the Palo Alto Flood Basin (PAFB).

Some improvements are expected to the perimeter levee of the PAFB and the levee behind the Palo Alto Baylands/Airport (extending to the San Francisquito Creek levee improvement project currently being developed by the JPA as part of the SSF Bay Shoreline Study). Potential improvements in this area are not shown on the current maps.

Ravenswood Complex

A new levee will be constructed along the south side of Highway 84 (approach to the Dumbarton Bridge) to protect the roadway. The levee will turn southward along the backside of SF2.

A similar new levee will be constructed on the north side of Highway 84, along the backside of Pond R2. This will connect with the existing engineered levee around the perimeter of the Sun Microsystems complex, and then extend west on the backside of Pond R3. It will turn north to isolate ponds R5 and S5 as managed ponds, and tie into the high ground at Bayfront Park.

4.3.3 Preliminary Levee Cross-Sections

A concept-level design of each levee category meeting the FEMA maximum condition of coastal analysis is developed for each SBSP complex, assuming an earthen levee with trapezoidal shape. Conceptual levee cross-sections for a levee without an outboard marsh (maximum exposure), with an outboard tidal marsh (moderate exposure), and with an outboard managed pond (minimal exposure) are shown in Figure 10, Figure 11, and Figure 12. These conceptual levee cross-sections are preliminary and are meant for a programmatic level of evaluation only.

The conceptual levee cross-sections also show crest elevations that correspond to the FEMA minimum condition for coastal protection. A range of crest elevations are provided as estimates of those required to be certified by FEMA. For the flood protection levee with an outboard marsh, it is assumed that the marsh will act as barrier with a high degree of roughness due to emergent marsh vegetation. For the levee case without an outboard marsh, it is assumed that a tidal bench will be constructed to break incident waves, minimize levee erosion, and create an environment for vegetation to be established. For the levee case with an outboard managed pond, the outside levee is assumed to act as a wind-wave fetch break and reduce transmitted wave energy. Armoring against wave erosion will be required but is not addressed in this report.

Pond bottom elevations shown on the conceptual levee cross-section figures are from recent surveys (Foxgrover and Jaffe 2005; TerraPoint 2005) and internal managed pond outboard levee elevations are approximate and within the range reported by Siegel and Bachand (Siegel and Bachand 2002). Average elevations for the Alviso and Ravenswood existing perimeter levees are from Moffatt & Nichol (Moffatt & Nichol Engineers 2005b). The elevation of the Eden Landing perimeter levee is approximate and taken from (Siegel and Bachand 2002).

Unless otherwise noted, levee slopes for each of the three levee exposure categories were assumed to be 3:1 (H:V) and levee crest widths are 4.6 m (15 ft). Stability berms will likely be required to prevent subgrade failure that may otherwise occur due to the rapid placement of soil during construction. These berms will be needed on both sides of a flood protection levee. Stability berms are assumed to have crest elevations approximately 0.6 m (2 ft) above MHHW. The widths of the stability berms will vary depending on the type of levee and assumed to have side slopes of 3:1 (H:V).

The conceptual levee cross-sections will be refined during subsequent design based on site-specific topography, geotechnical analysis, and other site-specific considerations. The levee cross-sections described in this report are “post-settlement” geometries. Typically, levees are “over-built” to allow for initial settlement that results from consolidation of the subgrade in response to the weight of the new fill. Additional long-term settlement is typically addressed by raising the levee crest as needed or adding flood walls. In addition to “over-building,” possible cross-sectional changes include revisions to stability berm dimensions and levee slopes. These changes are not expected to affect the findings in this report.

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5. FLUVIAL FLOOD ANALYSES

This section provides a description of the fluvial modeling strategy and methodology for the Guadalupe River / Alviso Slough analyses. The methodologies were developed to identify and examine potential fluvial flood management impacts of the proposed restoration alternatives (including that of the No Action Alternative). The detailed fluvial flood analysis was conducted on Alviso Slough/Guadalupe River to focus on the potential impacts along a system that currently benefits from offline storage in the salt ponds. Other fluvial systems not using the salt ponds for flood storage can be expected to show higher levels of flood reduction benefits.

Alviso Slough extends from the Coyote Slough to the UPRR Bridge crossing in Alviso. The slough is the major Bay connection for the Guadalupe River and the 170 square miles of watershed land to the south. The Slough is tidal from the Bay upstream to Montague Expressway (a distance of 7.1 miles upstream) (Santa Clara Valley Water District 2001). The tidal reach of the Alviso Slough is a depositional environment characterized by low channel slopes and low energy conditions, and siltation has been gradually reducing the depth and cross-section of the slough ever since the salt pond construction.

The hydrologic conditions (flow regime) of the Guadalupe River watershed have been described in prior reports (Jones & Stokes 2001; PWA et al. 2005; Santa Clara Valley Water District 2001). The watershed hydrology has been altered by changes in land use and reservoir operations. The upper-basin water supply reservoirs are not operated for flood protection purposes though they do provide flood management benefits (Santa Clara Valley Water District 2001). Existing District and USACE approved peak flow rate and hydrographs were used in subsequent hydraulic modeling.

5.1 Hydraulic Modeling Methodology

The hydraulic modeling approach was developed from prior flood studies using District approved hydrodynamic models. The initial step in the hydraulic modeling study was to develop an existing conditions "steady-state" hydraulic model that could subsequently be expanded to an unsteady state model. An unsteady (dynamic) analysis approach is important to describe changing boundary conditions and/or other factors influencing capacity (tides, vegetation, marsh accretion, sedimentation, levee overtopping, offline storage, etc.). This analysis began with the existing Guadalupe River/Alviso Slough HEC-RAS model, obtained from the District. The model uses a 1D, steady-state modeling approach that describes the conveyance of flood flows from the Lower Guadalupe River, through Alviso Slough, to the San Francisco Bay. PWA updated the HEC-RAS model using 2004 survey data to develop the Existing Conditions Model (steady-state). The Existing Conditions Model was run in a steady-state condition to develop an effective model that adequately represented existing topographic and hydrologic conditions. Two scenarios were run using the Existing Conditions Model. The first scenario uses the 10-year Bay tide water elevation with 100-year fluvial peak flow rate and serves as the basis for the alternatives modeling. The second scenario examines the 100-year Bay tide with the 10-year peak fluvial flow. Each boundary condition used in the two steady-state model scenarios were provided by the District.

Existing Conditions HEC-RAS Model (steady-state)

- EC (10-year still water elevation with 100-year fluvial peak flow rate)
- EC100 (100-year still water elevation with 10-year fluvial peak flow rate)

The second step in the hydraulic modeling approach was to obtain and update an unsteady-state hydraulic model from the District. The available unsteady, spatially varied flow model was developed as a planning tool to evaluate the hydraulic characteristics of the 25-square mile Baylands Study area that includes: Alviso Slough; Guadalupe Slough; Coyote Creek; and the adjacent salt ponds. The Existing Conditions Model and the unsteady-state hydraulic model from the District were combined to develop a 1D (quasi 2D flow), unsteady-state HEC-RAS model to simulate the river and slough hydraulics under existing conditions. Sensitivity tests were conducted on the model to ensure the results from the earlier models were being represented correctly in the combined model.

The unsteady-state model was then used to describe the system functioning under both baseline conditions and future conditions, with and without the proposed SBSP project. Various short-term and long-term scenarios were used to model the with-project alternatives. The following eight hydraulic model scenarios were developed for the study.

Baseline Conditions HEC-RAS Model (unsteady-state)

- BC Short-term

Alternative A — No Action

- A Long-term (50-years in the future, typical)

Alternative B — Managed Pond Emphasis (50:50 Ratio of Tidal Habitat to Managed Pond)

- B-1 Short-term
- B-2 Long-term (no channel scour)
- B-3 Long-term (with channel scour)

Alternative C — Tidal Habitat Emphasis (90:10 Ratio of Tidal Habitat to Managed Pond)

- C-1 Short-term
- C-2 Long-term (no channel scour)
- C-3 Long-term (with channel scour)

The current flood management program for the lower Guadalupe River uses the storage capacity of ponds A8, A7, A6, and A5 as an integral part of the flood management scheme. In the potential future alternatives for ponds A8, A7, A6, and A5, tidal circulation would be allowed in the ponds. Under these conditions the available overflow flood storage in the ponds might be less depending on the tide level. However, removing or lowering the levees along the channel would increase flow capacity by widening the cross sectional area of the flow path. The unsteady modeling examines each scenario to compare the substitution of offline storage attenuation with an increase in channel conveyance. The reduced offline storage occurs when slough levees are removed and ponds A8, A7, and A6 are no longer constrained in a low water condition since they will be subject to tidal and fluvial flows after breaching. Increased channel conveyance is a result of slough morphologic changes in channel shape (increased channel area results from widening and deepening) in response to the increased tidal flow between the Bay and its pond. These changes were estimated using the “hydraulic geometry” analysis.

Hydraulic geometry analysis provides empirical relationships that are used to predict channel depth, width and cross-sectional area as a function of tidal prism or contributing marsh area (Williams et al. 2002). Hydraulic geometry calculations were performed for the No Action alternative and the restoration alternatives. Alternative A, the No Action alternative, assumes the west bank levee along ponds A6 and A7 will fail at specific predicted (unplanned) breach locations causing an increase in the tidal prism and subsequently an increase in the channel geometry downstream of these breaches. The channel geometry for Alviso Slough was increased for Alternatives B-3 and C-3 models. The channel volume change results in a reduction or increase in the water surface profile for each model.

Model Calibration

The District’s HEC-RAS model was calibrated using observed flow rates and water levels for the 1995 event. The Baseline Conditions Model was created in part from the District’s HEC-RAS and UNET models and new survey information from 2004. The Baseline Conditions Model was compared to the District’s HEC-RAS model and the District’s UNET model. This comparison showed that the model output maintained good correlation through the transition from the UNET to the Baseline Conditions Model (Table 15 and Table 16). However, the Baseline Conditions Model has not been calibrated with any recent rainfall or tide event that would be consistent with the current model parameters.

Model Review / Santa Clara Valley Water District Coordination

PWA staff performed a Quality Assurance (QA) review of the hydraulic parameters used to setup the hydraulic model. The QA review included sensitivity tests on the boundary conditions, initial conditions, hydraulic connections, and other empirically-derived or subjective inputs to the model.

Several meetings were conducted by PWA staff with the District to discuss the modeling criteria that lead to development of the current project. On November 3, 2005, PWA’s modeling staff met with the District’s technical leads for the Guadalupe River watershed to discuss the modeling methods, setup, and preliminary results. Ultimate model configurations were selected based on this input. On June 13, 2006, PWA again met with the District’s staff to confirm the appropriateness of the modeling approach, results, and reporting in the administrative draft Lower Guadalupe River / Alviso Slough Flood Modeling Memorandum. The District’s comments on the memo have been addressed and are incorporated in this report.

5.1.1 USACE HEC-RAS Program

The hydraulic modeling program used in this study was HEC-RAS version 3.1.3 (Hydrologic Engineering System – River Analysis System, May 2005). HEC-RAS is a widely used program developed by the U.S. Army Corps of Engineers (USACE) for 1D hydraulic calculations in natural and constructed channel systems (U.S. Army Corps of Engineers 2002b). The HEC-RAS program can simulate steady and unsteady flow evaluation in single or networked channels. HEC-RAS computes channel water levels based on the one-dimensional energy equation. Energy losses are represented by channel expansion/contraction and by friction losses. The model also allows for inclusion of storage areas, storage area connections, bridge hydraulics, culverts, gates and weirs. HEC-RAS replaces the program HEC-2. HEC-RAS version 3.0.0 replaces the one-dimensional unsteady flow simulator UNET.

HEC-RAS UNET Modeling Capability

In addition to solving the one-dimensional unsteady flow equations in a network system, the HEC-RAS UNET capabilities provide the ability to apply several external and internal boundary conditions, including; flow and stage hydrographs, gated and uncontrolled spillways, bridges, culverts, and levee systems. The UNET unsteady flow simulation is a three-step process. First a program called RDSS (Read DSS data) is run. This software reads data from a HEC-DSS file and converts the boundary condition time series data into the user specified computation interval. Then the UNET program is run. This software reads the hydraulic properties tables computed by the preprocessor, as well as the boundary conditions and flow data from the interface and the RDSS program. The program then performs the unsteady flow calculations. The final step (called "TABLE") takes the results from the UNET unsteady flow run and writes them to a HEC-DSS file (U.S. Army Corps of Engineers 2002b). HEC-RAS has the capability to import and run UNET models developed independently.

5.1.2 District's HEC-RAS Model (steady-state)

The District's Guadalupe River / Alviso Slough HEC-RAS model was developed sequentially by Northwest Hydraulic Consultants (NHC) and the USACE. NHC developed the lower Guadalupe River reaches (the Bay to Highway 880) of the model in support of the District's Lower Guadalupe River Planning Study (Northwest Hydraulic Consultants 2002; Santa Clara Valley Water District 2001). The NHC HEC-RAS model was based on a previously existing 1997 Santa Clara Valley Water District HEC-2 model. The HEC-2 model was refined and brought into the HEC-RAS format. The steady-state model was then calibrated to March 1995 flow conditions (calibration parameters included the extent of weir overtopping downstream of the Union Pacific Railroad Bridge and assigning the channel roughness factors) which was estimated to be 311 cms (11,000 cfs) at the stream gage below Los Gatos Creek. To aid in operation and maintenance of the channel, the USACE extended the model upstream to Highway 280. The calibrated model was then verified using storms from January 1995, 263 cms (9,290 cfs), and January 1997, 155 cms (5,470 cfs) (Santa Clara Valley Water District 2001). According to the metadata provided with the HEC-RAS model, the last model modifications by the USACE occurred in October 2003.

The District's Guadalupe River HEC-RAS model was designed specifically for hydraulic studies upstream of the UPRR Bridge but includes the downstream reach of Alviso Slough. The model was based on prior hydraulic design boundary conditions, including:

(1) the 100-year peak flow at I-880 defined by the USACE is 481 cms (17,000 cfs),
(2) gravity and pumped inflows entering the LGR downstream of I-880,

(3) channel and tide conditions in the Baylands downstream of UPRR Bridge, including LGR project details for proposed sediment excavation and bench area vegetation clearing just downstream of UPRR,
(4) channel cross-sections in the lower (NHC) reaches are based on 1997 aerial survey data and 2003 conditions in the upper (USACE) reaches,
(5) geometry data in the model simulates maximum sediment excavation (assumed immediately following LGR construction) and minimum vegetation management height (assumed immediately following channel maintenance),
(6) some Manning's n values were atypical for roughness coefficients (0.80), and
(7) the existing 2002 Alviso Slough west levee profile as shown in the Engineer's Report (Santa Clara Valley Water District 2001).

The District's HEC-RAS model[3] was provided to PWA in 2005. PWA ran the hydraulic model input file using the most recent HEC-RAS version available. Our review suggested the following modifications: The Manning's n values along the channel were outside the range of typical roughness coefficients and were adjusted based on communication with USACE and the District. The value of n=0.80 was adjusted to n=0.08. The design peak flow rate for the model was increased from 481 cms to match the peak flow conditions established for the SBSP project 518 cms (18,325 cfs). This modified model was then used to establish the Baseline Conditions Model for future analysis of the Alviso Slough Fall 2006 conditions and the future SBSP project alternatives.

5.1.3 District's UNET Model (unsteady-state)

A UNET (Unsteady NETwork model) model was developed by NHC for hydraulic analyses of the Baylands planning area downstream from the UPRR Bridge. The Baylands Area UNET model was developed and used to support plan formulation, develop flood mitigation components, and provide hydraulic information for environmental documents being prepared for the LGRP and Baylands reaches, which includes a portion of the SBSP project area. The purposes and results from the District's UNET model are summarized in the Engineer's Report and EIR reports for LGRP (Jones & Stokes 2001; Santa Clara Valley Water District 2001). The model setup and sensitivity-test results were also reported in the Final Reconnaissance Report (Northwest Hydraulic Consultants 2002).

The UNET model, obtained from the District in 2005, used the cross-section and bridge geometry from the District's HEC-RAS model. This UNET model will be referenced in this document as the District's UNET model. The model utilized the inflow hydrograph representing the USACE 100-year flood hydrograph and the interior drainage facilities hydrograph (peak flowrate = 481 cms). PWA ran the District's UNET model to obtain the UNET output. PWA used UNET version 4.0 from April 2001. This report evaluates the current HEC-RAS analysis against the UNET output from the District's UNET model, not previous reporting of the sensitivity-tests by NHC. The data in the District's UNET model output were converted to metric units for comparison; UNET does not work in metric.

[3] The District's contacts were Al Gurevich and Christy Chung

5.1.4 Combined Approach

The initial UNET model was developed from available information in the Baylands downstream of the formal LGR project reach. The information available included: top-of levee profiles, underwater cross-sections and profiles, pond elevations, interior berm profiles, existing overflow (spill) dynamics into and out of the ponds, wind and wave effects, sediment dynamics, levee stability and other typical design level information. The UNET model was developed as a reconnaissance planning tool intended for application to the Baylands reach only. The UNET model was used to estimate water surface profiles along Alviso Slough, rates of Baylands levee overtopping, and depths of ponding in the Cargill Salt Ponds for various river inflow and tidal scenarios. The UNET channel, levee and pond geometries (topographies) were estimated from available information supplied by the District, the USACE, Cargill and limited surveys conducted in 2002 by CH2M Hill.

Our modeling approach was to combine the District's HEC-RAS and UNET models described above into one integrated model to compare the performance of the current flood management approach (combined conveyance and offline storage) with the potential future conditions (no-project alternative or salt pond restoration option with improved open channel conveyance) using the unsteady flow regime capabilities of HEC-RAS. Before we developed the HEC-RAS unsteady model to evaluate the baseline conditions we first developed the Existing Conditions Model to test the capability of the model in steady-state (the most widely used flow regime). The Existing Conditions Model (steady-state) used current topographic and roughness data to update the District's HEC-RAS model and was run using the steady flow regime. Figure 14 plots a comparison of the Alviso Slough east and west levees from the UNET model to the HEC-RAS top of levee data (estimated from the cross-section information) that was used in the Baseline Conditions Model.

5.2 Existing Conditions Model Setup (steady-state)

The Existing Conditions Model was adapted from the District's HEC-RAS model. The Existing Conditions Model sets the foundation for the Baseline Conditions Model (unsteady) by demonstrating consistency with the steady flow regime. The model required updating the channel geometry, diversion weir dimensions, and hydrology. Figure 15 presents the Alviso Slough plan map for the project reach. The model was processed using the steady-state flow regime and checked for errors. The Baseline Conditions Model used the peak flow rate of the prescribed hydrograph for the unsteady model, 518 cms (18,325 cfs). Figure 16 presents the project reach profile and provides a comparison of the District's UNET model, District's HEC-RAS model and the Existing Conditions Model incorporating the updates.

Geometric Data

Cross-Section Data

The District provided recent (February 2004) channel cross-section data for Alviso Slough, at 60 m intervals. The USACE[4] provided additional background metadata on June 20, 2005. The metadata

[4] Donald Twiss, USACE Sacramento District

provided specific survey reference data regarding the 2004 survey. These include: basis of horizontal control: Guadalupe River Folder #130, Coordinate file for survey request #2002-085; Horizontal Datum: CCS Zone 3, 1983 Metric; Basis of vertical control: BM291 & BM1172; and Vertical Datum: NAVD88 Metric. The new ground data was used to update the District's HEC-RAS model to produce the Baseline Conditions Model for the SBSP project. PWA used alternating cross-sections resulting in 120 m reach lengths between cross-sections. Figure 15 includes key cross-sections inform the hydraulic model. Output at these locations is used to compare the results of subsequent models.

Levee Data

The west bank levee separating Pond A8 from Alviso Slough has been overtopped during previous flood events. This levee overtopping was located roughly 150 meters downstream from the UPRR Bridge crossing and acted as an informal, uncontrolled "weir". It was recognized that this weir provides a flood management benefit, by allowing rising flood waters to exit the channel into Pond A8, thereby maintaining lower water levels in Alviso Slough and reducing flood risks to the community of Alviso and other areas to the east and south. This hydraulic connection to Pond A8 has been assessed during previous flood hazard reduction strategies for Alviso Slough (Jones & Stokes 2001; Northwest Hydraulic Consultants 2002; Santa Clara Valley Water District 2001; Schaaf & Wheeler 2004). A design to replace the informal low point with an engineered weir was included in the LGRP Study, Engineer's Report (Santa Clara Valley Water District 2001). The lateral weir was included in the District's HEC-RAS model at the location of the prior levee low point. The District's UNET model hydraulic connection from Alviso Slough to Pond A8 was an assumed multi-crested weir unlike the eroded levee dimensions. The District's models represented the design conditions proposed in the Engineer's Report. The Existing Conditions model was updated to reflect the as-built levee crest.

PWA set up the Existing Conditions Model by updating the lateral weir dimensions into Pond A8 using the post-construction survey drawings by the District, dated March 28, 2005. The weir is trapezoidal in shape with the base at 3.5 m (NAVD88) and approximately 300 meters long. The weir alignment along an outside bend in the channel provides a straightforward path for high water levels to exit the channel. The HEC-RAS lateral weir flow optimization tool was used to estimate the peak flow leaving Alviso Slough. The flows between the slough channel and Pond A8 are computed using the standard weir equation.

The Alviso Slough east bank levee separating Pond A8 from the community of Alviso has also recently been improved from the entrance to the marina upstream along the top of the levee to the UPRR Bridge. The most recent levee topography is used in the models prepared for this study. Figure 14 presents the top of levee profile for the east and west levees along Alviso Slough.

