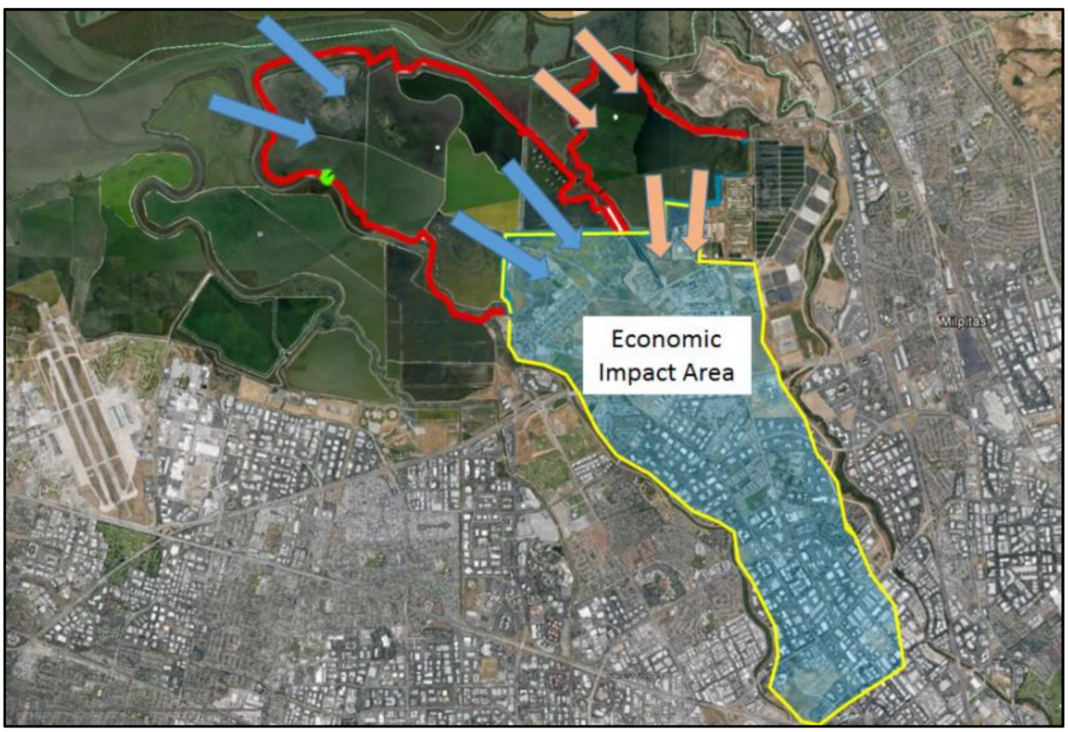
Appendix D2

Tidal Flood Risk Analysis Summary Report

US Army Corps of Engineers®

TIDAL FLOOD RISK ANALYSIS SUMMARY REPORT

South San Francisco Bay Shoreline (SSFBS) Study



June, 2015

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ACRONYMS

ACE	Annual Chance of Exceedance
AEHW	Annual Extreme High Water level
AEP	Annual Exceedance Probability
AFB	Alternative Formulation Briefing
ATR	Agency Technical Review
CDF	Cumulative Distribution Function
CNP	Conditional Non-exceedance Probability
CPT	Cone Penetrometer Test
DTM	Direct Transfer Method
DQC	District Quality Control
EAD	Equivalent (or Expected) Annual Damage
EC	Engineer Circular
EGM	Economic Guidance Memorandum
EIA	Economic Impact Area
EM	Engineer Manual
ENSO	El Nino-Southern Oscillation
ER	Engineer Regulation
ETL	Engineer Technical Letter
FRM	Flood Risk Management
FSM	Feasibility Scoping Meeting
FWOP	Future WithOut Project
FWS	United States Fish and Wildlife Service
GEV	Generalized Extreme Value
HEC-FDA	Hydrologic Engineering Center-Flood Damage reduction Analysis
LMSL	Local Mean Sea Level
MEHW	Monthly Extreme High Water level
MHHW	Mean Higher High Water
MLLW	Mean Lower Low Water
MSL	Mean Sea Level
NAVD88	North American Vertical Datum of 1988
NED	National Economic Development
NOAA	National Oceanographic and Atmospheric Administration

NTDE	National Tidal Datum Epoch
PDO	Pacific Decadal Oscillation
PED	Preconstruction, Engineering and Design
RSLR	Relative Sea Level Rise
SCVWD	Santa Clara Valley Water District
SLC	Sea Level Change
SPT	Standard Penetration Test
SSFBS	South San Francisco Bay Shoreline
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
WPCP	Water Pollution Control Plant
WSE	Water Surface Elevation

1.0 INTRODUCTION

1.1 PURPOSE OF THIS REPORT

This report was prepared to summarize analyses performed to determine if there is an economically justified (i.e., benefits exceed costs) tidal flood risk management project for the South San Francisco Bay Shoreline (SSFBS) Study under the three sea level change (SLC) scenarios used in U.S. Army Corps of Engineers (USACE) studies. USACE policy, as prescribed by EC 1165-2-212 (USACE, 2011) and ER 1100-2-8162 (USACE, 2013a), require that three specific USACE SLC scenarios be considered when formulating and evaluating plans for a study. The results for this study show that there are significant national net economic benefits of a project under each of the three USACE SLC scenarios, with annual net benefits ranging from approximately $15 million to $37 million. These results by themselves are not intended to determine Federal interest or a recommended plan. The results from this summary report, along with other information, will be used to determine Federal interest and a recommended plan. Federal interest and recommendations for a future project are documented in the Integrated Feasibility Report and Environmental Impact Statement/Report.

This report is organized into seven sections that address the second, third, and fourth questions listed above, regarding the mechanism, consequences, and probability of flooding: this Introduction section, followed by an Overview of the Flood Damage Analysis section, and then the three technical sections feeding into the flood damage analysis (Coastal Engineering Technical Summary, Geotechnical Engineering Technical Summary, and Economics Technical Summary), a Summary section, and References. The results supporting economic justification of a Federal flood risk management project are briefly given in the Overview of the Flood Damage Analysis section, in more detail in the Economics Technical Summary section, and are also summarized in the Summary section.