Model Parameters

Roughness

The roughness coefficients in the Baseline Conditions Model are revised from the District's HEC-RAS model. Channel roughness coefficients remained at 0.03 and overbank values were set at 0.08. In the area upstream of the lateral weir the Manning's n values represent the District's lower n-values that account for stream channel maintenance (0.04), i.e. vegetation removal. Manning's n values in the District's

UNET Model were typically set to 0.027-0.03 in the channel and 0.20 in the overbanks for Alviso Slough. These values were established by NHC, through calibration of the UNET model, to correspond to available high water data from the March 1995 flood event.

Boundary Conditions

The boundary conditions for the Existing Conditions Model represent existing conditions. The downstream boundary condition (water surface elevation in the Bay) for the Existing Conditions Model was set to the adopted 10-year tide (3.1 m NAVD88) from the San Francisco Bay Tidal Stage vs. Frequency Study (U.S. Army Corps of Engineers 1984a) for the study area.

The Existing Conditions model was run with an additional set of steady-state model boundary conditions: the 10-yr flow with the 100-year tide level. The District provided the estimate for the 10-year flow of 190 cms (6,700 cfs) taken from the 1977 USACE study (U.S. Army Corps of Engineers 1977) and the 100-yr tide level of 3.66 m NAVD88 (12 ft NAVD88) as determined by the USACE (U.S. Army Corps of Engineers 1984a).

5.3 Project Alternatives Modeling Setup (unsteady-state)

The HEC-RAS model was used to characterize existing conditions within Alviso Slough and to examine the effect of restoration alternatives on channel water levels. The changes in the longitudinal water surface profiles were compared for each alternative (short-term and long-term) throughout the length of Alviso Slough between the Bay and the UPRR Bridge. The upstream limit of the model is located just downstream of Highway 237.

The alternatives evaluated are:
Alternative A: No Action
Alternative B: Managed Pond Emphasis (50:50 ratio of Tidal Habitat to Managed Pond)
Alternative C: Tidal Habitat Emphasis (90:10 ratio of Tidal Habitat to Managed Pond)

The Alviso Slough hydraulic model was used to evaluate and compare the alternative scenarios listed in Table 9. In addition to Baseline Conditions and long-term No-Project Conditions (Alternative A), the model simulations included the project alternatives initially following levee modification (B-1 and C-1) and also for two long-term scenarios (B-2 and C-2; B-3 and C-3). The most conservative approach to the long-term simulations of alternatives (B-2 and C-2) is to assume that the channel geometry does not change from baseline conditions (no scour). Neglecting channel scour and reducing pond storage (due to marshplain development) results in less capacity than currently available under baseline conditions. The alternative “most likely” long-term scenarios (B-3 and C-3) incorporate projected channel geometric changes (expansions) due to scour provided by increased tidal prism. The No-Project Condition assumes levees on ponds A6 and A7 will fail due to lack of maintenance. Channel cross-sections downstream of these breaches show increased depths and widths due to the increase of tidal prism of ponds A6 and A7. Table 10 presents several model criteria or the assumptions used to establish the hydraulic models.

Table 9. Alviso Slough Hydraulic Simulations

Simulation	Model Designation	Timing	Comments
Baseline Conditions	BC	Fall 2006	Existing levees and channel geometry.
Alternative A No Action	A	Long-Term (50-year)	Includes projected relative sea-level rise, unplanned levee breaches, and channel scour (downstream of unplanned breaches).
Alternative B 50% Tidal / 50% Managed Ponds	B-1	Short-Term	Includes levee breaches and existing channel geometry (no scour).
Alternative C 90% Tidal / 10% Managed Ponds	C-1		
Alternative B 50% Managed Ponds / 50% Tidal	B-2	Long-Term- (50-year) No Channel Scour	Using existing channel geometry (no scour). Includes levee lowering, projected marshplain sedimentation and relative sea-level rise.
Alternative C 10% Managed Ponds / 90% Tidal	C-2		
Alternative B 50% Managed Ponds / 50% Tidal	B-3	Long-Term (50-year) Expected Channel Scour	Using long-term channel cross-sections based on hydraulic geometry calculations. Includes levee lowering, projected marshplain sedimentation and relative sea-level rise.
Alternative C 10% Managed Ponds / 90% Tidal	C-3		

Table 10. Hydraulic Model Assumptions

SBSP Restoration Alternative	HEC-RAS Model Assumptions						
	Boundary Conditions	**Initial Conditions**	**Ponds Modeled**	**Channel to Pond Weir Elevations**	**Pond Connection Elevations**	**Channel Scour**	**Manning's N Values**
Baseline Conditions	100-yr hydrograph upstream and 10-yr tide downstream	All ponds empty, except A5, A7, A9, & A10 (at ISP operation levels), initial upstream flow at 78.8 cms	A6, A7, A8, A8d, A9, A10, A11, and A12	Set at left and right levee elevations	Set at UNET elevations (1996 survey)	None	Channel = 0.03, Marsh & Overbank Areas = 0.11, Levee Sides and Tops = 0.03
Baseline Conditions Sensitivity Run	100-yr hydrograph upstream and 10-yr tide downstream and Pond A5/ Guadalupe Slough UNET hydrograph	All ponds empty, except A5, A7, A9, & A10 (at ISP operation levels), initial upstream flow at 78.8 cms	A6, A7, A8, A8d, A9, A10, A11, and A12	Set at left and right levee elevations	Set at UNET elevations (1996 survey)	None	Channel = 0.03, Marsh & Overbank Areas = 0.11, Levee Sides and Tops = 0.03
Alternative A	100-yr hydrograph upstream and 10-yr tide with SLR downstream	All ponds empty, except A5, A7, A9, & A10 (at ISP operation levels), initial upstream flow at 78.8 cms	A6, A7, A8, A8d, A9, A10, A11, and A12	Set at levee elevations with several breaches on A6 and A7	Set at UNET elevations (1996 survey)	Scour below A7 from unplanned breaches	Channel = 0.03, Marsh & Overbank Areas = 0.11, Levee Sides and Tops = 0.03
Alternative A-2 Sensitivity Run	100-yr hydrograph upstream and 10-yr tide with SLR downstream	All ponds empty, except A5, A7, A9, & A10 (at ISP operation levels), initial upstream flow at 78.8 cms	A6, A7, A8, A8d, A9, A10, A11, and A12	Set at levee elevations with several breaches on A6 and A7	Set at UNET elevations (1996 survey) Pond A8 West levee to 3.25 m	Scour below A7 from unplanned breaches	Channel = 0.03, Marsh & Overbank Areas = 0.11, Levee Sides and Tops = 0.03
Alternative B-1	100-yr hydrograph upstream and 10-yr tide downstream	All ponds empty, except A5, A7, A9, & A10 (at ISP operation levels), initial upstream flow at 78.8 cms	A6, A7, A8, A8d, A9, A10, A11, and A12	Set at levee elevations with several breaches on A6, A7, and A8	Set at UNET elevations (1996 survey)	None	Channel = 0.03, Marsh & Overbank Areas = 0.11, Levee Sides and Tops = 0.03

SBSP Restoration Alternative	HEC-RAS Model Assumptions						
	Boundary Conditions	Initial Conditions	Ponds Modeled	Channel to Pond Weir Elevations	Pond Connection Elevations	Channel Scour	Manning's N Values
Alternative C-1	100-yr hydrograph upstream and 10-yr tide downstream	All ponds empty, except A5, A7, A9, & A10 (at ISP operation levels), initial upstream flow at 78.8 cms	A6, A7, A8, A8d, A9, A10, A11, and A12	Set at levee elevations with several breaches on A6, A7, A8, A9, A10, A11, and A12	Set at UNET elevations (1996 survey)	None	Channel = 0.03, Marsh & Overbank Areas = 0.11, Levee Sides and Tops = 0.03
Alternative B-2	100-yr hydrograph upstream and 10-yr tide with SLR downstream	All ponds empty, except A9 & A10 (at ISP operation levels), initial upstream flow at 78.8 cms	A9, A10, A11, and A12	Set at right levee elevations	Set at UNET elevations (1996 survey)	None	Channel = 0.03, Marsh & Overbank Areas = 0.11, Levee Sides and Tops = 0.03
Alternative B-2 Sensitivity Run	100-yr hydrograph upstream and 10-yr tide with SLR downstream	All ponds empty, except A9 & A10 (at ISP operation levels), initial upstream flow at 78.8 cms	A8, A8d, A9, A10, A11, and A12	Set at right and left (A8 only) levee elevations	Set at UNET elevations (1996 survey)	None	Channel = 0.03, Marsh & Overbank Areas = 0.11, Levee Sides and Tops = 0.03
Alternative C-2	100-yr hydrograph upstream and 10-yr tide with SLR downstream	Initial upstream flow at 78.8 cms	None	None	None	None	Channel = 0.03, Marsh & Overbank Areas = 0.11, Levee Sides and Tops = 0.03
Alternative B-3	100-yr hydrograph upstream and 10-yr tide with SLR downstream	All ponds empty, except A9 & A10 (at ISP operation levels), initial upstream flow at 78.8 cms	A9, A10, A11, and A12	Set at right levee elevations	Set at UNET elevations (1996 survey)	~1 foot of scour at mouth	Channel = 0.03, Marsh & Overbank Areas = 0.11, Levee Sides and Tops = 0.03
Alternative B-3 Sensitivity Run	100-yr hydrograph upstream and 10-yr tide with SLR downstream	All ponds empty, except A9 & A10 (at ISP operation levels), initial upstream flow at 78.8 cms	A8, A8d, A9, A10, A11, and A12	Set at right and left (A8 only) levee elevations	Set at UNET elevations (1996 survey)	~1 foot of scour at mouth	Channel = 0.03, Marsh & Overbank Areas = 0.11, Levee Sides and Tops = 0.03

SBSP Restoration Alternative	HEC-RAS Model Assumptions						
	Boundary Conditions	**Initial Conditions**	**Ponds Modeled**	**Channel to Pond Weir Elevations**	**Pond Connection Elevations**	**Channel Scour**	**Manning's N Values**
Alternative C-3	100-yr hydrograph upstream and 10-yr tide with SLR downstream	Initial upstream flow at 78.8 cms	None	None	None	~3 feet of scour at mouth	Channel = 0.03, Marsh & Overbank Areas = 0.11, Levee Sides and Tops = 0.03
Alternative C-3 Sensitivity Run	10-yr peak flow upstream and 100-yr stillwater with SLR downstream (steady-state)	None	None	None	None	~3 feet of scour at mouth	Channel = 0.03, Marsh & Overbank Areas = 0.11, Levee Sides and Tops = 0.03

Geometric Data

Cross-Section Data
The channel geometry was updated in the Existing Conditions Model (steady-state) with the 2004 survey data. For some alternatives, cross-section geometry changes were made based on hydraulic modeling approach or expected geomorphic responses. These changes are listed for each alternative.

Levees
At present the levees in the Alviso Pond Complex range from engineered flood protection levees (for example, along Alviso Slough adjacent to the community of Alviso) to internal salt pond levees with minimal structural integrity. Under the various restoration options, levees may be removed, lowered, or rebuilt in alternate locations. The long-term No Action Alternative assumes the levees on ponds A5, A6, and A7 will not be maintained and unplanned levee breaches will occur. The location of the breaches was assumed to be at the location of historic slough channels. In each alternative considered, an alignment is proposed for a shoreline and fluvial channel levee system that will provide comparable or improved flood hazard management compared with baseline conditions. Our analysis assumes structural integrity of all affected levees (whether along Alviso Slough or interior to the former salt ponds) during the storm duration.

Model Parameters

Alviso Slough Geomorphic Evolution
The original construction of the salt ponds initiated geomorphic changes to Alviso Slough. The amount of water flowing in and out of the slough on a daily basis decreased drastically when the slough levees were constructed to dike off the salt ponds. The twice-daily tide prism in Alviso Slough was equivalent to about a 3-year runoff peak flow (70 cms to 85 cms twice a day). These daily flows represented the "channel forming" discharge. Tidal processes convey and redistribute sediments in the Baylands and lower reaches of the river, forming marsh surfaces and maintaining the low flow channel. Although diking the salt ponds reduced the tidal prism, the Bay continues to provide a source of sediment. As a result the lower reaches of Guadalupe River continue to fill-in with sediment.

Following either planned or unplanned levee breaches, pond areas will be open to the full tidal range available. Because of subsidence, the salt ponds will initially provide much greater tidal prism than the historic salt marshes. Therefore, the short-term tidal prism values represent the maximum potential tidal prism. This large tidal prism may initiate fairly rapid scour of Alviso Slough. Short-term scour effects were not incorporated into the hydraulic modeling scenarios.

All long-term tidal prism and channel volume calculations (for Alternatives A, B-3, and C-3) have been developed using hydraulic geometry relationships and represent estimated long-term equilibrium conditions. Long-term effects were estimated for the alternatives 50 years after levee breaches. For these scenarios, sediment will begin to fill the salt ponds, gradually recreating the salt marsh and reducing the tidal prism to historic (natural) levels. The channel will reach the cross-section size associated with the maximum tidal prism relative to the actual channel morphology; a function of the relative rates of channel scour and pond sedimentation. Long-term scenarios assume that pond areas will be filled with sediment to (approximately) the projected MHHW level. Marsh area is assumed to be the dominant force in controlling channel geometry for the long-term. Ultimately (over a period of decades), the marsh plain and channel morphology will reach a new equilibrium.

Tidal prism calculations for Alternative B assume that the west Alviso Slough and internal pond levees are breached at historic channel locations and that the contributing marsh area is estimated by the tributary area of the connecting higher ordered channels. For Alternative B this area represents about half of the pond area between Alviso Slough and Guadalupe Slough. Figure 17 shows the estimated tidal (habitat) area contributing to the tidal prism for Alternative B. Tidal prism calculations for Alternative C assume that the east and west Alviso Slough levees and the internal pond levees are breached at historic channel locations. Figure 18 shows the estimated tidal (habitat) area contributing to the tidal prism for Alternative C. For each alternative, the contribution to tidal prism from the Alviso Slough channel itself is relatively small and is assumed to be unchanged from baseline conditions.

Due to the expected expansion of the channel in response to the high potential tidal prism, there exists the potential for levee erosion along the channel. Levee erosion may affect the integrity of the levees providing ongoing flood protection. (The final project design will identify the levees required to maintain ongoing flood protection.) Appropriate measures will be developed to maintain any necessary downstream levee.

Hydraulic Geometry

Hydraulic geometry provides empirical relationships that describe slough channel dimensions based on scouring effects of tidal exchange. They are used to predict channel depth, width and cross-sectional area as a function of tidal prism or contributing marsh area (Williams et al. 2002). The relationships are based on measured parameters in equilibrium channels where geometry is determined by tidal processes. These relationships demonstrate a strong correlation between channel dimensions, tidal prism and marsh area. Hydraulic geometry relationships are a practical tool to assist in predicting channel geometry and equilibrium conditions for various tidal restoration options.

Within the SBSP, the purpose of the hydraulic geometry analysis is to estimate future channel geometry for the HEC-RAS modeling of Alviso Slough. For Alternative A (no-project alternative, 50 years in the future), hydraulic geometry was used to predict an increase in channel cross-section downstream from unplanned breaches to ponds A6 and A7. For each restoration alternative, channel volumes will increase in response to the increased tidal prism and subsequent channel scouring. The increase (or reduction) of channel capacity is used in the fluvial hydraulic analysis and the general impact analysis for each proposed alternative. Because of subsidence, the potential tidal prism in the salt ponds is much greater than it would be for natural tidal marshes in these locations. When levees between the ponds and Alviso Slough are breached, the available tidal prism is large and the channel is expected scour quite rapidly, deepening and widening the channel. Channel enlargement will aid in floodwater conveyance, allowing fluvial runoff to reach the bay more efficiently. The increased channel scour will reduce or halt the ongoing Alviso Slough channel siltation and may also provide additional capacity for internal drainage to be pumped into the channel. As siltation gradually raises the elevation of the salt ponds back to the natural marsh elevations, the rate of scour and channel response will slow, until a state of dynamic equilibrium is reached. Table 11 presents the results of the tidal prism calculations used for the long-term alternatives modeling.

Table 11. Marsh Area and Tidal Prism Values Used in Long-Term Alternatives Modeling

Model	RS 300		RS 1740		RS 3420		RS 4380		RS 5220		RS 6300	
	Marsh Area (km^2)	Tidal Prism (x1000 cu-m)	Marsh Area (km^2)	Tidal Prism (x1000 cu-m)	Marsh Area (km^2)	Tidal Prism (x1000 cu-m)	Marsh Area (km^2)	Tidal Prism (x1000 cu-m)	Marsh Area (km^2)	Tidal Prism (x1000 cu-m)	Marsh Area (km^2)	Tidal Prism (x1000 cu-m)
Alternative A	6.14	1,708	5.02	1,352	4.04	1,048	N/A	N/A	N/A	N/A	N/A	N/A
Alternative B-3	7.77	2,252	6.72	1,901	5.74	1,580	3.76	964	2.49	594	1.81	410
Alternative C-3	10.71	3,278	8.81	2,610	7.15	2,044	4.88	1,307	3.22	802	1.81	410

Note: Tidal Prism values derived from (Williams et al. 2002) – calculated in metric

For Alternative A, there are no unplanned breaches upstream of RS 3420, and thus no changes in hydraulic geometry from baseline conditions.

Hydraulic geometry relationships were calculated for Alviso Slough from the slough mouth at Coyote Creek to the Gold Street Bridge, within the community of Alviso. The hydraulic geometry assessment includes an evaluation of baseline conditions and the multiple alternatives immediately after project completion (short-term channel cross-section reflect “no scour”) and also for long-term equilibrium conditions. See Figure 19 for a typical slough channel cross-section for baseline conditions and for each restoration alternative.

Roughness Coefficient

In addition to channel topography and flow rates, the water surface in the slough is also affected by the channel “roughness” (a “rougher” channel reduces flow velocity and results in higher water levels, while a “smooth” channel allows faster flow and lower water levels). Channel roughness is characterized by Manning’s “n” value in the model. Marsh vegetation becomes established at approximately the elevation of mean high water (MHW). Below MHW the channel remains free of vegetation and lined with bay mud. Manning’s n values for these areas were established using the “Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains” (Arcement and Schneider 1989) to characterize the existing condition of the channel. We assumed a Manning’s “n” value of 0.03 for the un-vegetated channel reaches with smooth mud. For the floodplain we used the “modified channel method” from the USGS source document to estimate an n value that used a base value of “n” to represent the bare soil surface (.025), a factor for surface irregularities (.003), values for obstructions (.004) and vegetation on the flood plain (.075). The calculated Manning’s “n” value for the floodplain was 0.11 and assumes dense cattails as vegetation cover. Field reconnaissance of the project site was conducted by PWA staff to verify reasonable Manning’s “n” values.

Model documentation (Northwest Hydraulic Consultants 2002) suggests that roughness factors be increased if the model is to be used for events smaller than 240 cms (8,500 cfs), to reflect the relatively greater influence of channel vegetation during smaller flows. In an unsteady model flowrates vary for the duration of the event. Our analysis used constant roughness coefficients for all flows.

Boundary and Initial Conditions

Boundary Conditions

The unsteady flow model can accommodate variable boundary conditions occurring both upstream and downstream. Each of the unsteady-state hydraulic models provide time-varying estimates of the water surface elevation in Alviso Slough based on the changing downstream tide elevations (in the south San Francisco Bay) and the varying flood hydrograph entering the slough from the Guadalupe River. The estimated 10-year tide cycle elevation was used to set the downstream conditions while the base flood hydrograph (defined at Highway 237) was input at the upstream end of the reach. These hydrographs use different time-steps. The appropriate time step used for the HEC-RAS unsteady models was selected based on model convergence within the limited number of iterations. The resulting time-step for all unsteady scenarios is one minute.

The downstream water surface elevation for the unsteady flow models is controlled by tidal elevations in South San Francisco Bay. Figure 20 shows the design tidal time series used in the unsteady models. The unsteady models use the six day tide signal from March 2000 measured at the Dumbarton Bridge and extrapolated to the mouth of Alviso Slough. The tide signal is represented with an hourly time-step with the peak (2.9 m) occurring 32 hours into the simulation. In order for the unsteady analysis to match consistently with the starting boundary conditions of the steady state model, the entire tide signal was raised 0.2 m, creating the short-term downstream stage hydrograph with a 3.1 m (10.2 ft) peak condition to match the elevation of the District's 10-year tide at the mouth of Alviso Slough.

For future conditions, the tidal signal was modified to reflect expected sea-level rise. Sea level is estimated to increase 0.15 m (0.5 ft) in the next 50 years (IPCC 2001). To account for this expected rise in the downstream boundary condition, the tide signal for the short term modeling effort was increased to 3.25 m (10.7 ft) for the 50-year projection.

The design basis selected for the fluvial analysis is to assume that the 10-year tide coincides with the 100-year flood event. The Guadalupe River watershed 100-year flood hydrograph from the USACE (U.S. Army Corps of Engineers 1977) was used as the baseline for flood flows downstream of UPRR Bridge. The 1977 USACE hydrologic study resulted in a design flood hydrograph with an instantaneous peak flow of 481 cms (17,000 cfs). Immediately upstream of the UPRR Bridge the total contribution from fourteen interior drainage facilities (pumps and gravity outfalls) was calculated to be 37 cms (1,325 cfs). A cumulative hydrograph for all of the drainage facilities was developed and combined with the 100-year Guadalupe River flood hydrograph to produce a cumulative peak hydrograph scenario. The peak discharge resulting from this scenario is 518 cms (18,325 cfs) near UPRR Bridge at the community of Alviso (Northwest Hydraulic Consultants 2002). The cumulative peak hydrograph was applied to the SBSP project alternatives models directly downstream of Highway 237. The hydrograph simulates a four day runoff event at 15-minute time increments. Base flow of the hydrograph was set to 72 cms (2,540 cfs). Use of this peak flow rate was confirmed with the District. Figure 13 shows the design 100-year hydrograph for the study reach.

To provide conservative water surface estimates, the timing of the peak hydrograph was set coincident with the peak conditions of the 10-year tide. To establish the coincident peak in time, and spatially along the channel, the hydrograph was lagged until the peak of the 10-year tide reached the location of the lateral weir concurrent with the peak of the hydrograph reaching the same location.

Initial Pond Conditions

Consistency with the District's UNET model was maintained in setting up the Baseline Conditions Model. The network of ponds and hydraulic connections were replicated in HEC-RAS. Lateral weirs were used to duplicate the levees along Alviso Slough and provide a continuous connection to the adjacent ponds. The pond initial conditions (starting water levels), assumed storage volume (pool area), and pond connection dimensions and elevations were identified from the technical memorandums and output from the UNET model. These values were used initially in the set up of the Baseline Conditions Model.

Model setup required assumptions to complete the network of contributing ponds. The District's UNET model did not contain links to ponds A9 and A10 which have been added to the Baseline Conditions Model. The extent of the UNET model included the Baylands area contributions from Coyote Creek, Alviso Slough, Guadalupe Slough, as well as inputs from the Santa Clara County Treatment Facility.

The starting water surface elevation for each pond was based on 1998 Cargill surveys of the existing water surfaces (brine solution) in the salt ponds (Northwest Hydraulic Consultants 2002). Table 12 below provides the storage area, base pond elevation, initial conditions and the current operations for each pond as defined in the Initial Stewardship Plan (ISP) (Life Science 2003).

The District's UNET model network links Pond A12 to Pond A13, but no other link is made that would cause flows from a coincident event in Coyote Creek to impact the anticipated flood storage capacity in the ponds adjacent to Alviso Slough. Nevertheless, the connection from A12 to A13 was established in the Baseline Conditions Model. The results of the HEC-RAS analysis do not show any flows from Alviso Slough reaching Pond A13.

Table 12. Initial Pond Conditions

Pond	Area (x1000 sq-m)	Average Pond Base Elevation (m NAVD88)[2]	Initial Water Surface Elevation (m NAVD88)	Current Operations[1]
Pond A8 (wet)	1,679	-1.04	0.16	Seasonal Pond
Pond A8 (dry)	732	-0.15	0.88	Seasonal Pond
Pond A5	2,537	-0.58	0.96	Circulation Pond, Intake from Guadalupe Slough
Pond A6	1,388	0.91	0.91	Seasonal Pond (not in ISP, exist. FWS refuge)
Pond A7	1,052	-0.24	0.96	Circulation Pond, Outlet to Alviso Slough
Pond A9*	1,505	0.15	0.92	Circulation Pond, Intake from Alviso Slough
Pond A10*	1,024	-0.24	0.48	Circulation Pond
Pond A11	1,068	-0.55	1.28	Circulation Pond
Pond A12	1,259	-0.61	1.11	Batch Pond
Pond A13	1,093	-0.34	0.99	Batch Pond

** Ponds A9 and A10 were not included in the NHC UNET Model – Estimates from UNET Revision 6, July 5, 2001 Technical Memorandum(Santa Clara Valley Water District 2001)*
1 Source: Initial Stewardship Plan (ISP) (Life Science 2003)
2 Source: (Moffatt & Nichol Engineers 2005a; Siegel and Bachand 2002)

The pond areas relate linearly to the available storage. The pond areas used for this analysis are from the UNET modeling results (Santa Clara Valley Water District 2001) which are slightly less than pond areas reported by Moffatt & Nichols (Moffatt & Nichol Engineers 2005a; Siegel and Bachand 2002). NHC estimated the flood storage capacity for ponds A5, A7, and A8 (with no freeboard and without impacting Pond A6) above their initial starting water surface elevation at approximately, 14,300 cu-m [x 1000] (11,600 acre-feet) (Santa Clara Valley Water District 2001). Pond bottom elevations in the project alternatives model were set at the initial water surface elevations shown in Table 12.

The lateral spillways are used throughout the unsteady models to directly connect Alviso Slough to the adjacent ponds. Flows along the slough that exceed the top of levee height may spill into the adjacent pond. Conversely pond stages that exceed low elevations of the slough levees may cause flows to return to the slough channel. The lowest elevation along the lateral spillway connections used for the unsteady models is shown in Table 13.

Each pond that shares a common interior levee has been assigned a pond connection that allows flows to be exchanged between ponds. The pond connection data (dimensions, shape, and weir coefficients) are taken from the District's UNET Model. The weir coefficient for all the interior levees was 1.44 $m^{0.5}/s$ (2.6 $ft^{0.5}/s$). The weir coefficient for the engineered weir was increased to 1.7 $m^{0.5}/s$ (3.20 $ft^{0.5}/s$) to account for the improved shape and improved roughness of the weir, accounting for the vegetation clearing and assumed maintenance. In UNET, pond connection weir shapes represent the total distance along the weir at each given elevation in a tabular format (they are not precisely spatially accurate but reflect the expected hydraulic performance). HEC-RAS converts this table into a weir profile and assumes a symmetric stepped-shape. Weir flow calculations are the same for UNET and unsteady HEC-RAS. Table 13 provides detail for each pond connection used in the network.

Table 13. Lateral Spillways and Pond Connections

Pond Connection Designation	Pond	Minimum Elevation of Spillway Crest (m NAVD88)	Length of Spillway (m)
1	Alviso Sl to Pond A8 (wet)	3.50	300
2	Alviso Sl to Pond A8 (wet)	3.82	2,410
3	Alviso Sl to Pond A12	3.66	1,500
4	Alviso Sl to Pond A11	3.44	900
5	Alviso Sl to Pond A7	3.68	1,980
6	Alviso Sl to Pond A10	3.41	2,220
7	Alviso Sl to Pond A6	3.18	1,620
8	Alviso Sl to Pond A9	3.64	660
9	Pond A8w to Pond A8d	0.90	1,801
10	Pond A8w to Pond A5	0.99	649
11	Pond A8w to Pond A7	1.22	930
12	Pond A7 to Pond A5	1.60	1,829
13	Pond A7 to Pond A6	3.12	356
14	Pond A5 to Pond A6	3.05	496
15	Pond A12 to Pond A13	1.43	1,829
16	Pond A12 to Pond A11	1.83	610

Pond Connection Designation	Pond	Minimum Elevation of Spillway Crest (m NAVD88)	Length of Spillway (m)
17	Pond A11 to Pond A10	3.08*	970
18	Pond A10 to Pond A9	3.08*	1,400

** Values were assumed.*
Source: (Northwest Hydraulic Consultants 2002), if available

This modeling effort did not account for flow diffusion into New Chicago Marsh. The District's UNET model provides a connection between Alviso and New Chicago Marsh that is set at 0.91 m (3.0 ft). No connection between Pond A12 and New Chicago Marsh was identified in the UNET model. While our Baseline Conditions Model does not provide a network connection the resulting water surface in Pond A12 is compared with the minimum levee elevation (0.91 m) separating the two areas.

5.3.1 Baseline Conditions Model

The steady-state Existing Conditions Model was expanded to an unsteady model to represent baseline conditions. This allows modeling of time varying flow and inclusion of tidal variation in the Bay. The Baseline Conditions Model integrates the prior District fluvial modeling analysis into a single current unsteady flow network containing Alviso Slough and the contiguous ponds. PWA coordinated with the District to establish appropriate modeling methods and procedures to conduct this analysis (PWA 2005). The District provided guidance on model criteria and model boundary conditions.

Figure 16 compares the water surface profiles for the District's HEC-RAS and UNET models to the SBSP project Existing Conditions Model (steady-state) and the Baseline Conditions Model results. The unsteady model profiles represent the maximum channel water surface elevation along the reach during the event. Figure 21 shows the plan layout and flow chart for the hydraulic model.

Geometric Data

Cross-Section and Levee Data

The channel and levee geometry was updated in the Baseline Conditions Model. The top of the levees along Alviso Slough were defined as lateral weirs to allow flow transfer between ponds adjacent to the slough. No new updates were required to setup the Baseline Conditions Model. The linear coverage of the channel model included the reach from the mouth of Alviso Slough to just downstream of the Highway 237 Bridge crossing of Guadalupe River.

Pond Storage

The District's UNET model provided the base for setting up the geometric pond data and the hydraulic connections linking the network. Table 12 and Table 13 above present the data duplicated from the UNET study.

Model Parameters

Alviso Slough Geomorphic Evolution
Baseline Conditions Model channel geometry is from the 2004 survey data. No hydraulic geometry estimates were applied to the Baseline Conditions Model. This survey data is anticipated to be consistent with the EIS/EIR definition of baseline conditions for Fall 2006.

Roughness Coefficient
Roughness values for the Baseline Conditions model are described above in Section 5.3.

Boundary and Initial Conditions

Boundary Conditions
The boundary conditions for the Baseline Conditions Model were established upstream and downstream in the unsteady flow models. The downstream boundary condition is the 10-year design tide signal that has been adjusted temporally to peak at the estimated 10-year flood elevation. The upstream boundary condition is the cumulative hydrograph representing the effects of the base flood flow in the river and interior drainage being pumped into the river during the same event. These boundary conditions are similar for Alternatives B-1 and C-1 (short term).

Initial Pond Conditions
Baseline Conditions initial pond levels were set at the values listed in Table 12.

Model Sensitivity Testing

The Baseline Conditions Model was prepared to examine the effects of Guadalupe River-only contributions to the project site. This analysis did not account for flow entering the study area from Coyote Creek, Guadalupe Slough or the Santa Clara County Treatment Facility. The results of the UNET show that flow overtops the levee between Pond A5 and Guadalupe Slough. Flow enters Pond 5 during the peak flood event in Guadalupe Slough. As the water levels in Pond A5 increase from Alviso Slough fluvial flooding, water is discharged back into Guadalupe Slough. Flow into Pond A5 from Guadalupe Slough reduces the storage potential for flows from Alviso Slough during concurrent flood events, while flows out of Pond A5 to Guadalupe Slough will allow water to leave the Alviso system which may provide flood benefits. The UNET results show that the peak fluvial flow in the Guadalupe Slough precedes the timing of the peak water levels in pond A5.

A sensitivity test was performed on the Baseline Conditions Model to identify the effect that volume exchange to Guadalupe Slough would have on the offline storage capacity for Alviso Slough flood flows. The District's UNET model results show a total net volume of 1,900 cu-m [x1000] (1,543 acre-feet) overtopping the levee and flowing from Pond A5 to Guadalupe Slough levee during a concurrent event. The contributing inflow hydrograph from Pond A5 to Guadalupe River was extracted from the UNET model results and was added to Pond A5 as an additional boundary condition. Although the HEC-RAS sensitivity test resulted in a reduction in storage in Pond A5 and caused more flow to enter Pond A6 (resulting in the same final water level as the UNET simulation), there was not a significant change to the amount of flow diverted from Alviso Slough at the engineered weir. Therefore the Pond A5 initial water surface elevation of 0.96 m (ISP operations) was reset in our Baseline Conditions Model to study Guadalupe River-Only flood flows.

Table 14 below presents the results from the District's unsteady UNET model compared to the results from the Baseline Conditions and Baseline Conditions Sensitivity Run for each pond. The volumetric outcomes are consistent between the UNET model and the Baseline Conditions model. The change in storage volume and maximum waters surface elevation for Pond A6 are consistent between the UNET model and the Sensitivity Run due to the addition of the lateral inflow hydrograph. The Baseline Conditions results show that the WSEL in ponds A5, A7, and A8 achieve a uniform inundation-elevation at 3.41 m NAVD88

Table 14. Final Storage Volume and Water Surface Elevations

Pond	Change in Storage Volume (x 1000 cu-m)			Maximum Water Surface Elevation (m NAVD88)		
	UNET	Baseline Conditions	Baseline Conditions Sensitivity Run	UNET	Baseline Conditions	Baseline Conditions Sensitivity Run
Pond A8 (wet)	5,455	5,457	5,289	3.42	3.41	3.31
Pond A8 (dry)	1,877	1,852	1,779	3.42	3.41	3.31
Pond A5	6,370	6,216	5,962	3.38	3.41	3.31
Pond A6	827	2,304	833	1.51	2.57	1.51
Pond A7	2,674	2,577	2,474	3.40	3.41	3.31
Pond A9	0	0	0	-0.10	0.92	0.92
Pond A10	0	0	0	-0.26	0.48	0.48
Pond A11	0	0	0	1.28	1.28	1.28
Pond A12	0	365	365	1.10	1.40	1.40
Pond A13	4	0	0	1.00	0.99	0.99

Note:Change in Storage Volume is the difference between the volume at the maximum water surface elevation from the initial pond volume.
UNET Source: District's model dated June 6, 2002. Units converted to metric.
HEC-RAS Source: Baseline Conditions Model Results

5.3.2 Alternative A: No Project

Alternative A assumes that the California Department of Fish and Game (CDFG) and the U.S. Fish and Wildlife Service (USF&WS) will operate and maintain the ponds in a manner similar to the Initial Stewardship Plan (ISP), although ongoing operations and maintenance activities would be reduced. The ISP is intended as an interim plan for the period while the long-term restoration plan is developed and implemented. In the absence of a long-term restoration plan (i.e., the "No Action Plan"), the ISP would be replaced by a smaller set of prioritized operations and maintenance actions. The No Action Alternative assumes that the CDFG and USF&WS would not have funding to maintain full ISP operations over the 50-year planning horizon.

Initially under the No Action Alternative, pond pumping would be discontinued. Ponds that require pumping for water circulation would be dewatered or allowed to evaporate. These ponds would fill with rainwater and dry through evaporation. The landowners would manage water circulation in some or all of the remaining ponds using gravity-flow control structures, with the extent of management depending on the funds available. Over time, water management would be discontinued on a pond-by-pond basis as hydraulic structures break.

The Alternative A model represents the preliminary alternative definition that assumes the CDFG and USF&WS would not maintain the pond levees surrounding ponds A5, A6, and A7. With continued levee subsidence and sea-level rise, the levees would be prone to failure. Unplanned breaches may occur and will not be repaired. This definition of Alternative A has been revised from its preliminary alternative definition to assume long-term levee failure at ponds A5, A6 and A7. The Alternative A model assumes the levees are allowed to erode, and tidal action will be restored to some ponds through uncontrolled breaching. Flood risks and damages would increase over time due to deteriorating levee conditions and future sea-level rise.

Geometric Data

Cross-Section and Levee Data

Alternative A presents a 50-year projection of the slough conditions assuming the "no-project alternative" was implemented. The west bank levee that protects ponds A6 and A7 from Alviso Slough floodwaters are anticipated to fail because of unplanned breaches during the 50-year projection. It is anticipated that the unplanned breaches are most likely to occur at locations of historic slough channels and along the interior pond levees. These breaches will allow tidal water to fill and drain ponds A5, A6, and A7 and contribute to channel scour downstream of the most upstream breach (estimated at station 3420). The scour occurring downstream of the breaches will increase channel capacity in this vicinity. Estimates of the extent of channel scour were based on hydraulic geometry methods. Upstream of RS 3420, channel cross sections are the same as Baseline Conditions.

Pond Storage

The Alternative A model maintains the same network of ponds and connections as setup in the Baseline Conditions Model. Breaches into ponds A6 and A7 will convey tidal waters into in ponds A5, A6, A7, A8 (wet), and A8 (dry) and reduce the storage capacity. The volume of flood flow passing over the engineered weir to Pond A8 is expected to decrease due to the loss of storage in the former salt ponds. The future limiting factor of the engineered weir to maintain existing levels of flood hazard reduction is the ability of the weir to counter the water surface profile rise and reduces water levels upstream.

Model Parameters

Alviso Slough Geomorphic Evolution
The geomorphic changes associated with Alternative A are a result of scour associated with unplanned breaches. This results in wider and deeper cross-sections downstream of the most upstream breach.

Roughness Coefficient
Alternative A channel and overbank roughness assumptions are consistent with the Baseline Conditions Model. Cross-sections modified to represent the expected hydraulic geometry changes maintain the same Manning's n values for channel and marshplain defined in the Baseline Conditions Model.

Boundary and Initial Conditions

Boundary Conditions
As in all future conditions models, the downstream boundary condition has been adjusted to represent sea-level rise as estimated by the Intergovernmental Panel on Climate Change (IPCC 2001). The model maintains the same four-day stage-hydrograph frequency. To account for the estimated sea-level rise, the design tide signal was increased by 0.15 m (0.5 ft).

Initial Pond Conditions
For Alternative A, initial pond elevations are the same as Baseline Conditions.

5.3.3 Alternative B: 50% Tidal

Alternative B emphasizes approximately 50:50 mix of tidal habitat and managed pond. The outboard levee along the marsh corridor will require maintenance until marsh vegetation develops. New or improved flood management levees would be located along the landward edge of the project site or, in a few locations, possibly bayward of the managed ponds. Much of the public access and recreation would be integrated with flood protection and managed pond levees.

Alternative B has been evaluated in this analysis using the following assumptions. The ponds east of Alviso Slough remain isolated from the slough by the existing slough levee. Flood storage in these facilities is only accessible for flows that overtop the existing levee. In the model, the west Alviso Slough levee is removed to the elevation of MHHW and the conveyance corridor is widened to the east levees of Guadalupe Slough. Given that the downstream mouth widens toward the Bay, the effects of coincident peak flows in Coyote Creek are diminished. The analysis does not incorporate the flood flow contribution from the Guadalupe Slough or Coyote Creek. We assume flows from these two systems are contained within their levees.

Geometric Data

Cross-Section and Levee Data

The assumption of removing the west Alviso Slough levees increases the cross-sectional shape of the channel and results in a loss of off-line storage in ponds A5, A6, A7, and A8. The storage loss is compensated by the increased conveyance area in the west bank. The Alternative B model assumes that sedimentation gradually fills the ponds to the marshplain elevation in the long-term. Opening ponds A5, A6, A7, and A8 to tidal exchange increases the tidal prism and the channel geometry. The Alternative B-3 channel dimensions reflect scoured conditions occurring within the slough channel as a result of restoring salt pond to tidal action and increasing the net marsh area. The channel is scoured downstream RS 5820. Upstream of this location, the cross sections in Alternative B-3 are the same as Baseline Conditions.

An additional model (Alternative B-2) was run assuming that the channel does not scour as a result of the increase in the tidal prism. This model is the long-term (50-year) no channel scour scenario. The model uses the existing channel geometry for the low flow channel, includes sedimentation of the marshplain on the west side of the channel, and relative sea-level rise.

Pond Storage

The removal of the west slough levee allows the channel flows to spread across the ponds A5, A6, A7, and A8. The ponds east of Alviso will be maintained as managed ponds. These ponds would provide offline storage benefits for flows that top the east Alviso Slough levee. The model output does not show that the east slough levees are overtopped.

Model Parameters

Alviso Slough Geomorphic Evolution

The long-term 50% Tidal scenario (Alternative B-3) assumes that the marsh plain in the vicinity of ponds A5, A6, A7, and A8 has aggraded to the future elevation of MHHW. The channel downstream of RS 5820 has come into equilibrium with the increased marsh plain area adjacent to the channel. This is reflected in the cross-sections with increased channel depths and widths based on hydraulic geometry relationships. Upstream of RS 5820, hydraulic geometry relationships predict depths shallower than those of Baseline Conditions. It was assumed that the channel is currently near an equilibrium state and that increasing the marsh area will not cause channel aggradation. Cross sections above this location were not altered from Baseline Conditions.

Roughness Coefficient

Alternative B channel and overbank roughness assumptions are consistent with the Baseline Conditions Model. Cross-sections modified to represent the expected hydraulic geometry changes maintain the same Manning's n values for channel and marshplain defined in the Baseline Conditions Model.

Boundary and Initial Conditions

Boundary Conditions
The downstream boundary condition for Alternative B-1 is consistent with Baseline Conditions. Alternatives B-2 and B-3 long-term conditions model use the 10-year tide signal adjusted for the estimated sea-level rise as the downstream boundary condition.

Initial Pond Conditions
For all 50% Tidal Alternatives, initial pond conditions are the same as Baseline Conditions.

Model Sensitivity Testing

For Alternative B, Pond A8 will be managed so that the breach or breaches connecting the pond to tidal action can be closed and the pond can be drained in order to lower water levels for the winter flood season. It is expected that increasing the pond's tidal prism will scour the channel downstream. As the downstream channel capacity increases and the potential for flooding is lessened over time, Pond A8 will be operated with more tidal prism resulting in higher pond water levels. This adaptive management approach will be studied in detail in the SBSP Phase I hydrodynamic analysis for Pond A8. Eventually, Alviso Slough will be scoured enough to allow the pond to be fully tidal.

Two sensitivity runs were performed on Alternatives B-2 and B-3 to model these interim conditions. The geometry for these scenarios included Pond A8, the engineered weir, and the lateral connection representing the levee between Alviso Slough and Pond A8. For these sensitivity runs, the water level in Pond A8 was set at the future elevation of MHHW to evaluate fully tidal conditions. The interior pond levee that separates ponds A5 and A7 from Pond A8 is raised to the 10-year tide elevation to prevent Bay flooding in Pond A8. At this elevation, Guadalupe River flood flows can enter Pond A8 and exit the pond to the west, into the restored marshplain of Ponds A5 and A7, as opposed to flowing north over the slough levee back into Alviso Slough. The results of these sensitivity runs show that Alternative B-2 Sensitivity Run (no channel scour, with Pond A8) water levels are greater than those of Alternative B-3 (channel scour, without Pond A8). This suggests that it will be possible to operate Pond A8 in such a way that will maintain flood protection throughout the adaptive management period.