1.2 BACKGROUND

The SSFBS study is authorized to study the bay shoreline for all of Santa Clara County and large portions of Alameda and San Mateo Counties, California. Due to the enormity of the study area, the study was initially divided into four smaller interim study areas (see Figure 1). The Alviso Ponds and Santa Clara County interim study area is the first interim study to be conducted. As recently as 150 years ago, the study area was dominated by tidal marsh habitat. The open water areas of the bay were very nearly surrounded by broad expanses of tidal mudflats and even broader areas of tidal marsh. Historic tidal marshlands were diked off from bay inundation beginning in the 1930s primarily to create solar salt-harvesting ponds. The tidal marsh was replaced with a series of ponds separated by dikes not designed for flood risk reduction. The system of ponds and dikes (also referred to as pond levees in this report), although not designed or intended as flood risk management structures, have been largely effective in reducing flood damages for an area adjacent to the bay with an elevation that is below mean sea level." Subsidence contributes to the study area's flood risk. The Santa Clara Valley has experienced regional land subsidence since the 1900s, primarily due to large-scale groundwater withdrawals. Subsidence was largely arrested by the mid-1960s, when state water deliveries began to arrive in Santa Clara County, but some areas, such as portions of the community of Alviso, are still several feet below mean sea level.

The SSFBS study analyzed the entire Alviso Ponds and Santa Clara County shoreline for future without-project conditions and initial planning measures through the USACE Feasibility Scoping Meeting (FSM) milestone. The study effort through the FSM milestone was quite extensive, requiring a large investment of time and funds. After the FSM the non-Federal study partners requested that this initial interim feasibility study be re-scoped to a smaller area with high potential flood risk reduction and ecosystem restoration benefits, with other areas being studied in subsequent phases. The community of Alviso and surrounding ponds (the Alviso Economic Impact Area (EIA)) was chosen for this first interim study area.

The Alviso EIA is also shown in Figure 1to show the reduction in size for the re-scoped study area. Initial work under the re-scoping primarily focused on the "USACE High" SLC scenario from EC 1165-2-212 (USACE, 2011); however, the results of that work indicated that further work was necessary to determine economic justification under all three scenarios. Therefore, the analyses summarized in this report cover all three USACE SLC scenarios in sufficient technical detail to determine if there is economic justification for a flood risk reduction plan in the re-scoped study area. The analyses presented herein are a consolidation of tidal flood risk information that is also captured in technical appendices to the Integrated Feasibility Report and Environmental Impact Statement/Report.

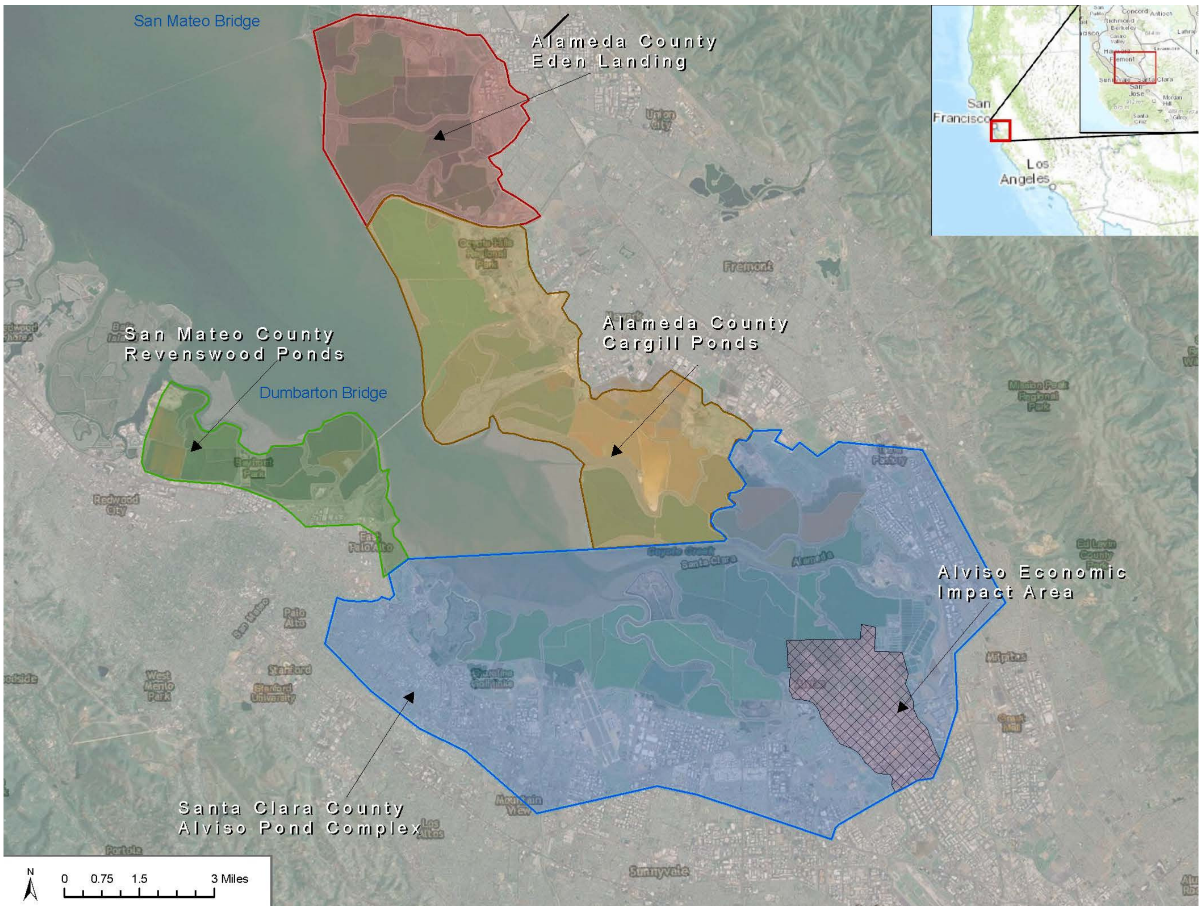


Figure 1: South San Francisco Bay Shoreline Study Areas

1.3 CHARACTERIZING FLOOD RISK

Characterizing flood risk involves the qualitative or quantitative description of the nature, magnitude and likelihood of the adverse effects associated with the flood hazard. The purpose of characterizing flood risk is to support decisions related to reducing the risk to people and property in the floodplain. Characterizing flood risk requires answering four important questions:

1. What can go wrong?
2. How can it happen?
3. What are the consequences?
4. How likely is it to happen?

The goal of the risk analysis that has been completed for the South San Francisco Bay Shoreline Feasibility Study is to answer these four questions in sufficient detail to support decisions that may reduce flood risk in the study area. The study is focused on reducing the risk of coastal flooding, which could happen if water from the bay overtops or breaches the non-engineered pond dikes that currently separate the bay from the community of Alviso and other people and property in the city of San Jose, CA. The consequences of a coastal flood event in the study area would be devastating: the community of Alviso is located at an elevation below mean high tide, and the region's largest water pollution control plant is located adjacent to the town in the floodplain. Thus, the answers to the first three questions posed above are relatively straightforward. The fourth question (likelihood) is the most challenging to answer, and requires the greatest level of effort and analysis.