5.3.4 Alternative C: 90% Tidal

Alternative C emphasizes tidal restoration, relative to the other alternatives, and provides an approximately 90:10 ratio of tidal habitat to managed pond. Alternative C would create the most extensive tidal marsh corridor of the three alternatives, allowing for a continuous corridor along the entire project area shoreline. New or improved flood protection levees would be located along the landward edge of the project site, on the inboard side of tidal restoration areas. Compared to Alternative B, there would be fewer opportunities for offline storage of flood flows, but increased flow conveyance resulting from levee removal/lowering and increased channel scour/expansion.

Geometric Data

Cross-Section and Levee Data

Both Alviso Slough channel levees are removed in the Alternative C model. The width of the overall channel corridor widens along the east bank to the western edge of the ponds immediately adjacent to Alviso Slough: ponds A12, A11, A10, and a portion of A9. The west bank maintains the dimensions introduced in Alternative B. Breaching the ponds will allow full tidal exchange. These ponds will fill with sediment and encourage growth of the marsh plain at approximately the future elevation of MHHW. The channel geometry proposed for the Alternative C channel condition was estimated empirically from hydraulic geometry relations. The expected increase in the marsh plain area will enhance the tidal prism and scour the slough channel downstream of RS 5820. Upstream of this location, cross sections for Alternative C-3 are the same as Baseline Conditions.

An additional model (Alternative C-2) was run assuming that the channel does not scour as a result of the increase in the tidal prism. This model is the long-term (50-year) “no channel scour” scenario. The model uses the existing channel geometry for the low flow channel, includes sedimentation of the marshplain on both sides of the channel, and relative sea-level rise.

Model Parameters

Alviso Slough Geomorphic Evolution

The long-term 90% Tidal scenario (Alternative C-3) assumes that the marsh plain adjacent to both sides of Alviso Slough has aggraded to the future elevation of MHHW. The channel downstream of RS 5820 has come into equilibrium with the increased marsh plain area next to the channel. This is reflected in the cross-sections with increased channel depths and widths based on hydraulic geometry relationships. Upstream of RS 5820, hydraulic geometry relationships predict depths shallower than those of Baseline Conditions. It was assumed that the channel is currently near an equilibrium state and that increasing the marsh area will not cause channel aggradation. Cross sections above this location were not altered from Baseline Conditions.

Roughness Coefficient

Alternative C channel and overbank roughness assumptions are consistent with the Baseline Conditions Model. Cross-sections modified to represent the expected hydraulic geometry changes maintain the same Manning’s n values for channel and marshplain defined in the Baseline Conditions Model.

Boundary and Initial Conditions

Boundary Conditions

The Alternative C boundary conditions are the same as the Alternative B boundary conditions. Sea level is expected to rise and is accounted for in the long-term conditions model. The upstream peak hydrograph boundary conditions remain consistent with all of the unsteady models.

Initial Pond Conditions
Initial pond levels for Alternative C-1 are consistent with the Baseline Conditions Model. No ponds are modeled in the Alternatives C-2 and C-3 models.

Model Sensitivity Testing

A steady-state sensitivity run was performed using the geometry for Alternative C-3 to estimate the water surface profile resulting from the 100-year Bay water level plus sea level rise (3.66 m + 0.15 m = 3.81 m) with a coincident 10-year fluvial discharge, Q10 = 189.9 cms (6,700 cfs). The boundary conditions were provided by the District. A second steady-state scenario examined the 10-year Bay water level (3.1 m) with the peak 100-year fluvial flow (518 cms). The boundary conditions for this scenario match the unsteady-state peak tide signal and peak hydrograph flow rate. These profiles are plotted in Figure 25. The "effective" flood profile is the highest water surface elevation of the two scenarios. The figure shows that the 100-year tide boundary with the 10-year fluvial component controls up to approximately River Station 6250. At that location, the water surface resulting from the 10-year tide elevation coupled with the 100-year fluvial flood controls.

The effective Alternative C-3 water surface profile is compared to the effective Existing Conditions water surface profile to estimate the flood hazard reduction. The figure shows that the effective Existing Conditions water surface profile exceeds the future effective profile at River Station 4900, suggesting that the with-project Alternative improves future flood protection during a 100-year tide event.

5.4 Hydraulic Model Results

The hydraulic models of the Alviso Slough system were compared to evaluate water levels within the channel and assess flow through the adjacent pond network for the No Action Alternative and the restoration alternatives. Figure 22 presents the water surface profile results of the Baseline Conditions Model (unsteady-state) compared to the short-term project Alternative B and Alternative C. Figure 23 presents the water surface profile results of the Baseline Conditions Model compared to the long-term alternative models including the without project, Alternative A. The downstream boundary condition for the future condition models has been increased by 0.15 m (0.5 ft) to account for sea-level rise. For the project restoration alternatives, the effects of pond management and breaching were tested and found to reduce water surface elevations and reduce fluvial flood hazards. The No Action alternative was modeled using unplanned breaches into ponds A5 and A6. The results of Alternative A modeling show flood water levels remain unchanged due to the combination of increasing water levels from SLR and improved conveyance resulting from unplanned breaches. Fluvial flood impacts of breaching are beneficial in the long-term due to increased channel conveyance (via channel scour) and increased conveyance over the restored marshplain.

Potential short-term fluvial flood impacts to the existing slough levees could occur as the channel begins to scour and as a result will need to be managed to protect the integrity of the slough levees.

Results of the analysis show that the engineered weir into Pond A8 currently provides an important flood management function for flood flows in Guadalupe River / Alviso Slough. Diverted flows drain to Pond A8 until the available storage capacity is reached. Pond connections, defined by the elevation and length of the interior levees, are used to route flows between ponds. The ponds west of Alviso Slough are connected with a network of pond connections that provide offline storage for flood flows entering at the Pond A8 engineered weir.

NHC Technical Memorandum, dated June 02, 2002, describes the net volume from Scenario 5 is 17,640 cu-m [x1000] (14,300 acre-feet) spilling into the ponds resulting in 1.0 m inundating the Refuge (Pond A6). In the UNET model, flood waters entering ponds A5, A7, and A8 spill into Pond A6 with some flow draining back over the levees into Alviso and Guadalupe Sloughs near the peak of the design event (Northwest Hydraulic Consultants 2002). Schaaf and Wheeler reported the total storage anticipated for ponds A5, A6, A7, and A8 was 16,282 cu-m [x1000] (13,200 acre-feet) for the 581 cms (18,350 cfs) flood hydrograph (Schaaf & Wheeler 2004). Table 15 provides a comparison of the SBSP restoration model results to these previous studies.

Table 15. Alviso Slough to Pond A8 – Peak Discharge and Net Spill Volume

Model / Study	Scenario	Flow Regime	Weir Peak Flowrate (cms)	Net Volume (x 1000 cu-m)
District's HEC-RAS	(Past)	Steady	273	N/A
District's UNET	(Past)	Unsteady	240	17,640
Schaaf & Wheeler Study			273	N/A
Existing Conditions	Current	Steady		16,282
Baseline Conditions	Fall 2006	Unsteady	234	16,403
Alternative A	Long-Term	Unsteady	236	16,615
Alternative B-1	Short-Term	Unsteady	149	8,753
Alternative B-2	Long-Term	Unsteady	-	-
Alternative B-2 Sensitivity Run	Long-Term	Unsteady	242	17,584
Alternative B-3	Long-Term	Unsteady	-	-
Alternative B-3 Sensitivity Run	Long-Term	Unsteady	200	11,214
Alternative C-1	Short-Term	Unsteady	-	-
Alternative C-2	Long-Term	Unsteady	-	-
Alternative C-3	Long-Term	Unsteady	-	-

Note: Alternatives B and C will convert Pond A8 to tidal habitat. The lateral weir will be removed.The Alternative B-2 and B-3 Sensitivity Runs include Pond A8 and the lateral weir.

The results of the SBSP hydraulic modeling of Alviso Slough restoration alternatives show a slight reduction in the maximum water surface elevation. A reduction in the water surface profile through the project site will propagate as reduced water levels to the upstream bridge crossings at the community of Alviso (RS 7007 and RS 7121.5). There is a net decrease in the water surface profile at these sensitive structures. Table 16 compares the resulting water surface elevation for each model at six other locations along the channel. The cross-section at beginning of the lateral weir is at RS 6780. Cross-section 6060 is just downstream of the former marina. Three more cross-sections (RS 1740, RS 3900, & RS 4380) are within the project reach and one cross-section is located at the mouth of Alviso Slough (RS 300). Figure 15 shows the location of these cross-sections within the project reach.

Table 16. Alviso Slough / Guadalupe River Water Surface Elevation Checks[1]

Model	Description	Section 1	Section 2	Section 3	Section 4	Section 5	Pond A8 Weir	UPRR	Gold Street
		RS 300	RS 1740	RS 3900	RS 4380	RS 5460	RS 6780	RS 7007	RS 7121.5
Existing Conditions		3.10	3.21	3.54	3.63	3.89	4.47	4.56	4.64
Baseline Conditions	Fall 2006	3.10	3.19	3.49	3.58	3.82	4.24	4.45	4.49
Alternative A		3.25	3.27	3.47	3.57	3.82	4.25	4.46	4.49
Alternative A-2[2] Sensitivity Run	Long-Term No Channel Scour	3.25	3.26	3.34	3.44	3.75	4.24	4.45	4.48
Alternative B-1	Short-Term	3.10	3.13	3.56	3.57	3.59	4.10	4.33	4.38
Alternative B-2	Long-Term No Channel Scour	3.25	3.34	3.68	3.76	3.98	4.35	4.42	4.46
Alternative B-2 Sensitivity Run	Long-Term No Channel Scour	3.25	3.28	3.60	3.65	3.84	4.26	4.33	4.38
Alternative B-3	Long-Term Channel Scour	3.25	3.30	3.49	3.56	3.77	4.17	4.26	4.31
Alternative B-3 Sensitivity Run	Long-Term Channel Scour	3.25	3.28	3.36	3.40	3.54	4.17	4.25	4.30
Alternative C-1	Short-Term	3.10	3.05	3.31	3.31	3.32	4.08	4.34	4.38
Alternative C-2	Long-Term No Channel Scour	3.25	3.28	3.64	3.72	3.89	4.19	4.35	4.39
Alternative C-3	Long-Term Channel Scour	3.25	3.27	3.36	3.40	3.53	3.96	4.11	4.18

1- Elevations are in meters NAVD88

2- Sensitivity Run to raise Pond A8 west levee

A comparison between the updated 2004 cross-sections with the former 1997 cross-sections shows that ongoing sedimentation has reduced the net flow area between the existing levees, reducing the capacity of Alviso Slough. The decrease in existing channel capacity resulted in slightly higher simulated floodwater elevations when comparing the UNET model results to the Baseline Conditions Model results.

The downstream boundary conditions for the 10-year tide used for this analysis assume that flood flows occurring in Coyote Creek will not control the WSEL at the mouth of Alviso Slough. The District's UNET model results were evaluated and the maximum water surface elevation computed in Coyote Creek at the mouth of Alviso Slough is 2.84 m NAVD. Thus, the downstream boundary conditions for the short term (3.1 m) and long-term (3.25 m) simulations have a higher downstream WSEL than the UNET model.

Figure 24 compares the Baseline Conditions Model water surface profile with the water surface profiles resulting from the project alternatives. The profile shows only the reach of Alviso Slough from the low elevation along the east slough levee up through the Gold Street Bridge. The top of the levees is compared to the computed water surface elevation to verify the available freeboard for the levee protecting the community of Alviso.

5.5 Alviso Slough Modeling Conclusions

This technical report presents the results of the Guadalupe River / Alviso Slough hydraulic analyses that evaluate the effects of the design flood on:

- the baseline channel conditions,
- the future channel, assuming no SBSP Restoration Project,
- the project alternative with 50% of the salt ponds tidal and 50% managed ponds, and
- the project alternative with 90% of the salt ponds tidal and 10% managed ponds.

The hydraulic model analysis was conducted using the USACE HEC-RAS computer program. The model was setup using two different flow regime models (one steady-state, one unsteady-state) provided by the District. New Alviso Slough ground survey information was used to improve the geometric data. The resulting model incorporated the analysis to date for the slough and pond network. Boundary conditions were established that delineate the downstream high tide and the upstream design flood hydrograph.

The baseline conditions and the final alternatives analysis was conducted using the unsteady flow regime capabilities of the HEC-RAS computer program. Baseline Conditions Model results are comparable to the previous study results, although the current analysis characterizes a more accurate representation of the current flood hazard conditions. Modeling of the SBSP project alternatives included a combination of offline storage and improved channel conveyance that resulted from scouring of the slough channel and widening the floodplain corridor over the marsh plain.

Table 17 presents the results of a comparison between the water surface elevation results for each model to the water surface elevation of the Baseline Conditions Model. The long-term alternatives show an increase in the downstream water levels due to sea level rise and a reduction in the slope of the hydraulic grade line. This results in an improved water surface elevation at the Gold Street Bridge for the (long-term) restoration alternatives.

Table 17. Change in Water Surface Elevations from Baseline Conditions Model

Model	Section 1	Section 2	Section 3	Section 4	Section 5	Pond A8 Weir	UPRR	Gold Street
	RS 300	**RS 1740**	**RS 3900**	**RS 4380**	**RS 5460**	**RS 6780**	**RS 7007**	**RS 7121.5**
Baseline Conditions	**3.10**	**3.19**	**3.49**	**3.58**	**3.82**	**4.24**	**4.45**	**4.49**
Alternative A	0.15	0.08	-0.02	-0.01	0.00	0.01	0.01	0.00
Alternative B-1	0.00	-0.06	0.07	-0.01	-0.23	-0.14	-0.12	-0.11
Alternative B-2	0.15	0.15	0.19	0.18	0.16	0.11	-0.03	-0.03
Alternative B-3	0.15	0.11	0.00	-0.02	-0.05	-0.07	-0.19	-0.18
Alternative C-1	0.00	-0.14	-0.18	-0.27	-0.50	-0.16	-0.11	-0.11
Alternative C-2	0.15	0.09	0.15	0.14	0.07	-0.05	-0.10	-0.10
Alternative C-3	0.15	0.08	-0.13	-0.18	-0.29	-0.28	-0.34	-0.31

Notes: Elevations are in meters NAVD88; Increases at Section 1 are due to sea-level rise

The conclusion of this analysis shows that the no-project alternative will maintain the effectiveness of Alviso Slough to convey the design flood flows from the Guadalupe River watershed due to unplanned breaches. Each of the final program-level most-likely restoration alternatives (B-3 and C-3) show an improvement to the conveyance through the project area with lower predicted upstream flood water elevations.

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Native files for this report are located at PWA's offices:
P:\Projects\1750_South_Bay_Salt_Ponds\Task04_Flood_Management\Reports\Flood_Rpt\Revised_Final

7. LIST OF PREPARERS

The following PWA members assisted in preparation of this document:

Vince Geronimo – Task Manager
Jeffrey Haltiner – Task Director
Bob Battalio
David Katzev
Matt Wickland
Julie Stephenson

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8. FIGURES

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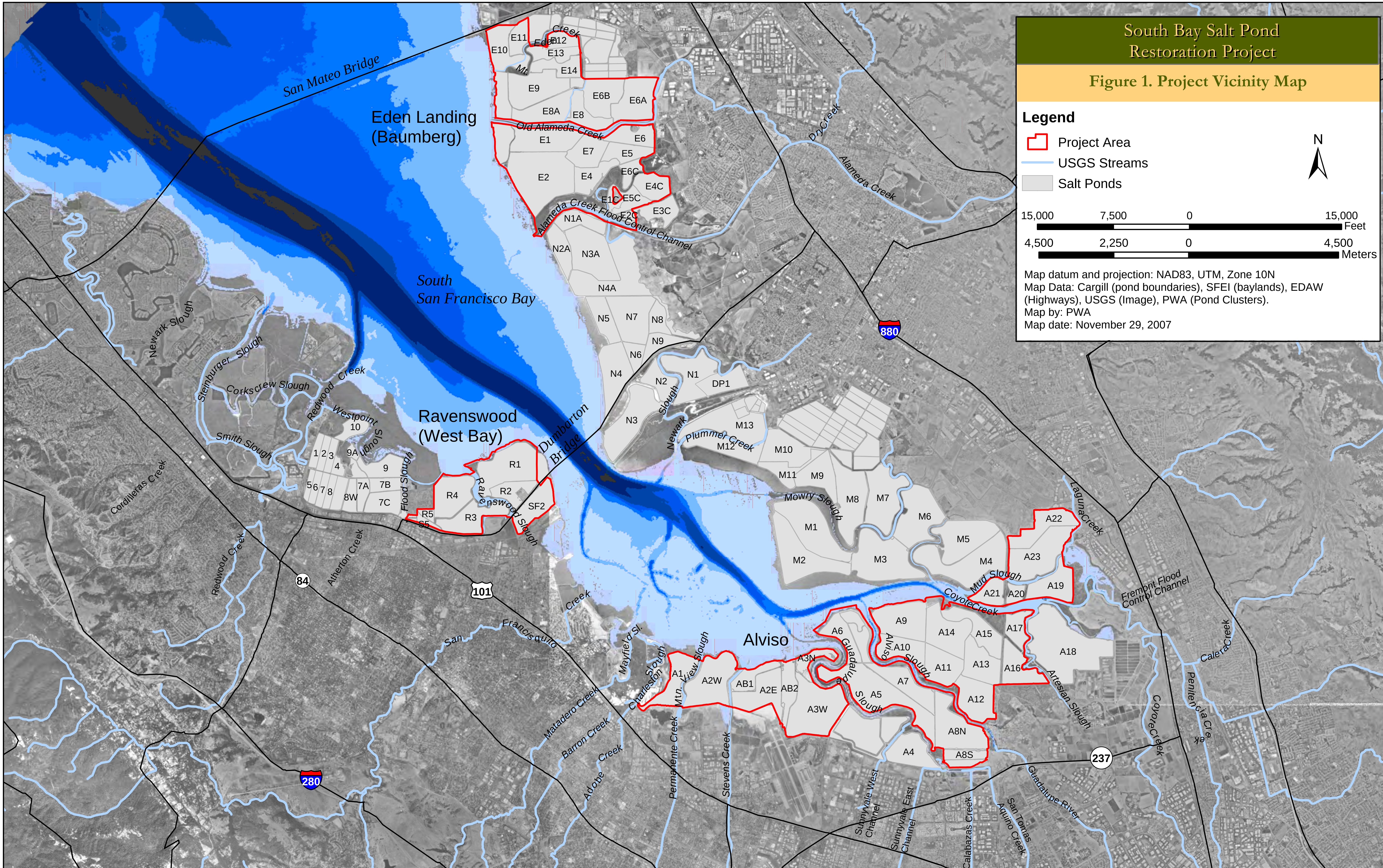
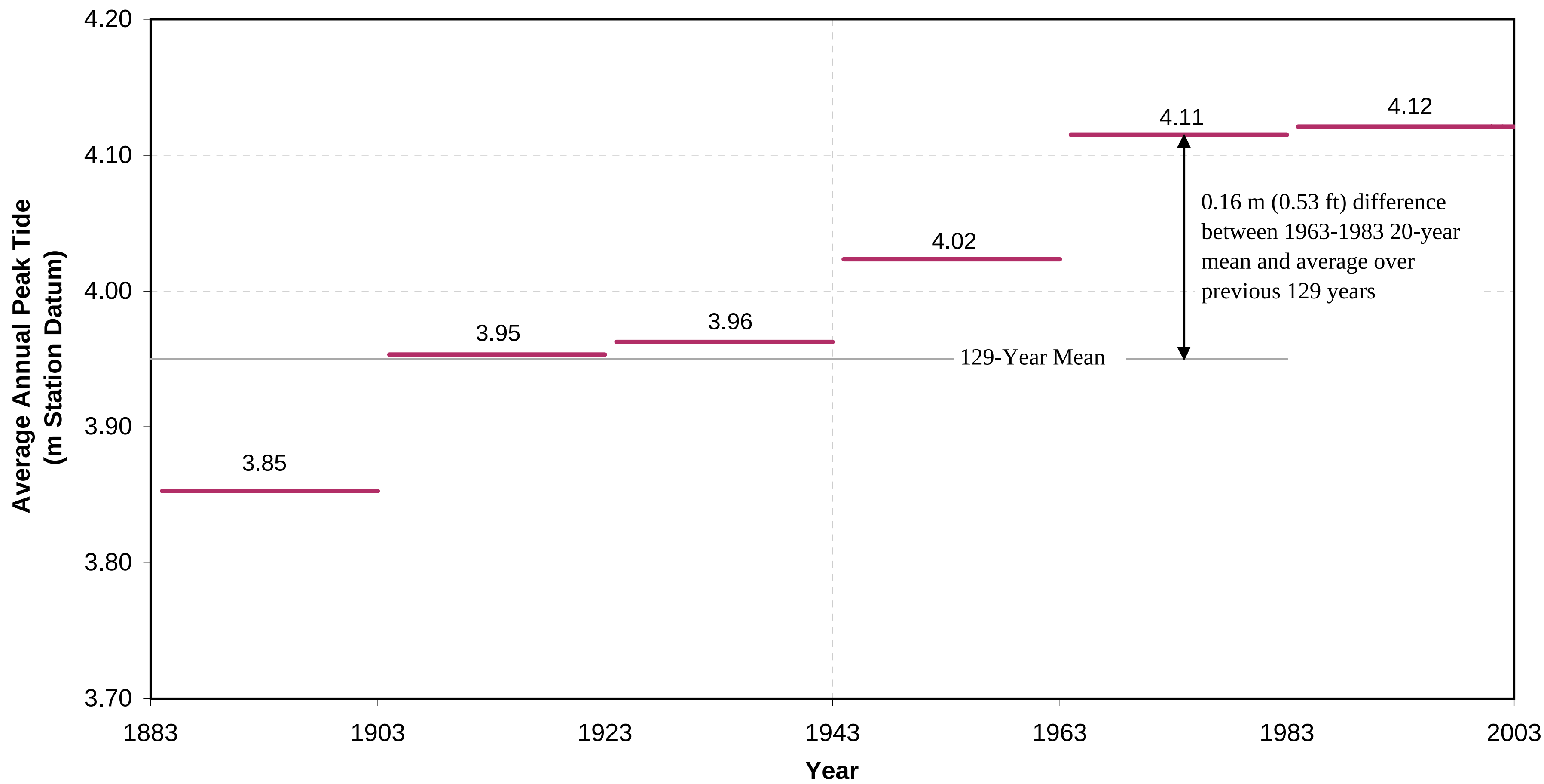

South Bay Salt Pond Restoration Project
Figure 1. Project Vicinity Map
Legend
Project Area
USGS Streams
Salt Ponds
N
15,000 7,500 0 15,000 Feet
4,500 2,250 0 4,500 Meters
Map datum and projection: NAD83, UTM, Zone 10N
Map Data: Cargill (pond boundaries), SFEI (baylands), EDAW (Highways), USGS (Image), PWA (Pond Clusters).
Map by: PWA
Map date: November 29, 2007
Eden Landing (Baumberg)
Ravenswood (West Bay)
Alviso
South San Francisco Bay
San Mateo Bridge
Dumbarton Bridge
Old Alameda Creek
Alameda Creek Flood Control Channel
Newark Slough
Plummer Creek
Mowry Slough
Mud Slough
Coyote Creek
Alviso Slough
Guadalupe Slough
Artesian Slough
Ravenswood Slough
Flood Slough
Westpoint Slough
Redwood Creek
Corkscrew Slough
Steinburger Slough
Smith Slough
Cordilleras Creek
Atherton Creek
San Francisquito Creek
Matadero Creek
Barron Creek
Adobe Creek
Permanente Creek
Stevens Creek
Charleston Slough
Mtn. View Slough
Sunnyvale West Channel
Sunnyvale East Channel
Calabazas Creek
San Tomas Aquino Creek
Guadalupe River
Laguna Creek
Fremont Flood Control Channel
Calera Creek
Penitencia Creek
Coyote Creek
Alameda Creek
Dry Creek
880
280
84
101
237



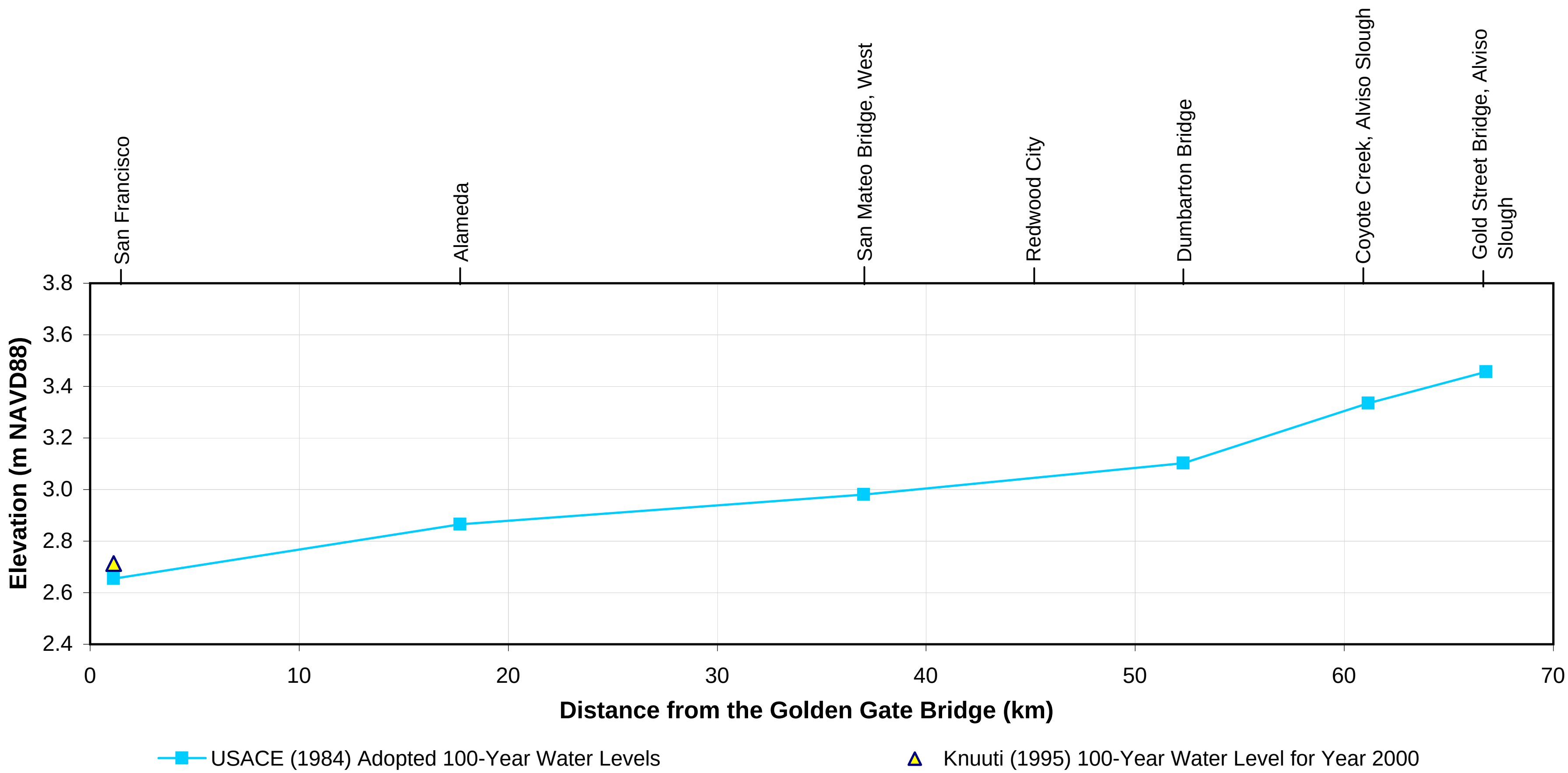
Source: NOAA NOS http://www.tidesandcurrents.noaa.gov/

f igure 2

South Bay Salt Pond Restoration Project

20-Year Means of Mean Annual Maximum Water Level at San Francisco

PWA



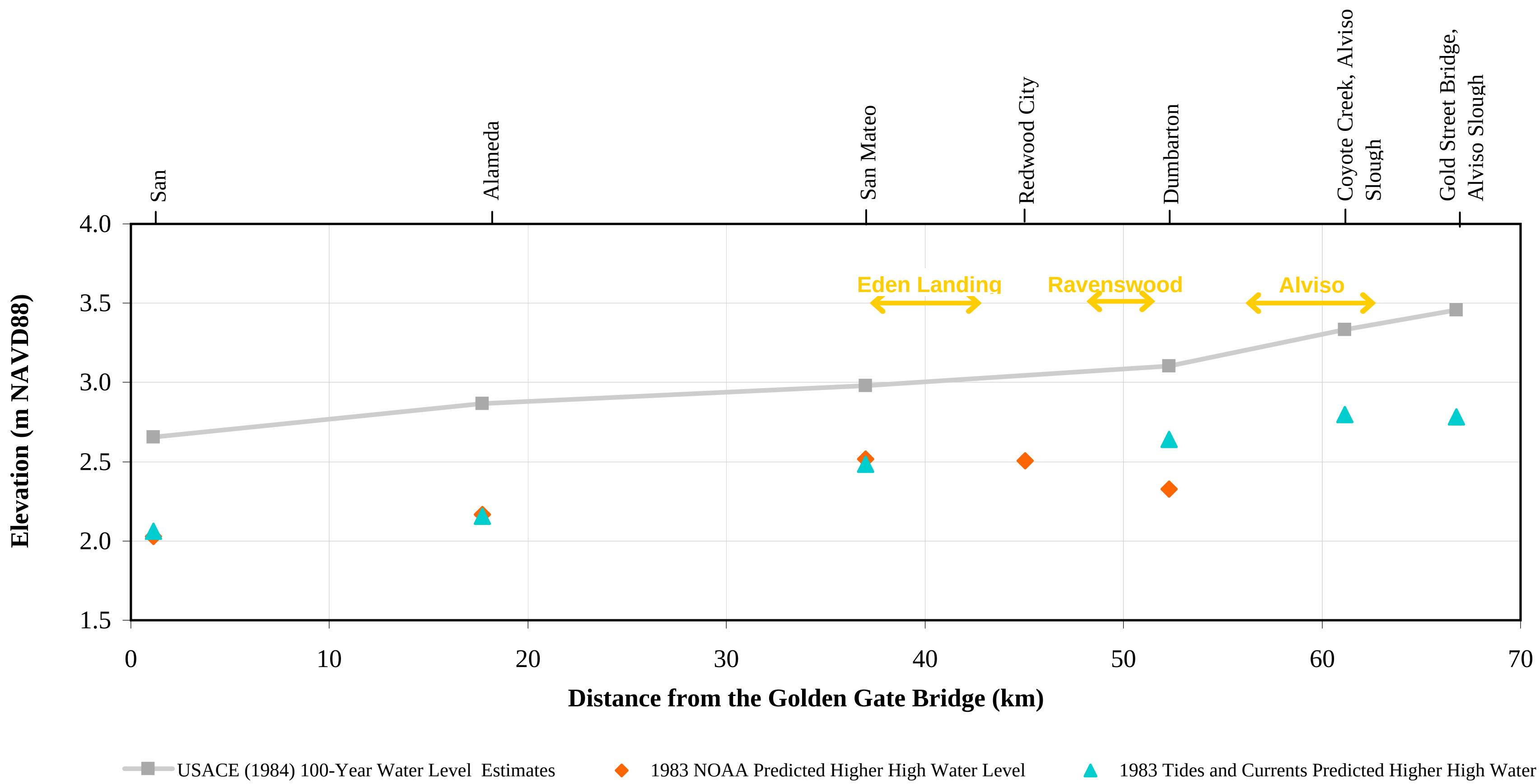
Notes:
USACE (1984): Adopted 100-year water levels are higher than those computed from a tidal stage vs. frequency analysis.

figure 3

South Bay Salt Pond Restoration Project

100-Year Water Levels from USACE (1984) and Knuuti (1995)

PWA



Source: USACE 1984, NOAA NOS CO-OPs (http://www.tidesandcurrents.noaa.gov/), Tides and Currents Pro 2.5

figure 4

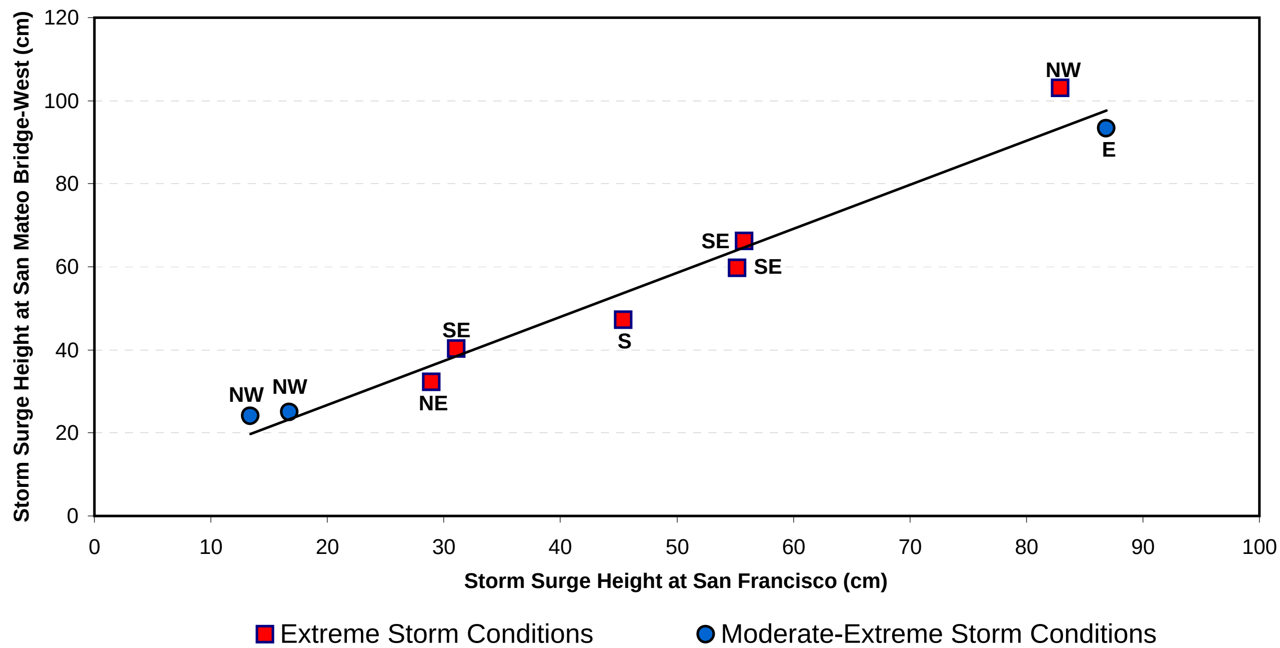
South Bay Salt Pond Restoration Project

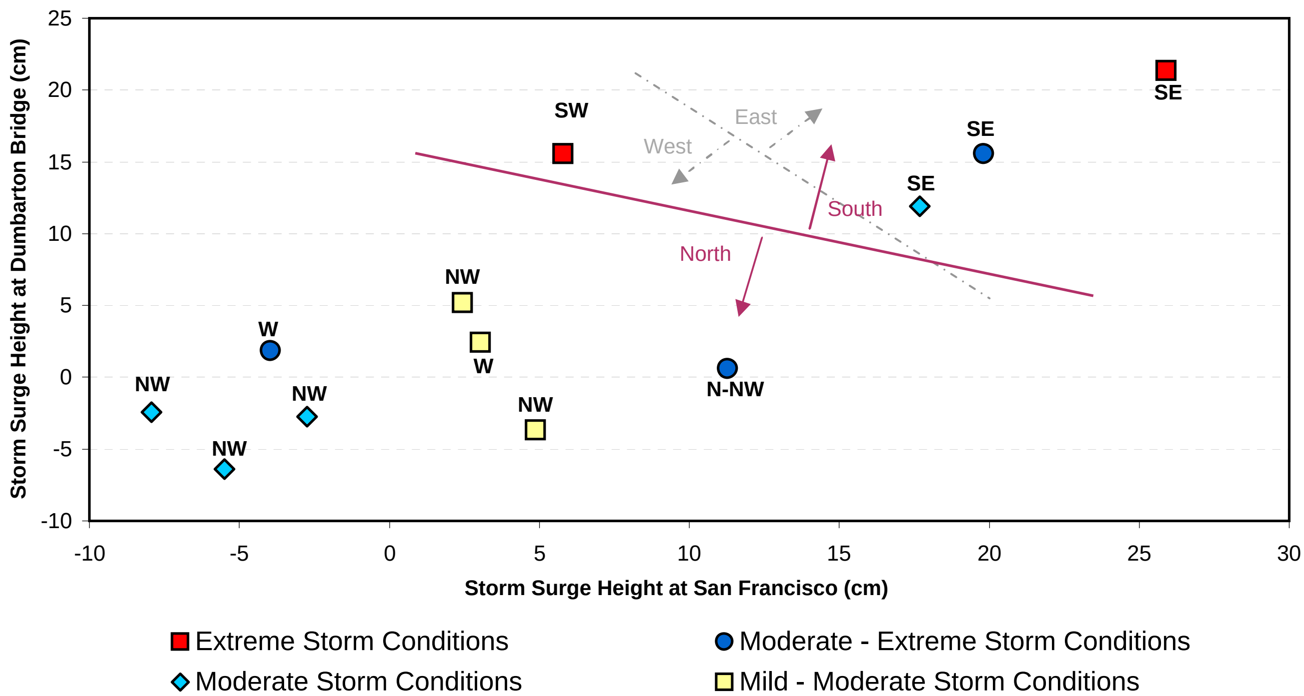
Comparison of Predicted and Measured Water Levels in South San Francisco Bay During a Storm Surge (1983)

PWA

\\Orca\pwa\Projects\1750_South_Bay_Salt_Ponds\Task04_Flood_Management\Reports\Flood_Rpt\Final\Figures

July 2006
1751.04





Notes: Data labels represent the predominant wind direction accompanying each storm/storm surge.

N = north	NW = northwest
S = south	NE = northeast
E = east	SW = southwest
W = west	SE = southeast

figure 5

South Bay Salt Ponds Restoration Project

Comparison of Storm Surge Heights at San Francisco, San Mateo Bridge and Dumbarton Bridge

PWA

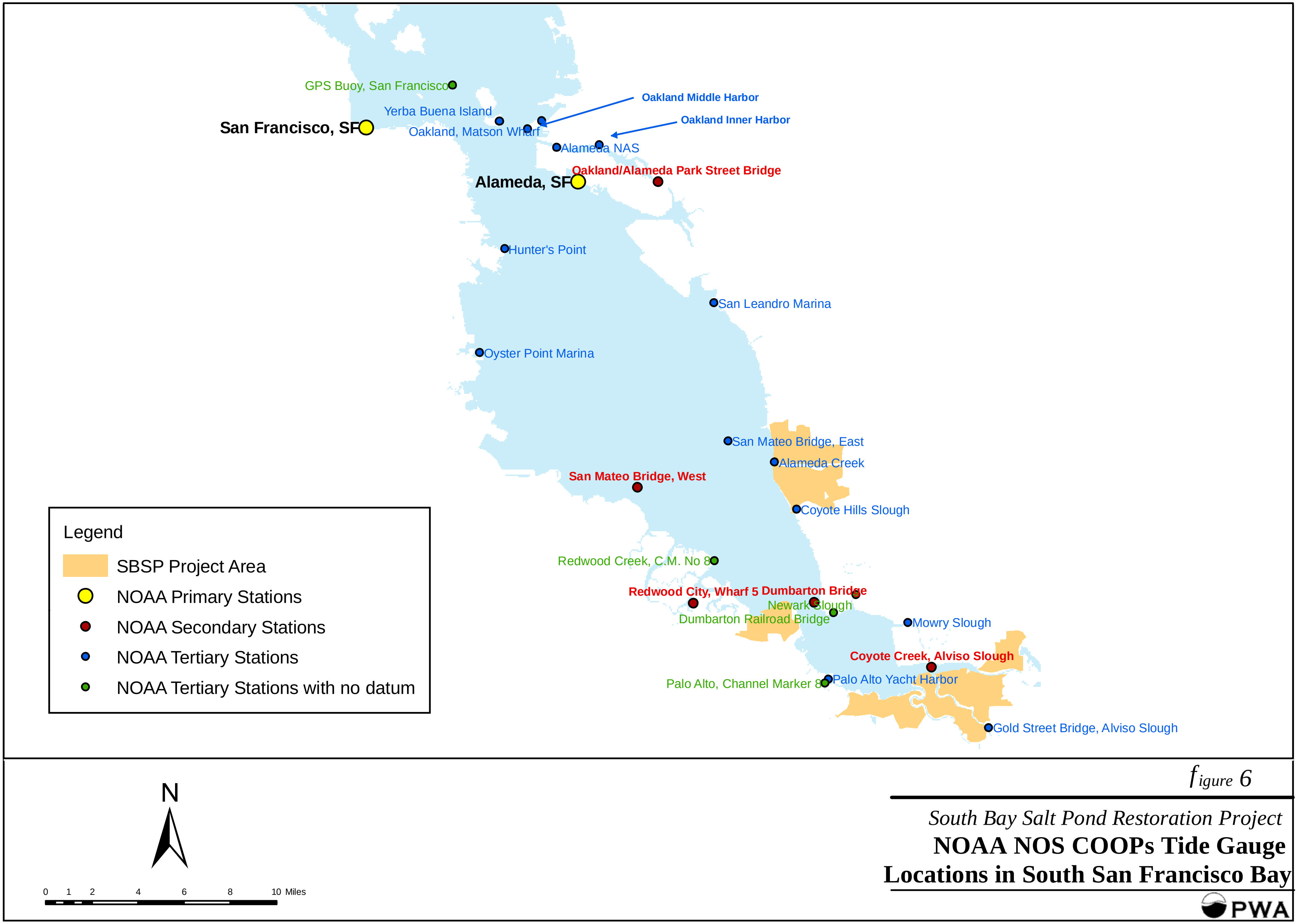
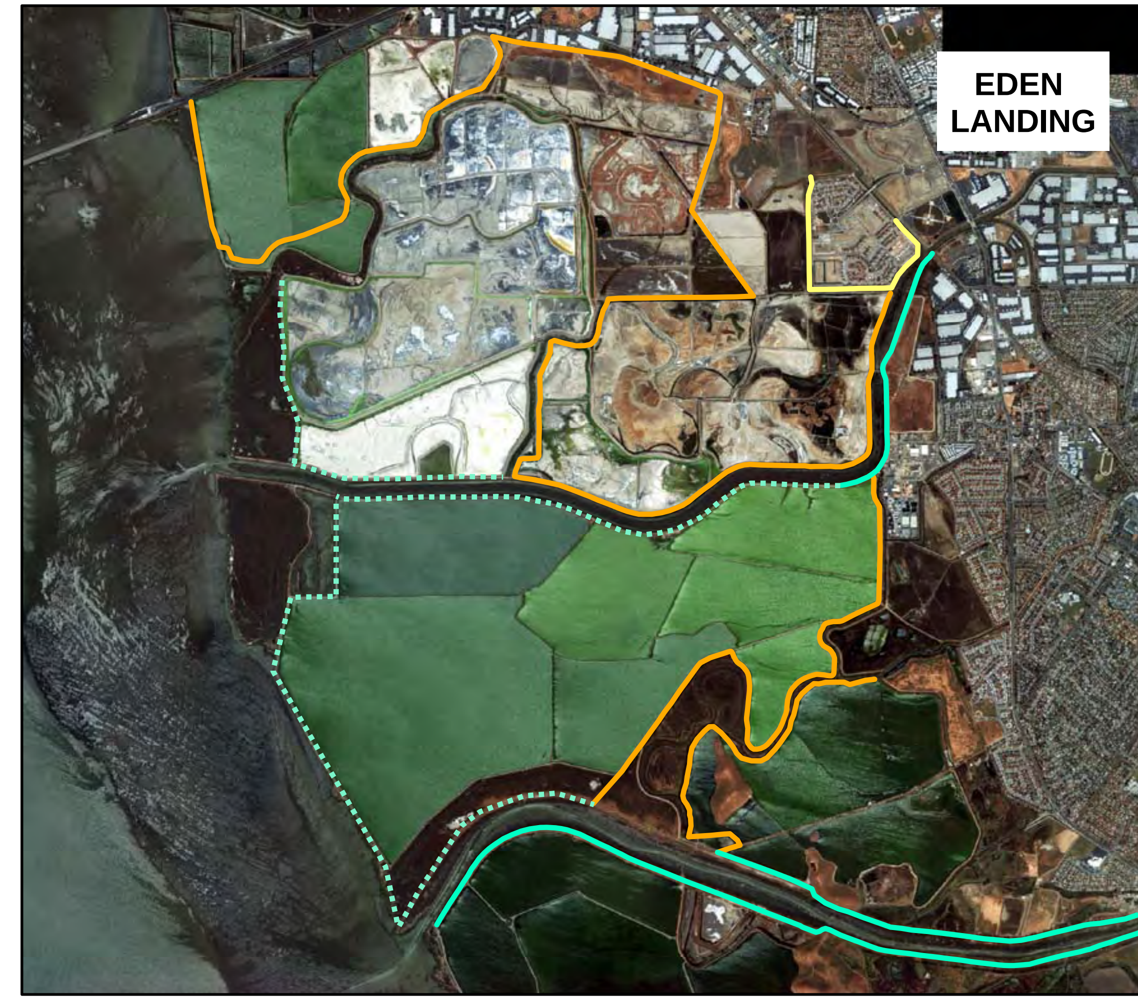


figure 6

South Bay Salt Pond Restoration Project

NOAA NOS COOPs Tide Gauge Locations in South San Francisco Bay

PWA



Legend

- **Existing High Ground***
- **Existing Levee* (includes engineered flood protection levees and non-engineered levees)**
- **Existing Pond Levee to be Maintained/Improved by Year 50***
- **Existing Levee to be abandoned by Year 50**

*Level of flood protection not specified

Note: Levees along creeks extend upstream of the endpoints shown.
All levee and high ground locations are approximate.