1.4 CHARACTERIZING THE FLOODPLAIN AND DIKE-POND SYSTEM

The Alviso economic impact area (Figure 1) is located in a flood plain with elevations that are typically at mean lower tide level (0 feet NAVD88). The community is protected from tidal flooding by an array of dikes and ponds that were once part of an integrated system for commercial salt production that was owned and operated by Cargill, Incorporated. The operation and maintenance of the dike pond system was transferred to U.S. Fish and Wildlife Service (FWS) in 2003. The FWS has also made modifications to water control structures and has breached several internal dikes that previously divided ponds to facilitate the development and expansion of habitat in the south bay. Water surface elevations in the managed ponds are at approximately mean sea level.

The dikes are not engineered and were not constructed or operated with the intent of managing flood risk. The dikes were constructed by pioneering into former tidal marsh and incrementally raising grades over time to both increase height, and to counteract loss of grade due to subsidence of soft foundation soils. Crest elevations and section width were maintained by borrowing from adjacent pond bottoms or modifying the dike section (e.g. windrowing and grading). Significant reaches of outer and inner dike would overtop at elevation 10.5 and 7 feet NAVD88, respectively (Figure 2). All dikes are characterized by non-uniform height and width, sparse vegetative cover, variable soil types, and unknown construction quality.

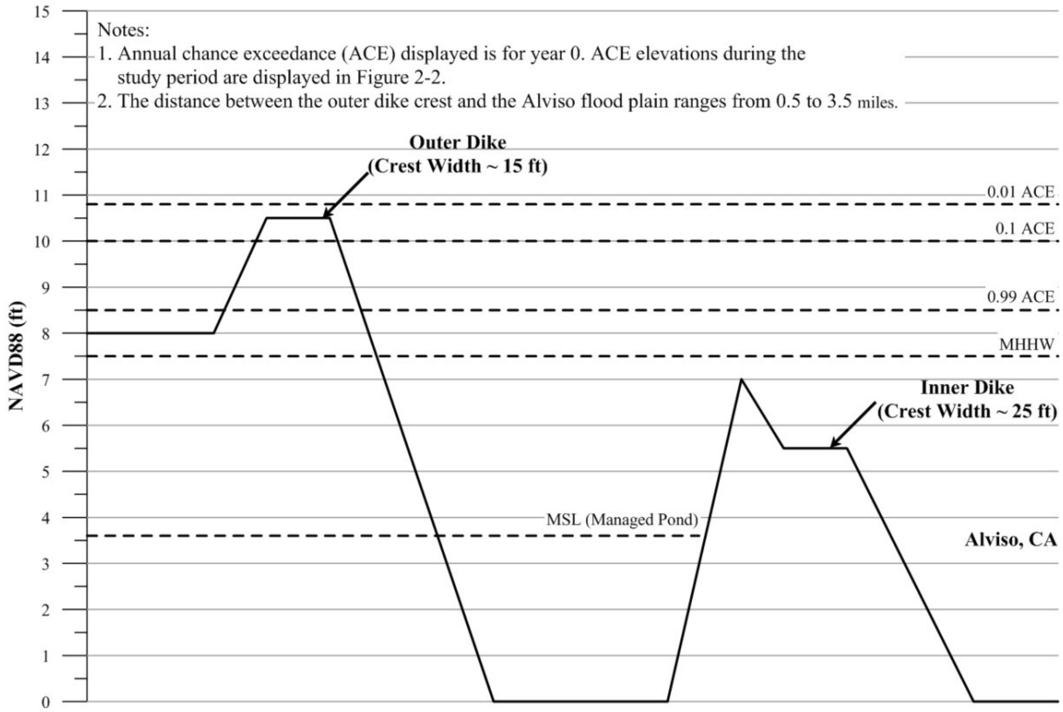


Figure 2: Typical cross-section of dike pond system and relevant elevations.

1.4.1 PAST PERFORMANCE

The community of Alviso could be flooded from both riverine and coastal events. In 1983 significant flooding occurred in Alviso from the Guadalupe River. Flooding in Alviso reached depths of 3 to 7 feet and caused major damage to 362, with lesser damage to 13 homes and 40 businesses (SCVWD 1983). Coincident high tides have been suspected to have contributed to the magnitude of flooding experienced in 1983. Nevertheless, there is no documented flooding from which substantial damages were incurred from purely tidal flood events. In the period 1980 to 2010 tidal water levels are estimated to have exceeded elevation 9 feet NAVD88 seven times, of which elevation 10 feet NAVD88 was exceeded four times.

The past performance against tidal flooding appears to have been excellent; however, the risk of overtopping and breach are judged to be high. There are reaches of the existing outer dike that have narrow (1 to 3 feet) elevated sections of the crest suggesting emergency grading and/or piling of material to prevent overtopping. It is likely several overtopping events may have been narrowly avoided with this type of action.

1.4.2 EXISTING CONDITION

It appears that FWS has maintained the dikes to similar lines and grades as those established by Cargill. No improvements to increase reliability or robustness have been executed or are planned. FWS executes maintenance actions similar to those performed by Cargill to restore height and section width to the outer dike. Maintenance efforts are prioritized and addressed on a reactive basis to maintain functionality of the dikes and managed ponds. Wave and tide conditions have been reasonably mild during the FWS ownership, which has proportionally reduced the maintenance efforts compared to those experienced under Cargill ownership.

1.4.3 OPERATION AND MAINTENANCE REGIME

FWS has modified a facility designed for salt production to in order to promote and benefit wildlife. FWS has breached multiple internal dikes to create hydraulic connectivity between several, but not all, managed ponds. The historic configuration of the ponds as independent "cells" provided redundancy in that a breach on the outboard dike would present a flood risk to a discrete location in the study area. Restoration activities have presumably nominally increased the likelihood of water reaching the floodplain and decreased the effectiveness of a targeted response to potential outboard dike breaches.

FWS has strived to maintain the same type and strategy of maintenance actions executed under Cargill. The extent of routine maintenance performed by Cargill from 1995 to 2005 is discussed in Section 4.2.1. Maintenance records were compiled mostly from regulatory permits issued by USACE or other regulatory agencies in the bay area. These records may not reflect all the emergency actions taken in advance of predicted storms or flood-fighting that may have occurred. In the ten year period Cargill performed at least 126 actions that covered approximately 47 miles of dike in the study area. Borrow material to support maintenance actions was historically obtained from pond bottoms and supplemented with off-site borrow and adjacent dike reaches with higher/wider sections.

The continuation of the same maintenance paradigm is likely unsustainable beyond the near term. Neither the availability of borrow, or the volume of borrow can be considered static. The ability to harvest from adjacent ponds is encumbered by past maintenance that has diminished available borrow, and, the FWS's ability to disturb potentially critical wildlife habitat within the managed ponds. Likewise, the practice of modifying reaches of dike crest and/or cannibalizing higher reaches to maintain and prevent against overtopping in lower reaches is finite. This paradigm proved successful in the past but its continued use will begin to increase the likelihood of levee failure in newly narrowed or lowered reaches. Lastly, the volume required to maintain equivalent functionality will increase with SLC.

1.5 CONCLUSIONS

The results presented in this report indicated that there is economic justification for a FRM project under all sea level rise scenarios. The analysis also showed that the annual exceedance probability (AEP) or probability of flooding in any given year for the study period is 32%. While the analysis demonstrated high confidence in the economic justification for a project, the high AEP did not correlate with past performance. The high AEP was believed to be associated with uncertainty in modeling inputs, effectiveness and limitations of HEC-FDA at modeling the study area, and assumptions that simplified the failure mode of the dike-pond system. Corrective actions proposed included more advanced modeling, refinement of model inputs, and/or multi-variant sensitivity analyses. However, all strategies were judged unlikely to improve the confidence of a reported AEP, or quantify the impact of the potential sources of error noted above.

A rigorous review of all model inputs was conducted. All inputs were concluded to reflect appropriate professional judgment, with the levee failure function judged to be the most open interpretation. A simplified sensitivity analysis was conducted by augmenting the levee failure function to "prevent" levee failures from occurring below a 10-year event. This analysis showed that there was still strong economic support for an FRM project and that the AEP could be reduced substantially to roughly correspond to past performance of the dike pond system. However, a correlation with past performance could not be achieved without making the outer dike unrealistically reliable at preventing flooding. The levee failure function was confirmed to be reasonable and to reflect sound engineering judgment. It was concluded uncertainties in all model inputs, the effectiveness of the model as an appropriate tool for diked/leveed communities below the ambient water level, and assumptions applied to the failure mode contributed to lowered confidence in the predicted AEP.

The project delivery team has moved forward with the existing analysis and has acknowledged the risks that remain in communicating flood risk via the tidal flood damage analysis for the existing without and FWOP conditions. The reduced level of confidence in the reported AEP can be attributed to multiple factors that define the complexity of the dike-pond system and floodplain in the study area. While an AEP of 32% may appear to substantially overstate the flood risk for the study period, there is reason to believe that existing flood risk to the Alviso economic impact area is fundamentally different than what past performance would otherwise indicate.

2.0 OVERVIEW OF THE FLOOD DAMAGE ANALYSIS

Most of the key flood risk metrics are an output of the (economic) flood damage analysis model certified for use in USACE feasibility studies – HEC-FDA. These metrics include an estimate of the event-based damages (dollar damage from, for example, a 10% annual chance of exceedance flood event), the expected or equivalent annual damage (EAD), the probability of flooding under the without-project and with-project conditions (annual exceedance probability or AEP), and the likelihood of a levee containing a particular probability flood event (conditional non-exceedance probability (CNP), also called assurance).

The major inputs to the flood damage model and the source of this information are shown in Figure 3 below. They include the water surface profile, levee failure function, interior-exterior flood elevation relationship, value and location of assets in the floodplain, and the relationship between depth of flooding and structure and content damage.

Figure 3: Major Components of the Flood Risk Analysis

WATER SURFACE PROFILE*	•Water surface elevation & probability at outboard dike •Source: Coastal Engineer
LEVEE FAILURE FUNCTION	•Likelihood of outboard dike failure at each water surface elevation •Source: Geotechnical Engineer
INTERIOR-EXTERIOR RELATIONSHIP*	•Relationship between water elevation at the outboard dike and Alviso - accounting for pond storage •Source: Coastal Engineer
FLOODPLAIN ASSETS	•Value and elevation of structures and assets •Source: Economist - field surveys
DEPTH-DAMAGE RELATIONSHIPS	•Relationship between depth of flooding and damage to structures and contents •Source: Guidance memos and other studies

**Denotes inputs that change with sea-level rise*

The HEC-FDA program was used to combine water surface profile data and economic data (structure inventory, etc.) in order to derive a stage-damage function and estimate expected annual damage (EAD) at various intervals of time within the study's fifty-year period of analysis (2017 – 2067) for each SLC scenario. HEC-FDA version 1.2.5a, a USACE certified model, was used and its use complies with the USACE Planning Model Improvement Program for Model Certification.

The consideration of SLC complicates the damage analysis because under each of the SLC scenarios the flood risk is continually increasing into the future. In a typical HEC-FDA model, a base year and a single future year would be entered into the model. The program then assumes a linear relationship between the base year and the future year conditions that have been specified in the model. However, because of the existence of the current system of pond dikes, because future SLC is not expected to be a linear function of time, and because of the need to consider the impact of structure relocations out of the area over the period of analysis, the traditional approach to flood damage modeling in FDA is not appropriate for this analysis. Instead, for this analysis, for each SLC scenario the fifty-year period of analysis (2017 – 2067) was separated into five without-project models – one for each decade of the period of analysis.

The sections below describe each of the major inputs to the flood damage modeling.

2.1 INPUT 1: WATER SURFACE PROFILE AT OUTBOARD DIKE

Updated water surface profile data was developed for each of the three SLC scenarios – USACE Low, Intermediate, and High. The water surface profiles predict how high the water will be for a given likelihood storm event, over the fifty-year period of analysis. To reasonably capture the change in water surface elevations over time as a result of SLC, data was provided for project year zero, or base year (2017) and for every tenth year thereafter over the fifty-year period of analysis. A plot of the USACE Intermediate SLC scenario water elevations over the period of analysis for each probability event is shown in Figure 4 below. More details on the water surface profile development can be found in the Coastal Engineering Technical Summary section of this report.