N

figure 7

South Bay Salt Pond Restoration Project

Levee Alignment for Alternative A

PWA

G:\1720SBSP\CoastalFlooding\Revised Levee Figures\Figure 7 Levees Alt A.mxd

December 2007
1751.04

Legend

- Fluvial Flood Protection Levee
- Proposed Flood Protection Levee with outboard managed pond
- Proposed Flood Protection Levee with outboard tidal marsh
- Existing Levee with outboard managed pond*
- Existing Levee with outboard tidal marsh*
- Existing Levee with no outboard tidal marsh or managed pond*
- Existing High Ground with outboard managed pond (may require improvement for flood protection)
- Existing High Ground with outboard tidal marsh (may require improvement for flood protection)

*Level of flood protection not specified

Note: Levees along creeks extend upstream of the endpoints shown.
All levee and high ground locations are approximate.

N

figure 8

South Bay Salt Pond Restoration Project

Levee Alignment for Alternative B

PWA

G:\1720SBSP\CoastalFlooding\Revised Levee Figures\Figure 8 Levees Alt B.mxd

December 2007

1751.04

Legend

- Existing High Ground with outboard tidal marsh (may require improvement for flood protection)
- Existing High Ground with outboard managed pond (may require improvement for flood protection)
- Existing Levee with no outboard tidal marsh or managed pond*
- Existing Levee with outboard tidal marsh*
- Existing Levee with outboard managed pond*
- Proposed Flood Protection Levee with outboard tidal marsh
- Proposed Flood Protection Levee with outboard managed pond
- Fluvial Flood Protection Levee

*Level of flood protection not specified

Note: Levees along creeks extend upstream of the endpoints shown.
All levee and high ground locations are approximate.

figure 9

N

South Bay Salt Pond Restoration Project

Levee Alignment for Alternative C

PWA

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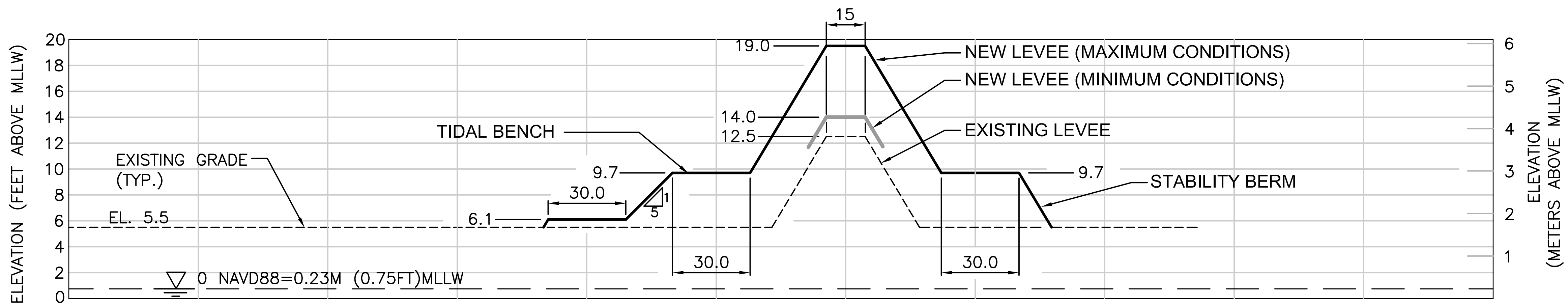
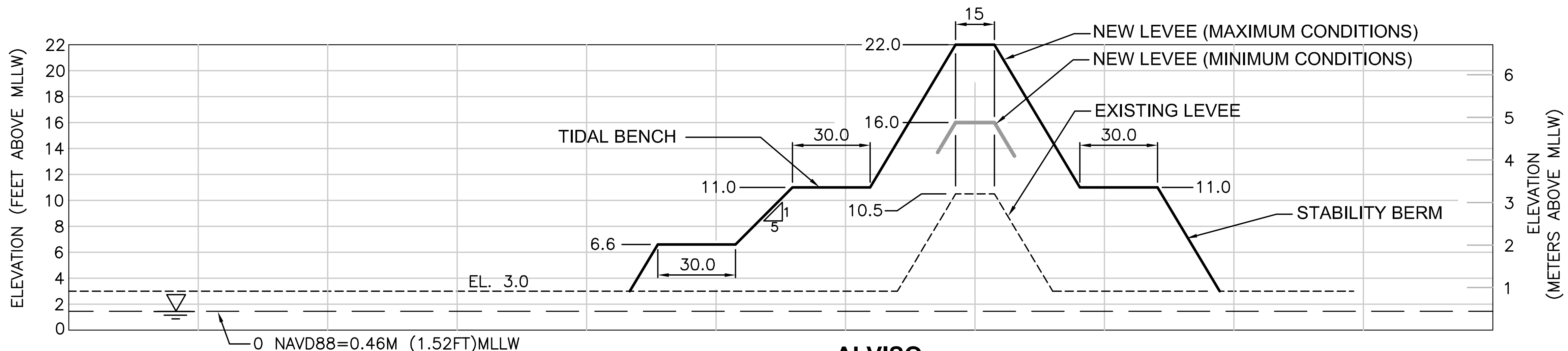
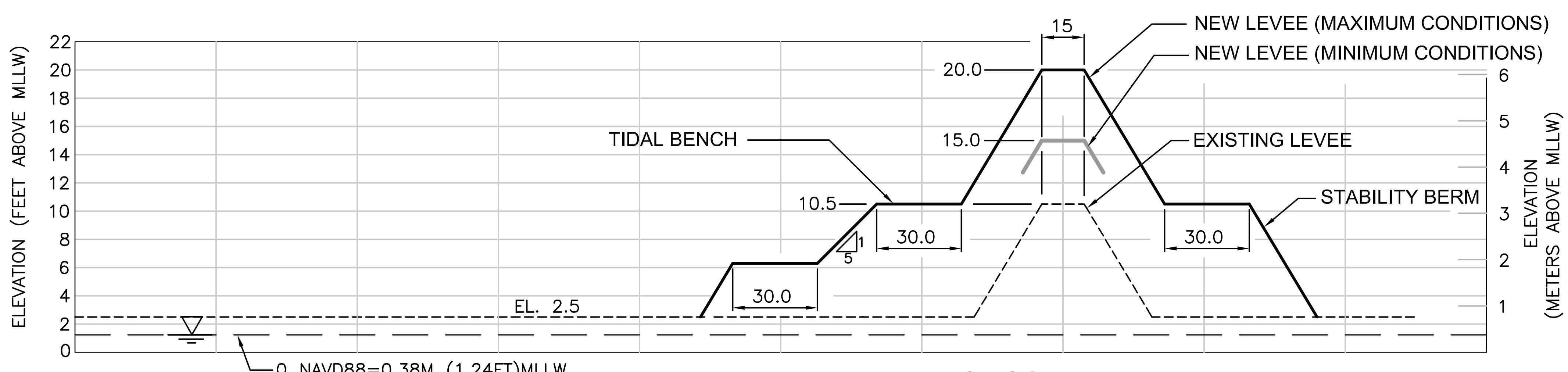
December 2007
1751.04



EDEN LANDING



ALVISO



RAVENSWOOD

NOTE: ALL SLOPES ARE 3:1 U.O.N.
ALL ELEVATIONS ARE IN FEET MLLW, U.O.N.

PWA *figure 10*

CONCEPTUAL LEVEE CROSS-SECTIONS
NO OUTBOARD MARSH

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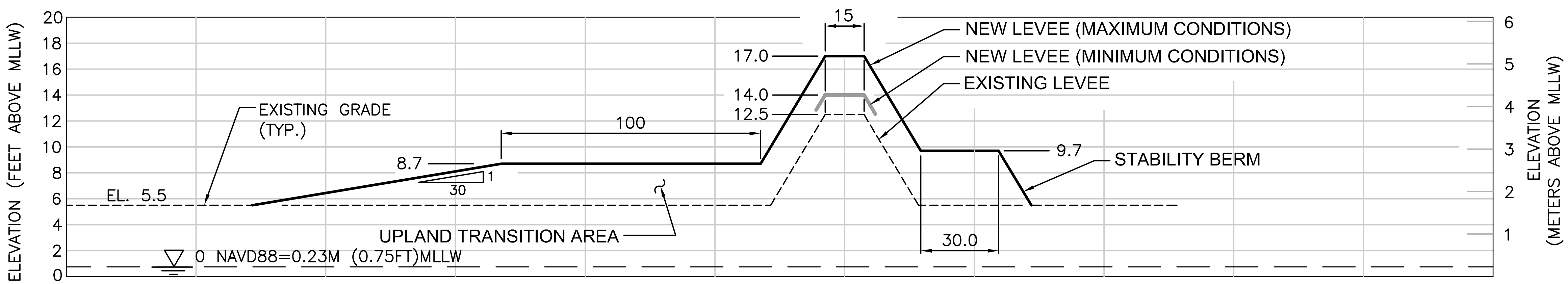

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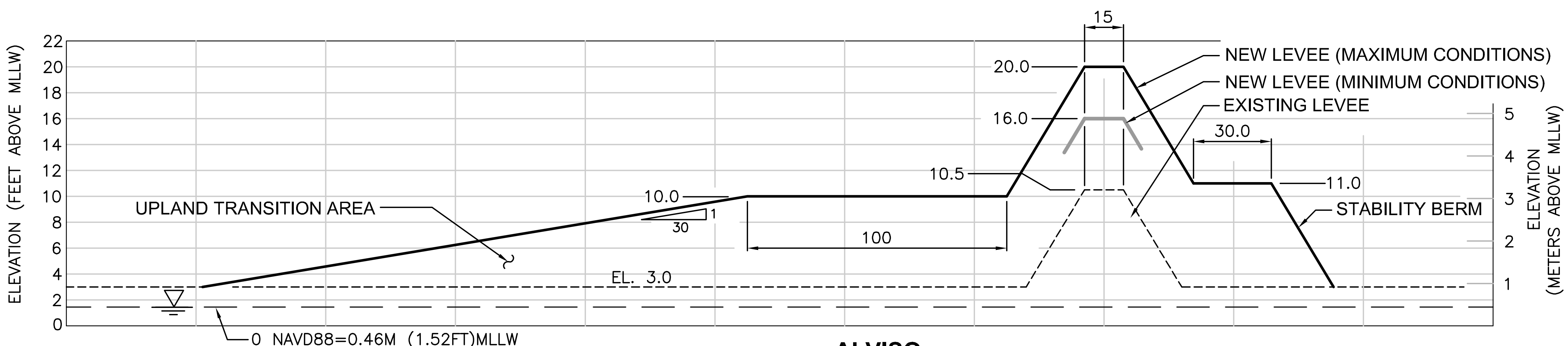
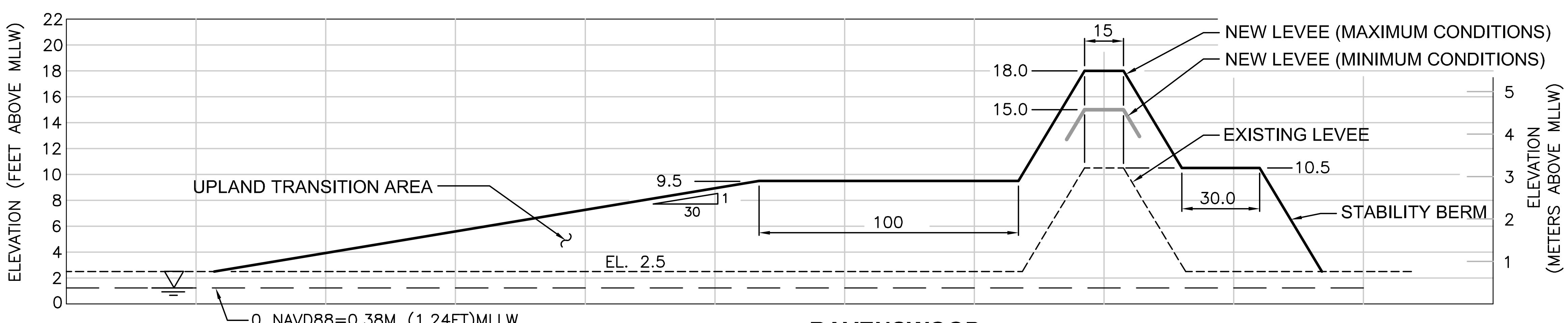


ALVISO



RAVENSWOOD

NOTE: ALL SLOPES ARE 3:1 U.O.N.
ALL ELEVATIONS ARE IN FEET MLLW, U.O.N.

PWA *figure 11*

CONCEPTUAL LEVEE CROSS-SECTIONS WITH OUTBOARD MARSH

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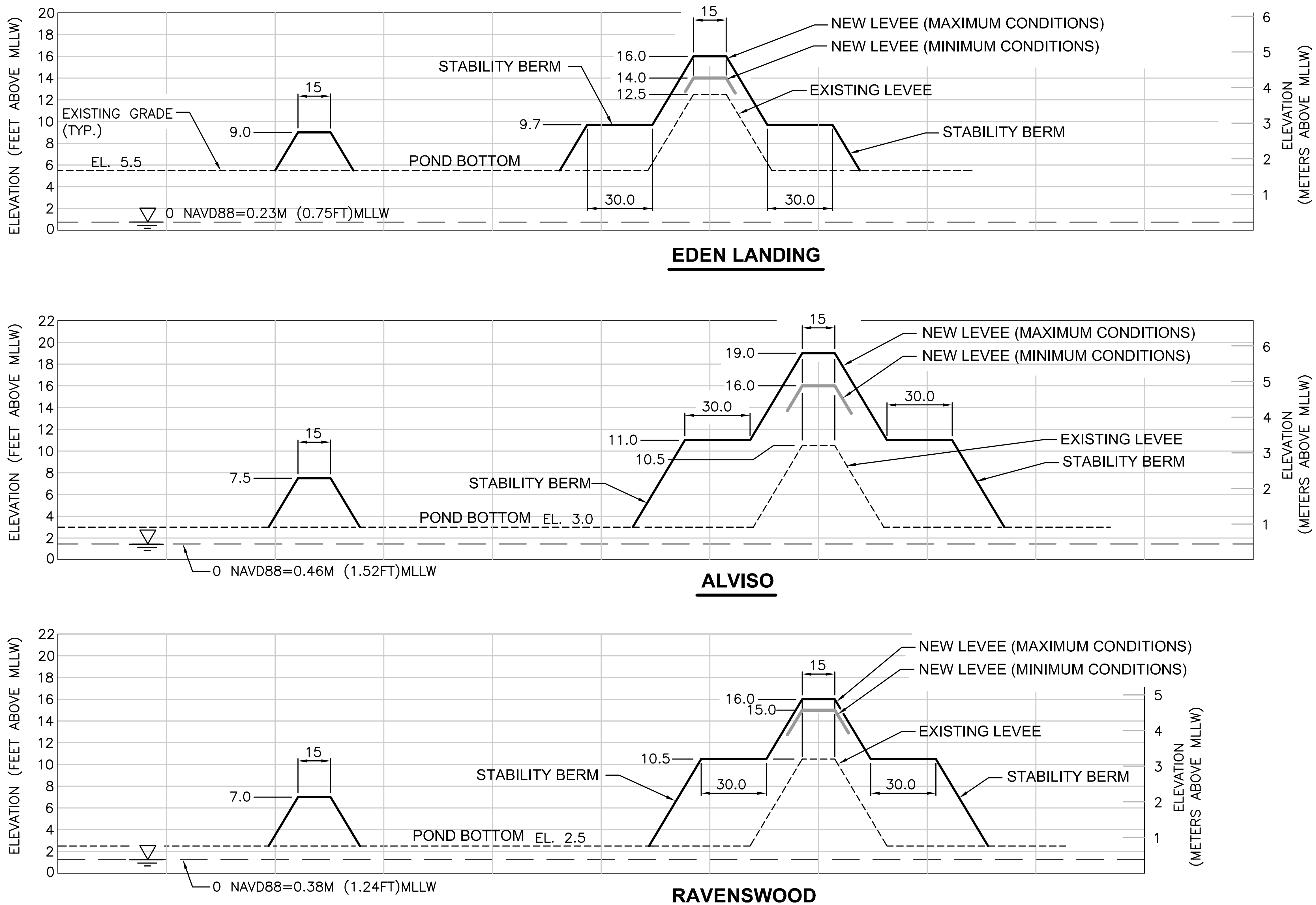

PWA *figure 12*

CONCEPTUAL LEVEE CROSS-SECTIONS WITH OUTBOARD MANAGED POND

NOTE: ALL SLOPES ARE 3:1 U.O.N.
ALL ELEVATIONS ARE IN FEET MLLW, U.O.N.