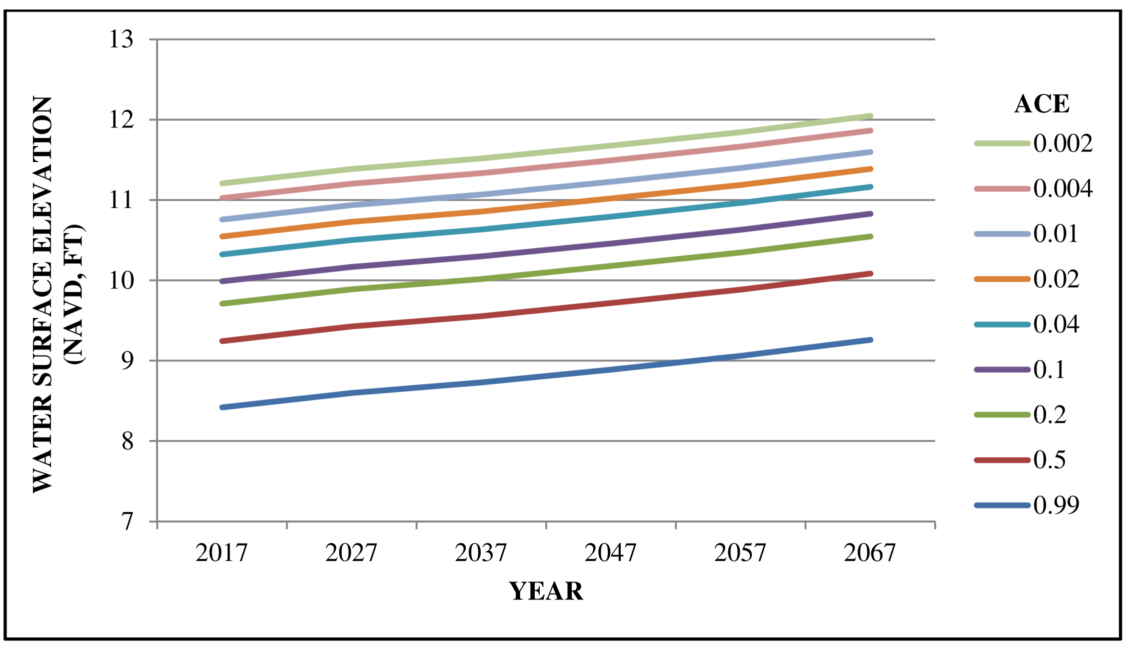


Figure 4: Input 1 (Example) - Water Elevation at Outboard Dike, USACE Intermediate SLC Scenario

2.2 INPUT 2: LEVEE FAILURE FUNCTION

A levee failure function, which indicates the probability of failure given a particular water surface elevation, was developed for the outboard pond dike to be used in the HEC-FDA models. A plot of the data entered into the HEC-FDA models is shown in Figure 5 below. More details on the development of the levee failure function can be found in the Geotechnical Engineering Technical Summary section of this report.

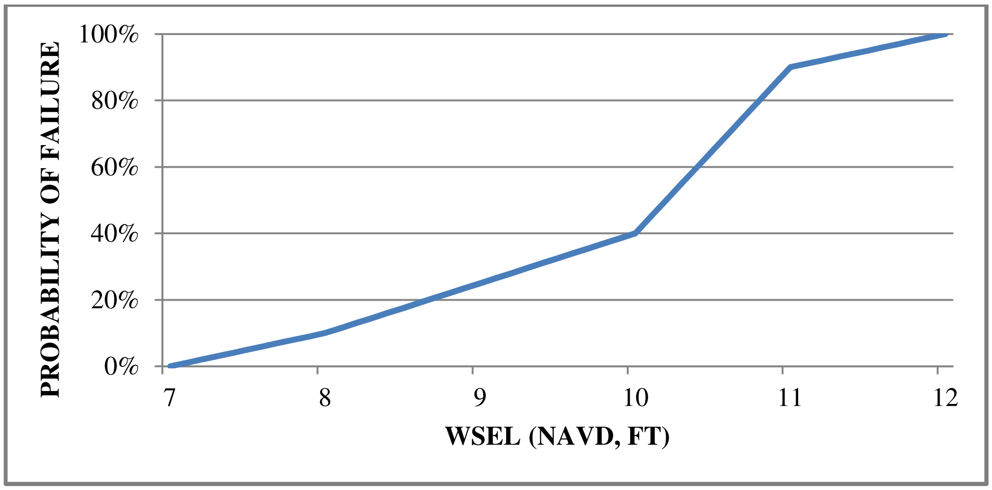


Figure 5: Input 2 - Outboard Dike Failure Function

2.3 INPUT 3: INTERIOR-EXTERIOR WATER SURFACE ELEVATION RELATIONSHIP

A breach of the outboard pond dike would not necessarily result in an equivalent elevation of flood water in the developed area of the basin (i.e., community of Alviso or near the water pollution control plant). The pond system between the outboard dike and the developed area would provide a limited amount of storage. Unless told otherwise, the HEC-FDA model assumes that the flood elevation in the developed area is equivalent to the outboard elevation at the time of dike failure. Not accounting for the storage in the ponds would generally result in an overestimation of the flood elevation and damage. For this reason, it was necessary to develop a relationship between the exterior water elevation at the outboard dike and the interior water elevation in the Alviso EIA in the event of a flood event. This relationship was entered into the HEC-FDA model.

The difference between the exterior and the interior water surface elevation varies over time, by annual chance of exceedance (ACE) flood event, and by SLC scenario, but is generally between zero and two feet. The difference in elevation generally decreases as the events get larger (less likely) because the ponds would fill up faster during larger events. However, there is a scenario in which the interior flood elevation may be greater than the exterior elevation that resulted in the initial dike breach. For example, this can happen when a dike failure occurs at a water surface elevation that is below the astronomical high tide. In this situation the pond storage may be sufficient to keep water from overtopping the inner dike and ponding in the developed area, but because the dike-pond system would then be open to the bay waters, subsequent high tides would be expected to overtop the inner dike (which is considerably lower than the outer dike in some places) and result in flooding in the developed area. For example, an outer dike breach that occurs at an exterior elevation of 7.5 feet would be expected to eventually result in an interior water elevation equivalent to mean high tide (7.8 feet NAVD88) at the base year and increases over time under all future scenarios considered. Likewise an outer dike breach at an exterior elevation of 9.0 feet would eventually equilibrate to an interior water elevation of 7.8 feet. More details on the development of the interior-exterior water surface elevation relationship can be found in the Coastal Engineering Technical Summary section of this report.

2.4 INPUT 4: FLOODPLAIN ASSETS

The structure inventory was conducted in 2010, and no notable changes in land use have occurred since then. The depreciated replacement values of the structures and contents in the floodplain were updated to 2014 price levels. Table 1 below shows the estimated structure and content value (rounded for presentation purposes) for each of the major structure categories in the 0.2% ACE floodplain. In total, more than $800M of structures and contents are exposed to some level of flood risk by the end of the period of analysis. This value should not be confused with event-based or expected flood damage.

Table 1: Structure & Content Value in the 0.2% ACE Floodplain at 2067, High SLC Scenario

Structure Type	Total Structure Value (1,000s)	Total Content Value (1,000s)
Commercial	$333,038	$297,407
Industrial	$70,615	$47,145
Public	$5,068	$1,841
Residential	$56,753	$27,892
Total	$465,474	$374,285

The analysis incorporates an assumption of structure relocation out of the floodplain over time under the future without-project condition. Using the decadal HEC-FDA models for each scenario, if a structure's first floor elevation was 1.5 feet or more below the 10% ACE event water surface elevation for ten years, then that structure was removed from all future HEC-FDA models. The 10% ACE elevation refers to the annual likelihood of that elevation of water occurring in floodplain when accounting for the combined probability of both a water elevation at the outboard dike and a failure of the outboard dike. The elevation also considers the interior-exterior relationship described in Section 2.3. For residential structures, 1.5 feet of flooding above the first floor elevation corresponds to structure damage equal to between one-quarter and one-third of the value of the structure. Over ten years, the chance of experiencing at least one 10% ACE event is 65%, and the chance of experiencing two or more is 26%. In the absence of specific USACE guidance or policy on relocation determination, the relocation threshold was based on professional judgment that considered both the likelihood of flooding and the expected damage.

Table 2 and Figure 6 below show the relocations over time according to the algorithm specified above and under the intermediate SLC scenario.

Table 2: Structure Relocations over Time - Intermediate SLC Scenario

Structure Type	Year					
	2017	**2027**	**2037**	**2047**	**2057**	**2066**
Residential	1035	951	927	884	832	822
Commercial	54	49	48	45	43	42
Industrial	42	22	21	19	15	14
Public	9	5	3	3	2	2
Total Structures	1140	1027	999	951	892	880
Cumulative Relocations	NA	113	141	189	248	260

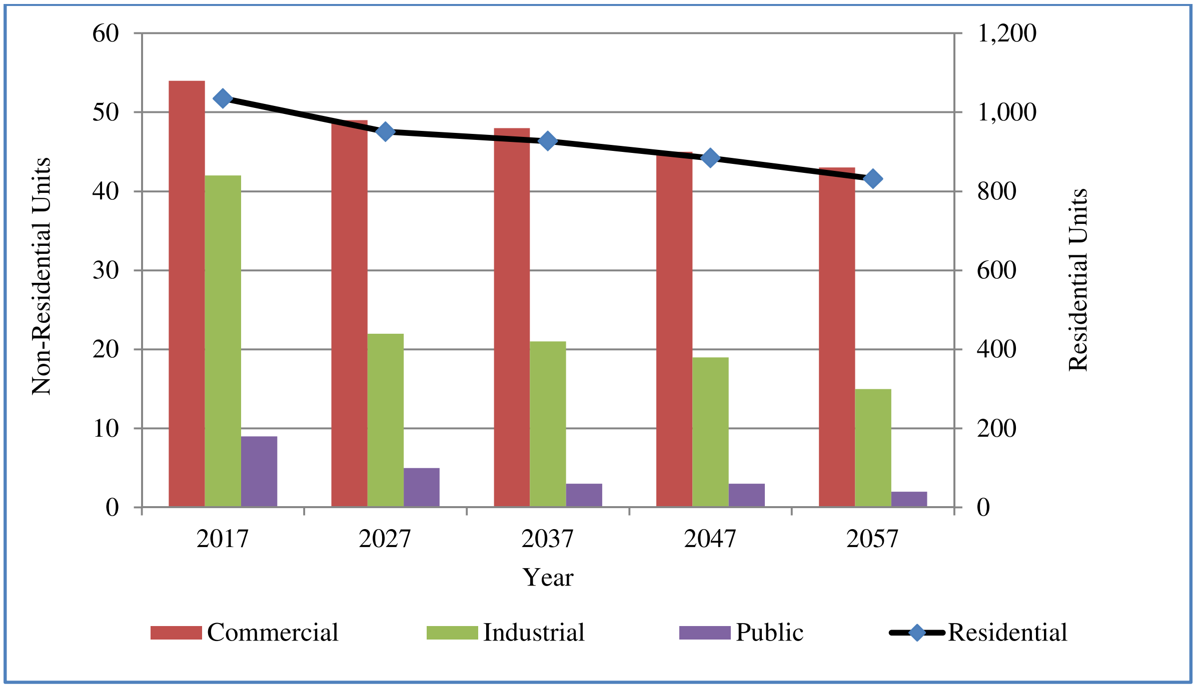


Figure 6: Structure Relocations over Time - Intermediate SLC Scenario

The cost of relocating to similar properties outside of the floodplain was included in the ultimate expected annual damage (EAD) calculations performed outside of the HEC-FDA model. The cost per structure was estimated by USACE Sacramento District Real Estate personnel in 2012.

Located in the study area is the region's largest water pollution control plant (WPCP). The plant has an estimated replacement value of more than $2 billion, and serves 1.4 million people and thousands of businesses. According to the officials at the plant, a flood event at the plant could cause in excess of $100M in damage and could result in the release of untreated sewage into the bay. Because the cost of relocation is expected to be in excess of $2B, it is assumed that the most likely response under the without-project condition would be to construct a ring levee to reduce the likelihood of coastal flood damage at this critical public facility. Additional details related to this assumption can be found in Section 5.0.