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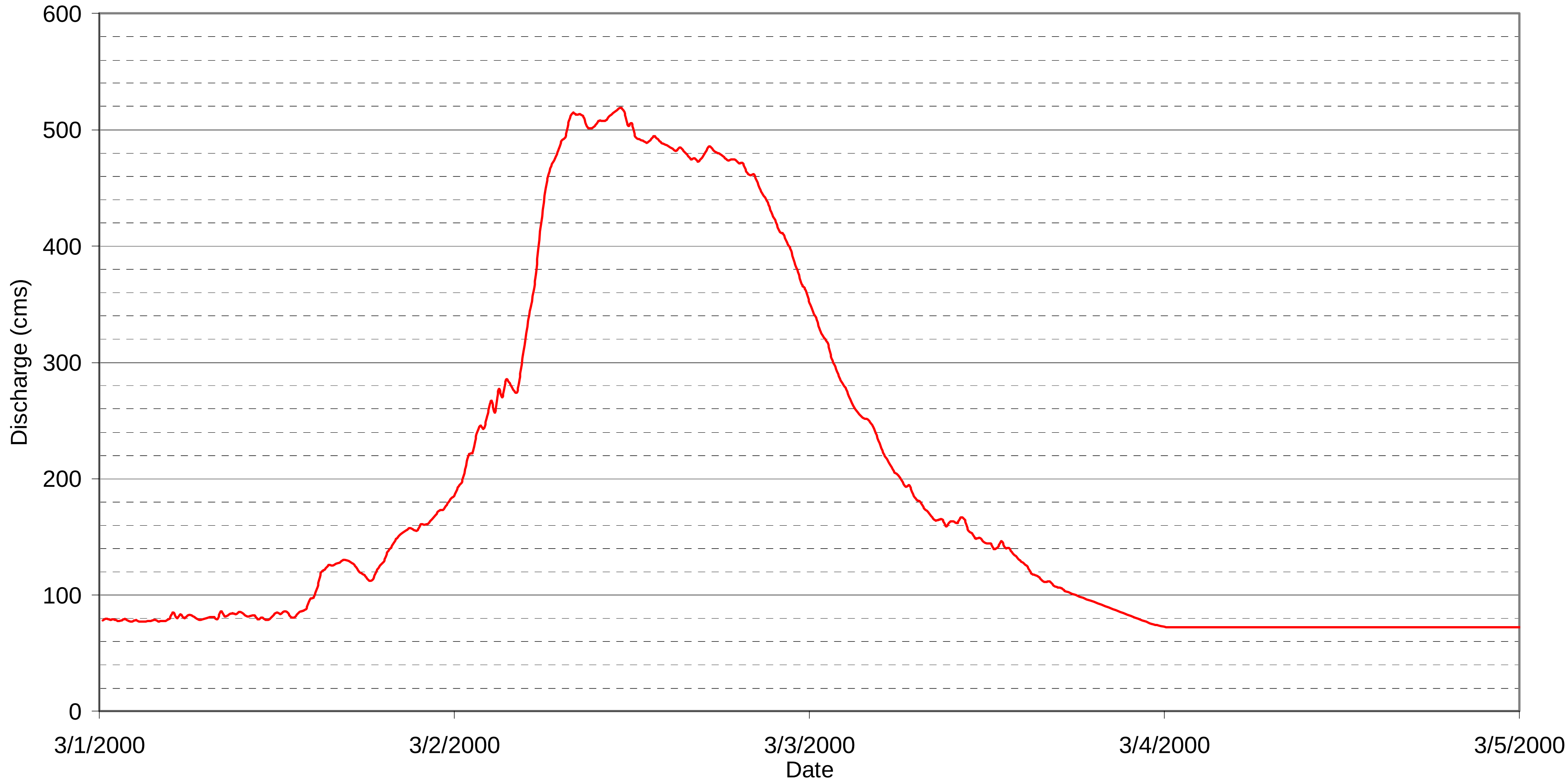


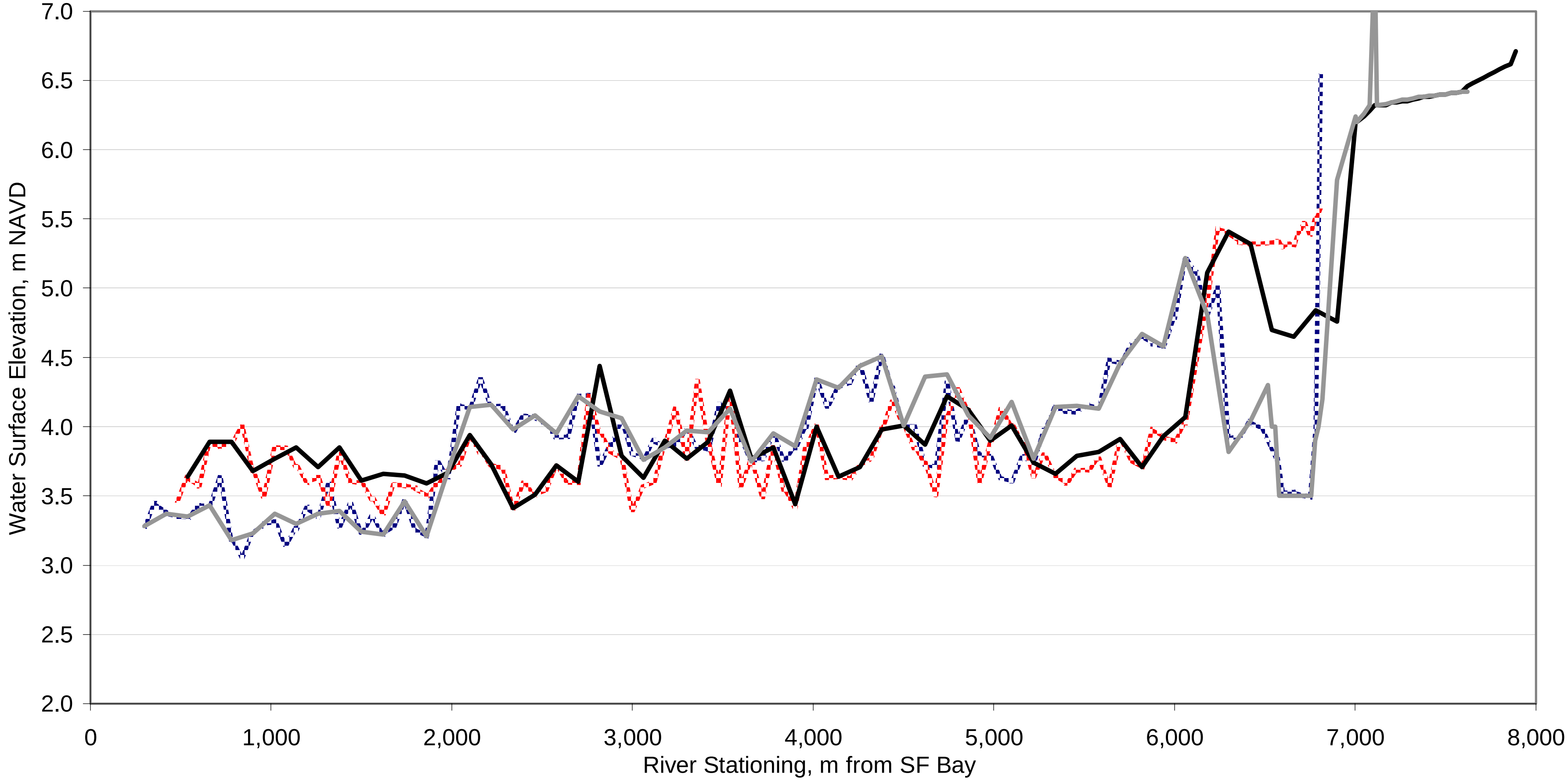
Comments / Sources:

figure 13

South Bay Salt Pond Restoration Project

Design Hydrograph

PWA



Comments / Sources:
UNET levee profile from SCVWD. HEC-RAS levee profile includes recent survey.

Legend:

East Levee (UNET)
West Levee (UNET)
East Levee (HEC-RAS)
West Levee (HEC-RAS)

figure *14*

South Bay Salt Pond Restoration Project

Alviso Slough Levee Crests –
UNET Compared to Baseline

PWA

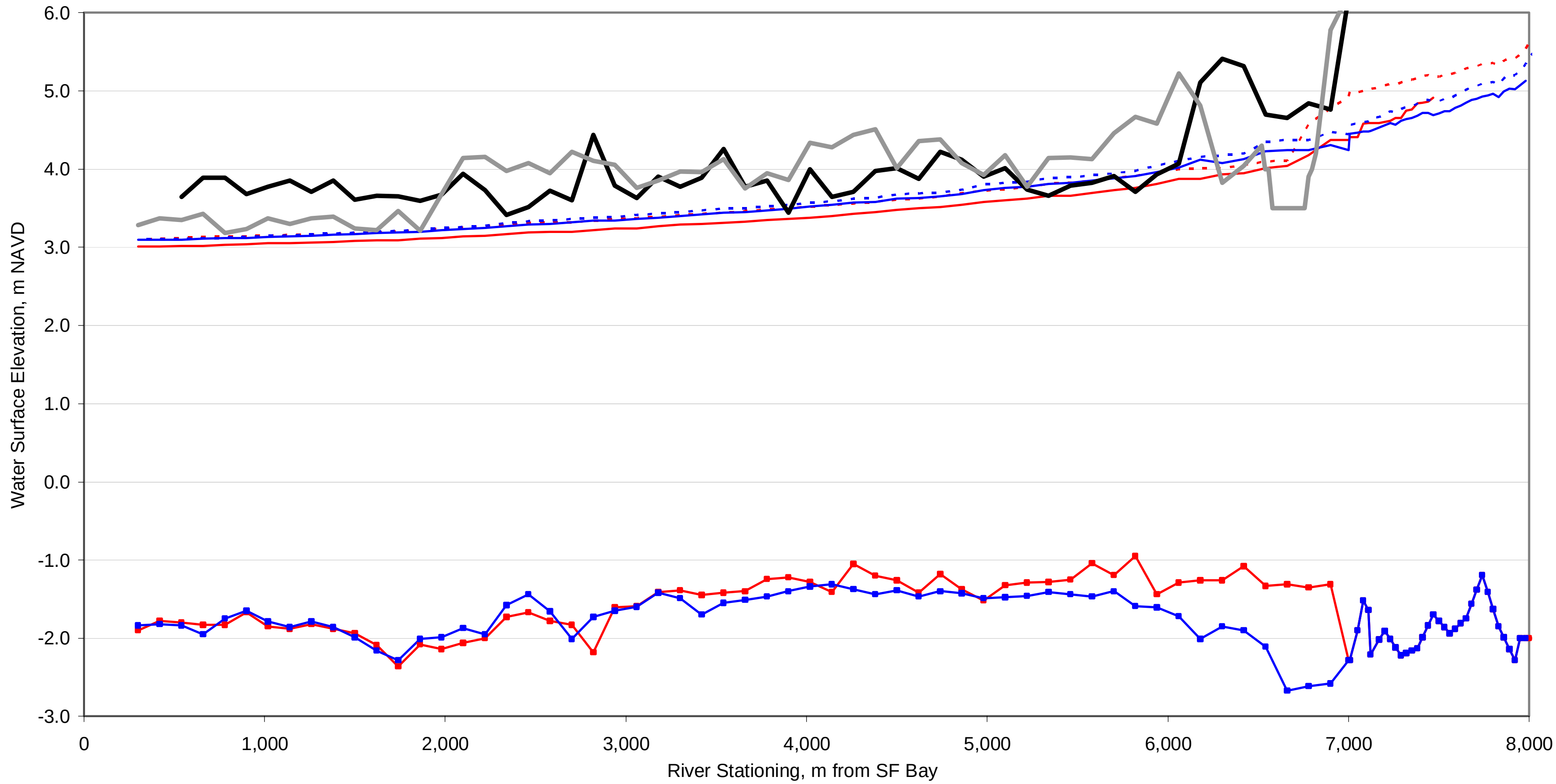
Comments / Sources:
Map Data: USGS (Image)

figure 15

South Bay Salt Pond Restoration Project

Alviso Slough Plan Map

PWA



Comments / Sources:
100-yr Flood Event (Q100 + 10-yr Tide)
Legend:

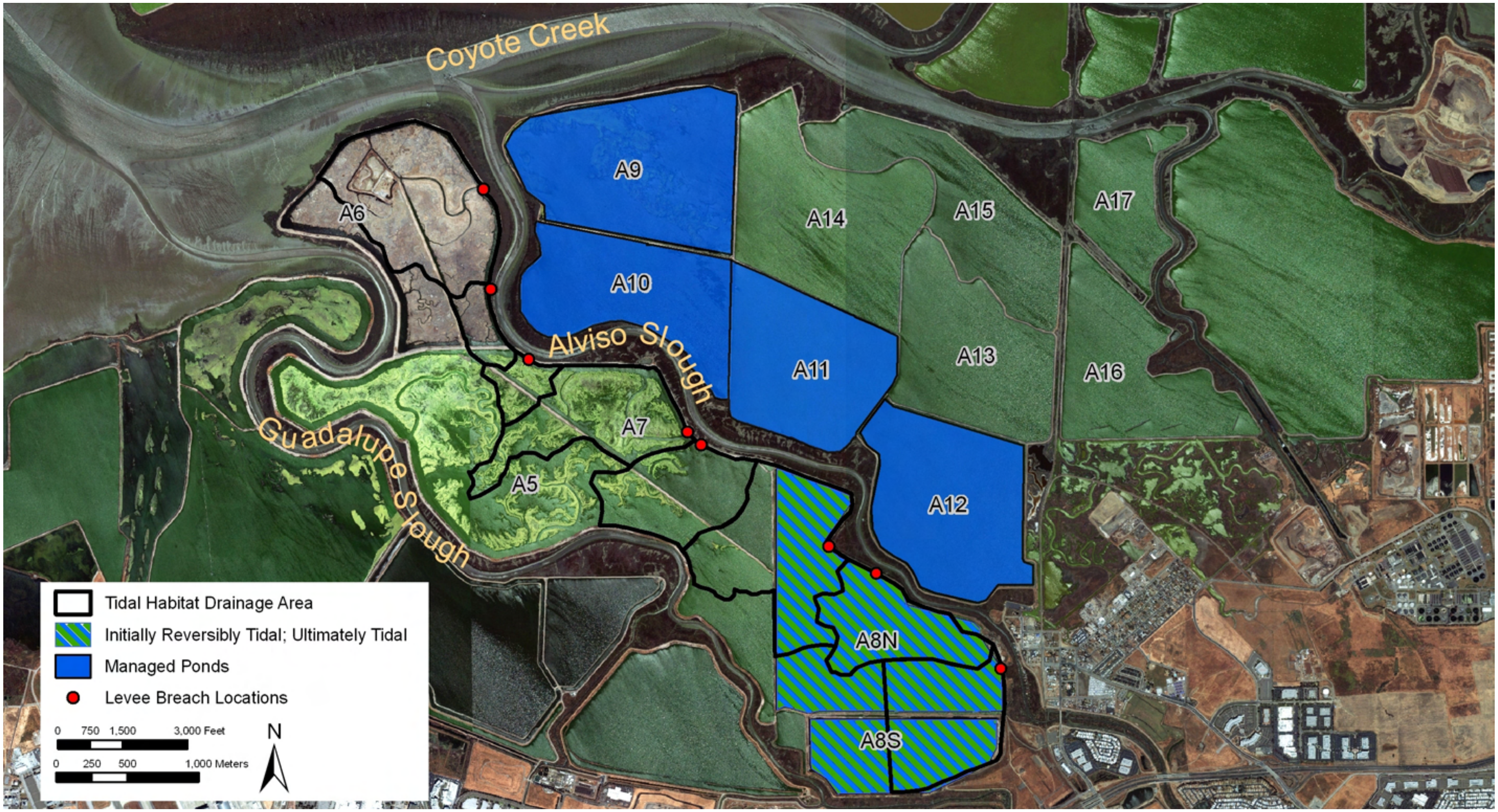
- District's HEC-RAS (Steady)
- Existing Conditions (Steady)
- East Levee Crest
- District's Thalweg (1996)
- District's UNET (Unsteady)
- Baseline Conditions (Unsteady)
- West Levee Crest
- Baseline Conditions Thalweg (2004)

figure 16

South Bay Salt Pond Restoration Project

Project Reach Water Surface Profiles

PWA



Comments / Sources:
Map Data: USGS (Image)

figure *17*

South Bay Salt Pond Restoration Project

Alternative B – Contributing Tidal Habitat Area and Levee Breach Locations

PWA

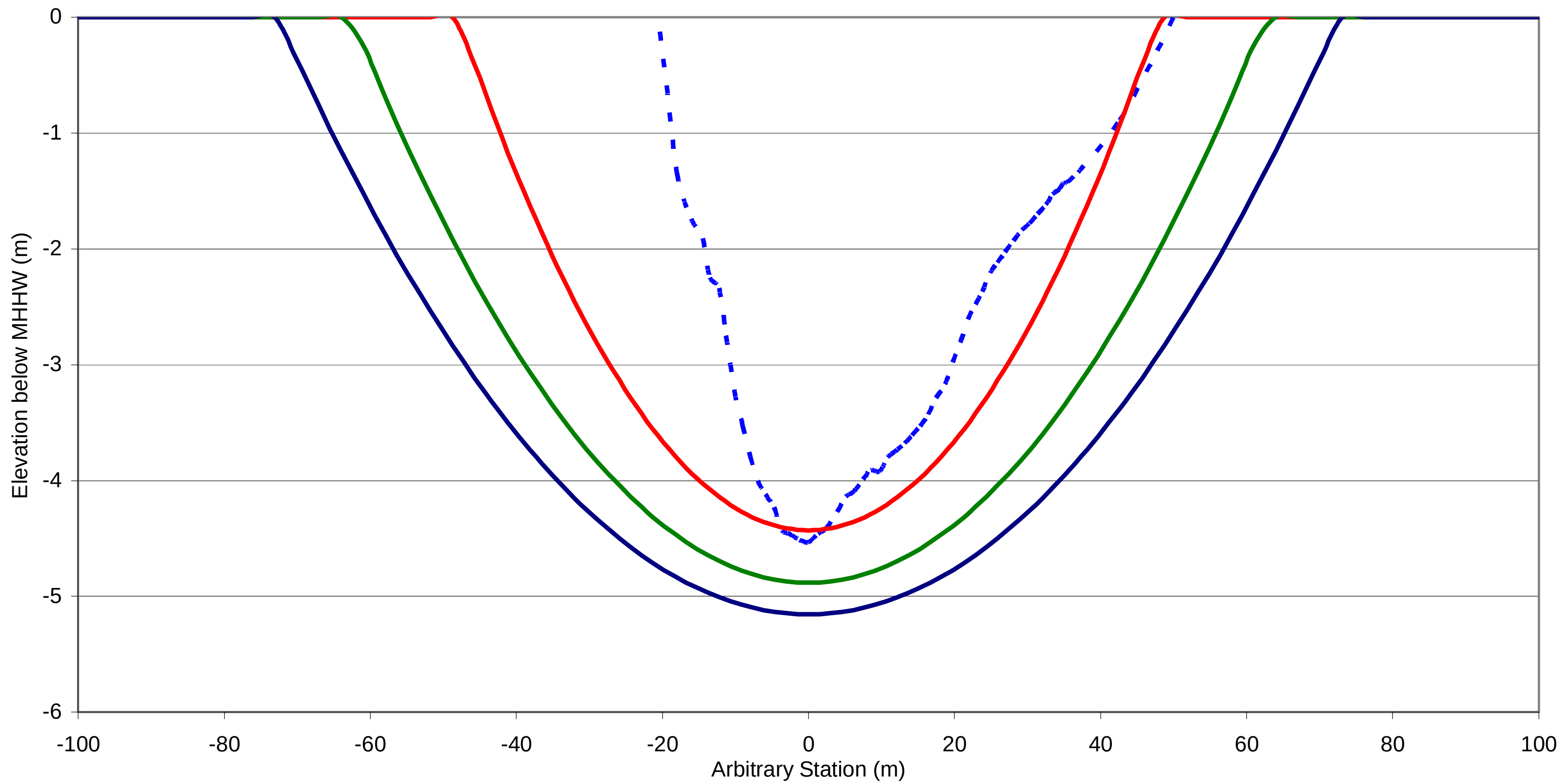
Comments / Sources:
Map Data: USGS (Image)

figure 18

South Bay Salt Pond Restoration Project

Alternative C – Contributing Tidal Habitat Area and Levee Breach Locations

PWA



Comments / Sources:

Cross Section approximately 1,740 m (5,700 feet) upstream of slough mouth.

Legend:

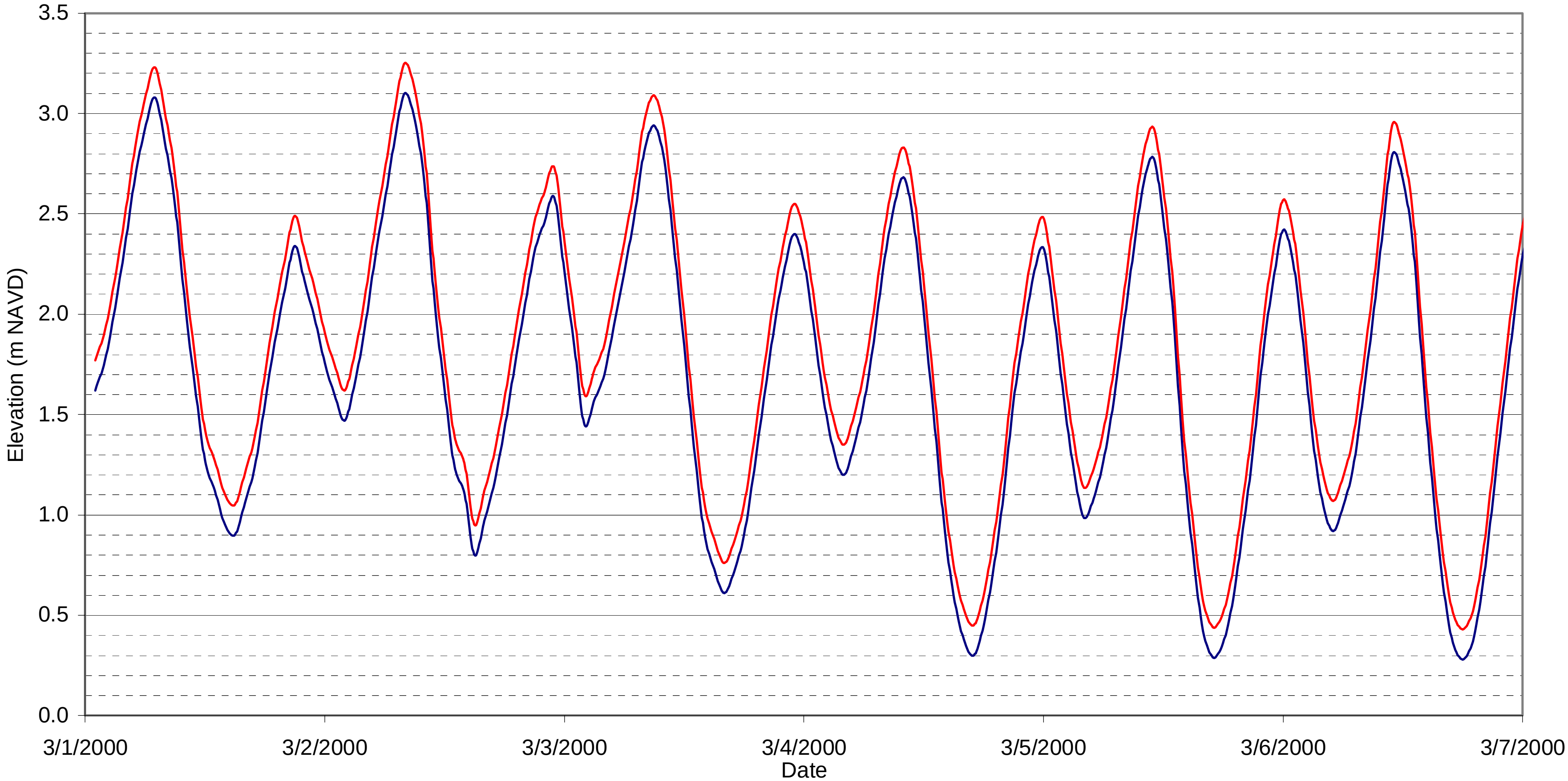
- Baseline Conditions
- Alternative A
- Alternative B-3
- Alternative C-3

figure 19

South Bay Salt Pond Restoration Project

Typical Slough Channel Cross Section

PWA



Comments / Sources:

Water level measured at Dumbarton Bridge and extrapolated to mouth of Alviso Slough. Sea level rise assumed to be 0.15 m (0.5 feet.).