2.5 INPUT 5: DEPTH-DAMAGE RELATIONSHIPS

The relationship between the depth of flooding and the damage to the structure and its contents varies by structure type. Also, saltwater is more damaging than freshwater for a given depth of flooding because of the corrosive effects of salt. The depth-damage functions used for this report are primarily taken from the results of an expert panel meeting in Louisiana in 1997. The USACE has only published freshwater depth-damage curves, which is why other sources were used. While the floodwaters may persist for many hours and potentially even days, the analysis uses estimates of short depth-damage relationships because the long-duration curves were developed with the hot and humid southern climate in mind. Using the long-duration curves would likely overestimate the flood damage in the study area.

Figure 7, Figure 8, Figure 9, and Figure 10 below show the relationships for the primary structure types. "SFR1" and "SFR2" stand for Single Family Residential 1-Story and 2-Story, respectively. The freshwater curves are taken from USACE Economic Guidance Memorandum (EGM) 04-01, and are shown for comparative purposes. It is important to note that the data for each relationship were developed independently and from different sources, resulting in inconsistencies across structure types. The curves for the "Displacement" cost category were taken from FEMA's Mitigation BCA Toolkit (FEMA, 2005).

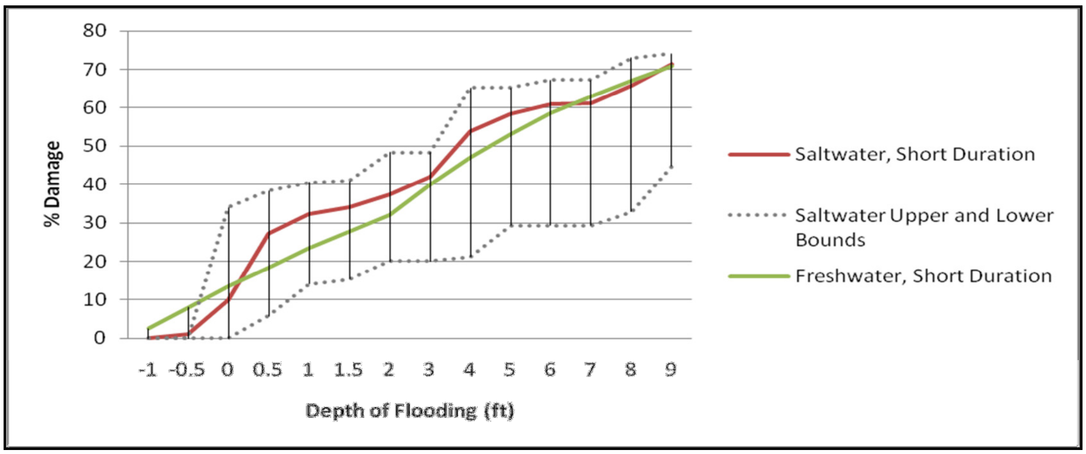


Figure 7: Structure Depth-Damage Relationship, 1-Story SFR

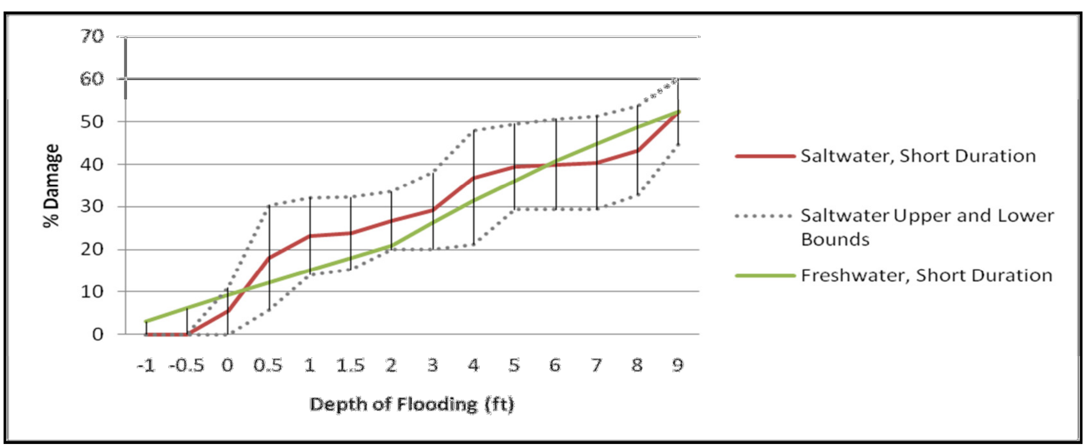


Figure 8: Structure Depth-Damage Relationship, 2-Story SFR

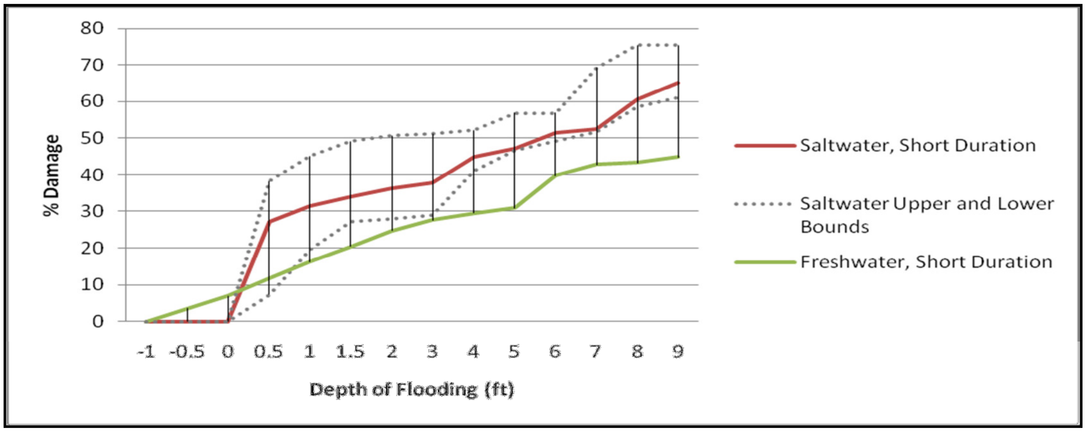


Figure 9: Structure Depth-Damage Relationship, Commercial

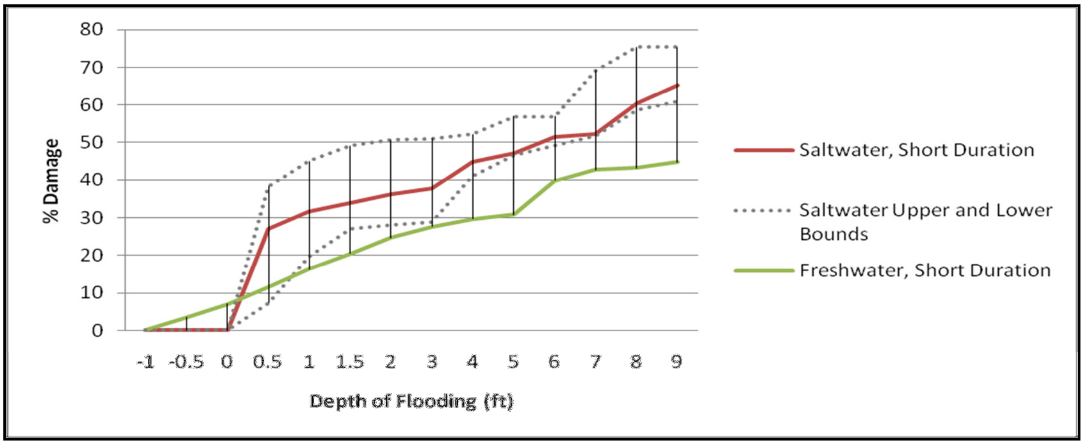


Figure 10: Structure Depth-Damage Relationship, Industrial

2.6 INTERIM RESULT 1: COMBINED PROBABILITY OF FLOODING

Multiplying the probability of the water surface elevation occurring in a given year by the probability that this elevation will cause a failure of the outboard non-engineered dike (which is the primary line of defense currently) results in a probability of a certain elevation of floodwater reaching structures, infrastructure, and people in the study area. Table 3 below displays how the flooding hazard in the study area increases over time for a given SLC scenario; the data shown is for the USACE Intermediate SLC scenario. In the table, “exterior” elevation refers to elevation at the outboard dike, and “interior” elevation refers to water surface elevation in the developed area (i.e., community of Alviso). In 2017, which is the base year for this study, there is just greater than a 1% chance of a flood event that results in 9.5 feet of water in Alviso. By the end of the period of analysis however, the annual probability of getting that same flood elevation in Alviso is 16%. This increase is due to the increase in relative sea level at the study location.

Table 3: Flood Hazard over Time (Intermediate SLC Scenario)