Legend:

10-Yr Tide Signal

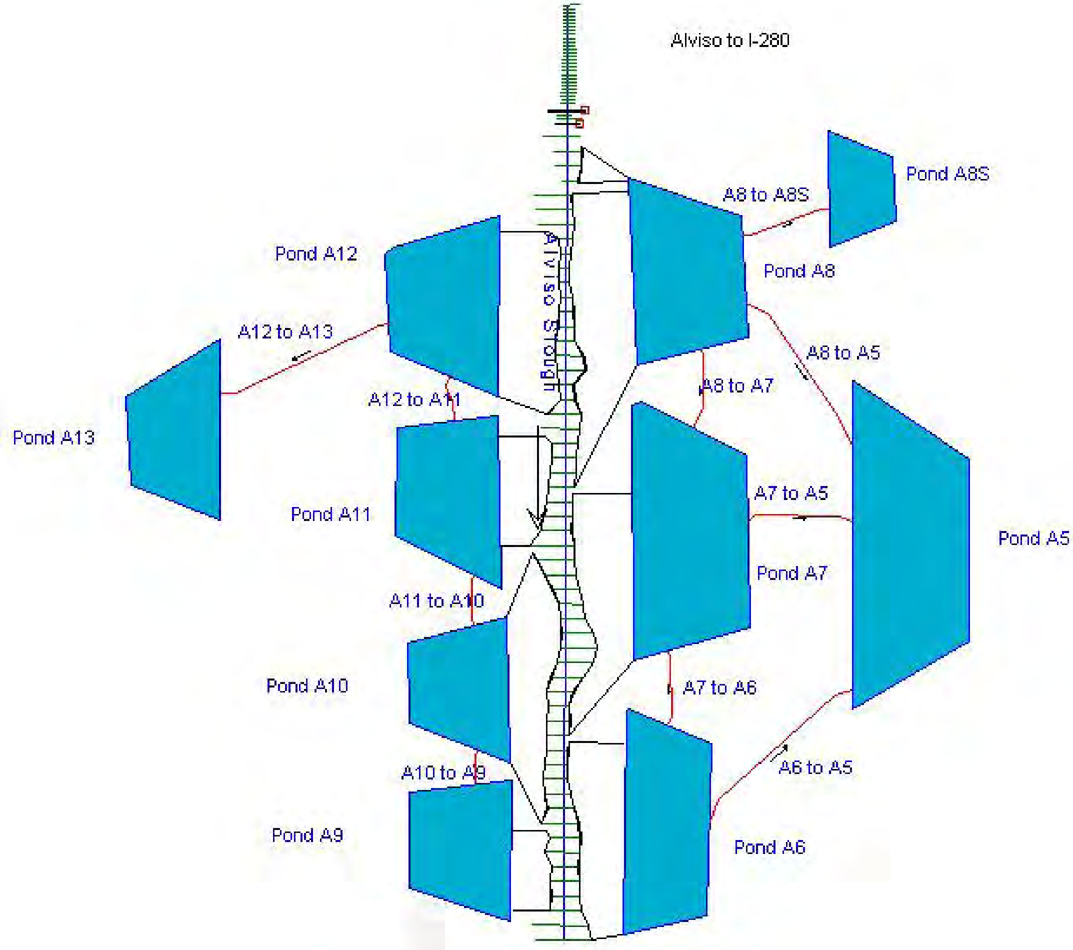
10-Yr Tide Signal with Sea Level

figure 20

South Bay Salt Pond Restoration Project

10-yr Tide Signal and
10-yr Tide Signal with Seal Level Rise

PWA



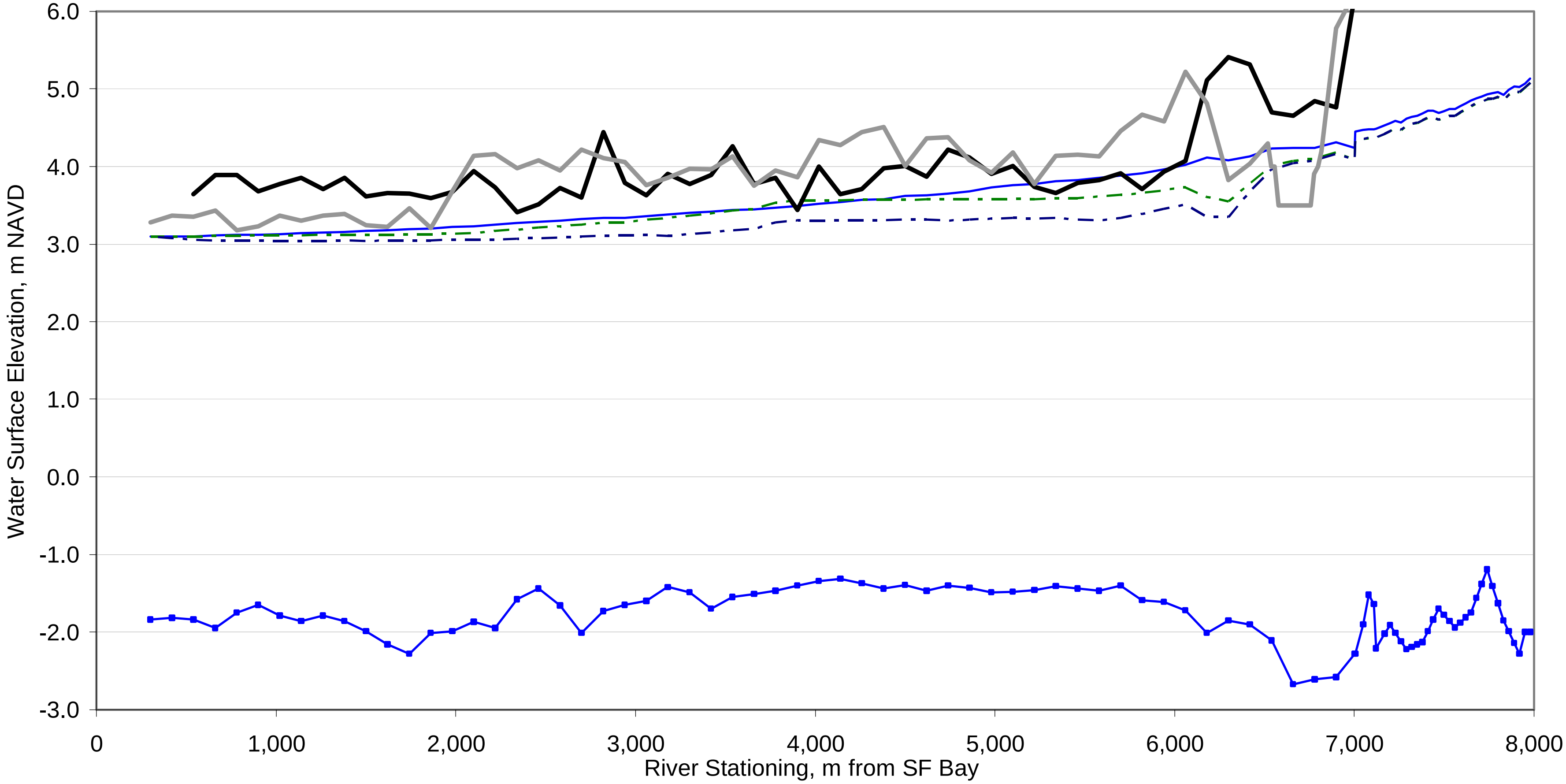
Comments / Sources:

figure 21

South Bay Salt Pond Restoration Project

HEC-RAS Baseline Conditions
Model Schematic

PWA



Comments / Sources:
Baseline Conditions and Short Term Project Alternatives (B-1 and C-1)
Legend:

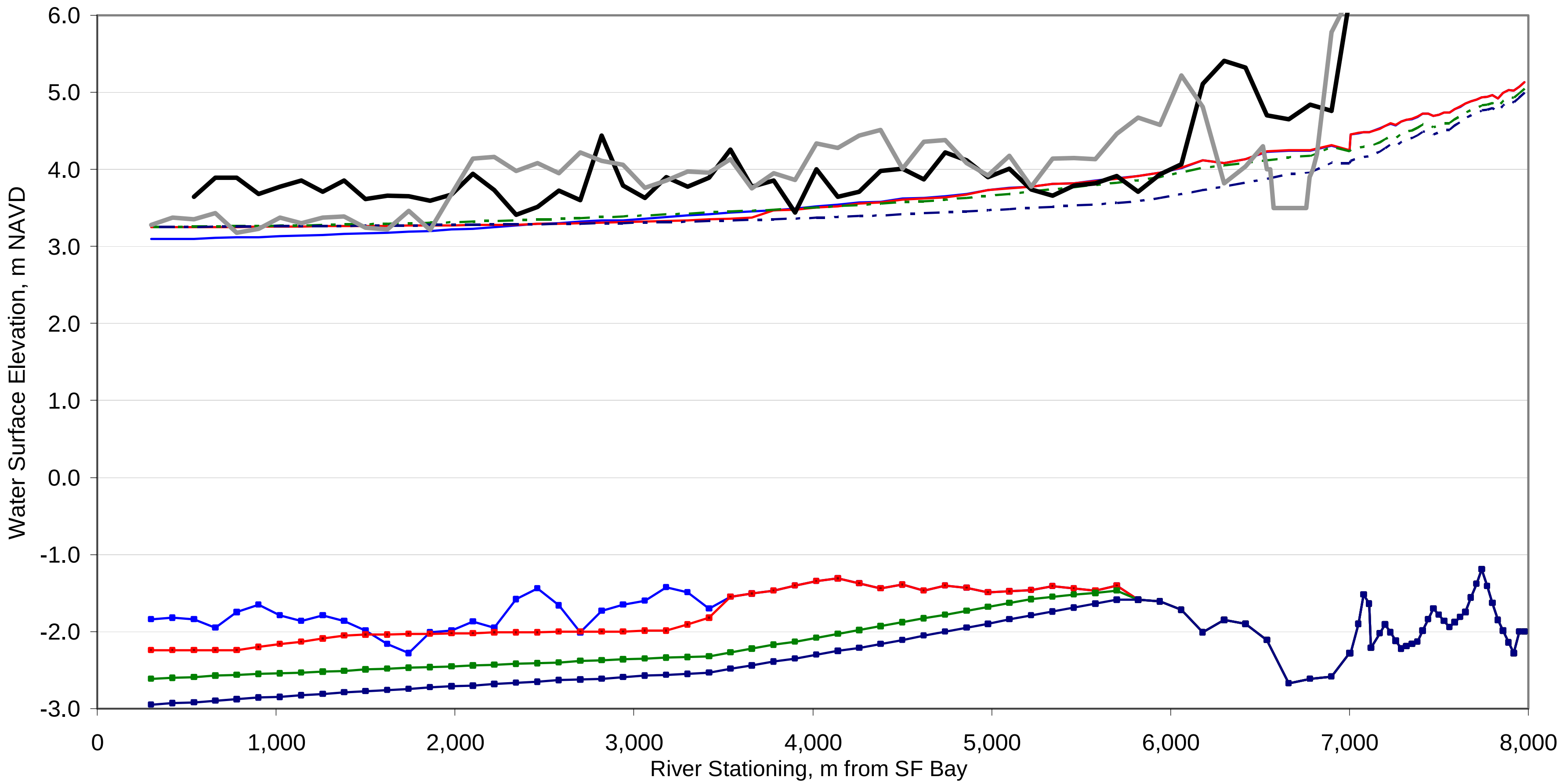
Baseline Conditions (Unsteady)
Alternative B-1
Alternative C-1
East Levee Crest
West Levee Crest
Baseline Conditions Thalweg

figure 22

South Bay Salt Pond Restoration Project

Project Reach Water Surface Profiles – Short Term Conditions

PWA



Comments / Sources:

Baseline Conditions and Long Term Project Alternatives (A, B-3, and C-3). Long Term model boundary conditions have been increased 0.16 m (0.5 feet) to account for sea level rise.

Legend:

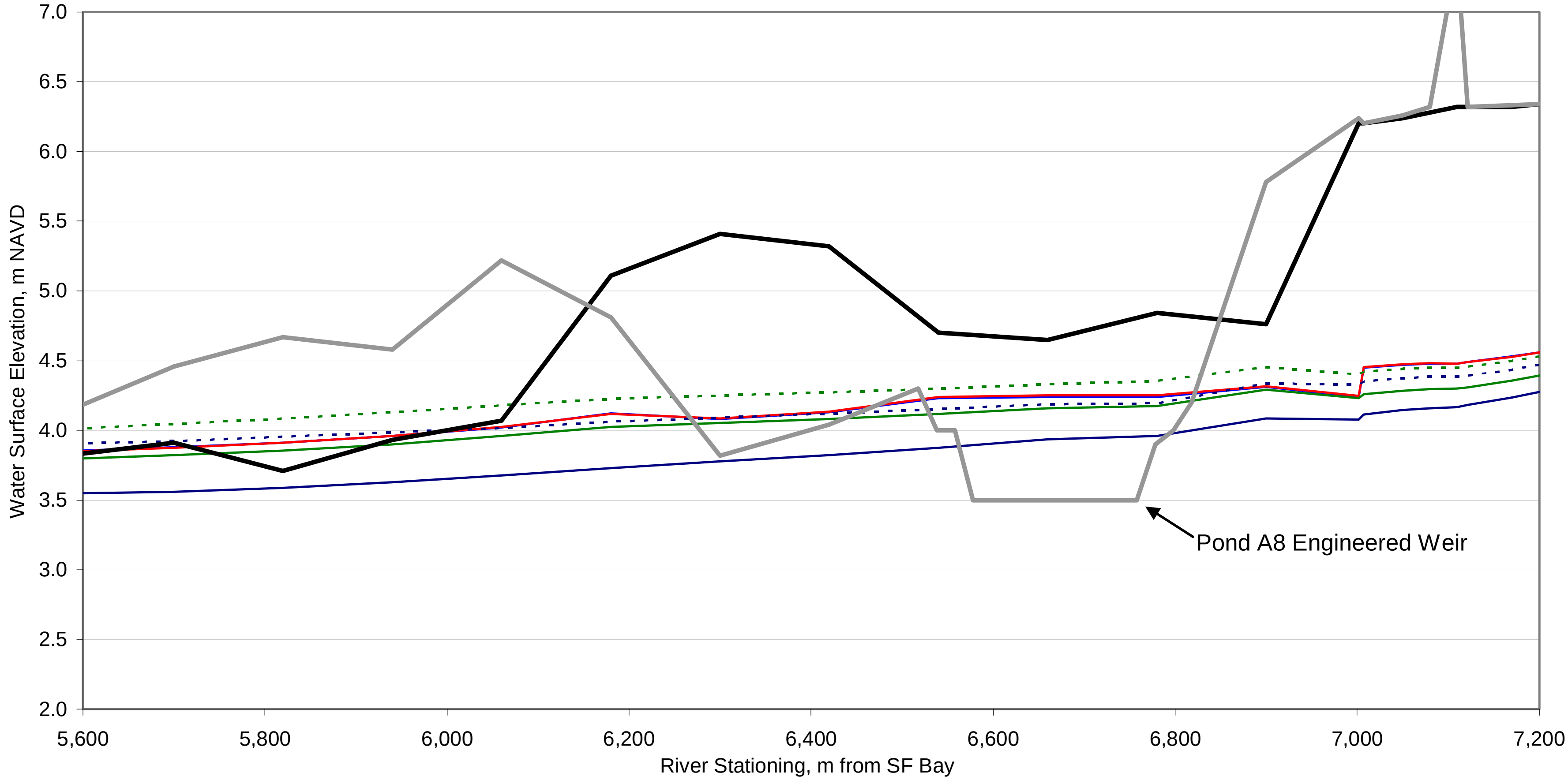
Baseline Conditions (Unsteady)
Alternative A
Alternative B-3
Alternative C-3
East Levee Crest
West Levee Crest
Baseline Conditions Thalweg
Alternative A Thalweg
Alternative B-3 Thalweg
Alternative C-3 Thalweg

figure 23

South Bay Salt Pond Restoration Project

Project Reach Water Surface Profiles – Long Term Conditions

PWA



Comments / Sources:
Baseline Conditions and All Project Alternatives (A, B-1, B-2, B-3, C-1, C-2, and C-3)
Legend:

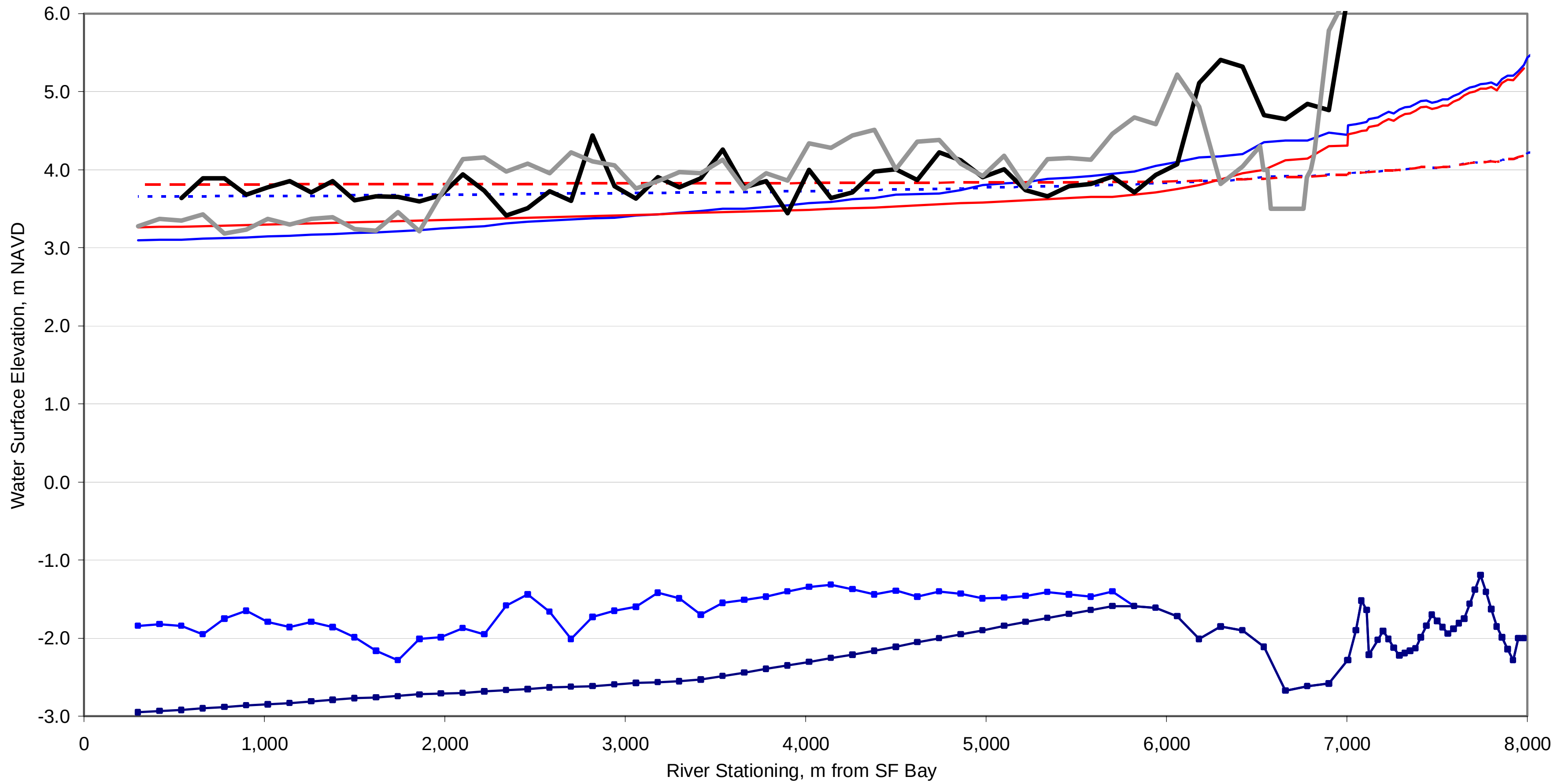
Baseline Conditions	Alternative A
Alternative B-2	Alternative B-3
Alternative C-2	Alternative C-3
East Levee Crest	West Levee Crest

figure 24

South Bay Salt Pond Restoration Project

Project Reach Water Surface Profiles –
Long Term Conditions

PWA



Comments / Sources:

Steady state Existing Conditions and Alternative C-3 results. The Q100 profiles are the 100-yr flow with a 10-yr tide downstream and the Q10 profiles are the 10-yr flow with a 100-yr tide downstream. The 10-yr and 100-yr tide levels for Alternative C-3 include sea level rise (0.15 m).

Legend:

Existing Conditions (Q100)
Alternative C-3 (Q100)
Baseline Conditions Thalweg
East Levee Crest
Existing Conditions (Q10)
Alternative C-3 (Q10)
Alternative C-3 Thalweg
West Levee Crest

figure 25

South Bay Salt Pond Restoration Project

Project Reach Water Surface Profiles – Steady State Water Level Analysis

PWA