	ACE of Exterior Elevation (Outboard Dike)	99%	50%	20%	10%	4%	2%	1%	0.4%	0.2%
2017	Exterior Elevation (ft)	8.42	9.25	9.71	9.99	10.32	10.55	10.76	11.02	11.21
	Prob. of Levee Failure	0.16	0.28	0.355	0.385	0.55	0.65	0.75	0.9	0.92
	Interior Elevation (ft)	7.81	7.81	7.81	7.81	9.34	9.49	9.63	11.02	11.21
	Combined Annual Prob. of Flooding	16%	14%	7.1%	3.9%	2.2%	1.3%	0.75%	0.36%	0.18%
2027	Exterior Elevation (ft)	8.60	9.43	9.89	10.17	10.50	10.73	10.94	11.20	11.39
	Prob. of Levee Failure	0.19	0.31	0.37	0.45	0.65	0.75	[illegible]	[illegible]	0.93
	Interior Elevation (ft)	7.99	7.99	7.99	8.5	9.4	9.68	[illegible]	[illegible]	11.39
	Combined Annual Prob. of Flooding	19%	16%	7.4%	4.5%	2.6%	1.5%	0.85%	0.37%	0.19%
2037	Exterior Elevation (ft)	8.73	9.56	10.02	10.30	10.63	10.86	11.07	11.33	11.52
	Prob. of Levee Failure	0.205	0.325	0.4	0.5	0.7	0.8	0.9	0.93	0.95
	Interior Elevation (ft)	8.12	8.12	8.5	9.5	9.8	10.6	11.07	11.33	11.52
	Combined Annual Prob. of Flooding	21%	16%	8%	5%	2.8%	1.6%	0.9%	0.37%	0.19%
2047	Exterior Elevation (ft)	8.89	9.72	10.18	10.46	10.79	11.02	11.23	11.49	11.68
	Prob. of Levee Failure	0.22	0.355	0.45	0.6	0.75	0.9	0.92	0.94	0.96
	Interior Elevation (ft)	8.28	8.28	9.45	9.65	10.4	11.02	11.23	11.49	11.68
	Combined Annual Prob. of Flooding	22%	18%	9%	6%	3%	1.8%	0.92%	0.38%	0.19%
2057	Exterior Elevation (ft)	9.06	9.89	10.35	10.63	10.96	11.19	11.40	11.66	11.85
	Prob. of Levee Failure	0.25	0.37	0.55	0.7	0.85	0.91	0.93	0.96	0.98
	Interior Elevation (ft)	8.45	8.45	9.78	10.49	10.96	11.19	11.4	11.66	11.85
	Combined Annual Prob. of Flooding	25%	19%	11%	7%	3.4%	1.8%	0.93%	0.38%	0.20%
2067	Exterior Elevation (ft)	9.26	10.09	10.55	10.83	11.16	11.39	11.60	11.86	12.05
	Prob. of Levee Failure	0.28	0.4	0.65	0.8	0.91	0.93	0.95	0.98	1
	Interior Elevation (ft)	8.65	8.65	10.55	10.83	11.16	11.39	11.6	11.86	12.05
	Combined Annual Prob. of Flooding	28%	20%	13%	8%	3.6%	1.9%	0.95%	0.39%	0.20%

A 1% ACE Flood Event at 2017 is a 16% Event by 2067

2.7 INTERIM RESULT 2: EXISTING DIKE-POND SYSTEM PERFORMANCE

The HEC-FDA program produces "performance statistics" that are an indicator of the likelihood of damaging flood events under both the future without- and with-project conditions. When levees (or dikes) are present that have some likelihood of geotechnical failure (as is the case under the without-project condition), the project performance is computed based on the joint probability of annual exceedance and probability of geotechnical failure. Table 4 below shows the performance results for the existing dike-pond system in the year 2017, which is the project's base year. The annual exceedance probability is the likelihood that a damaging flood event will occur in any given year, the long-term risk is the risk of a damaging event over some defined period of time for a particular water surface profile, and the conditional non-exceedance probability is the likelihood that the damages would not occur as the result of a particular exceedance probability event. According to the HEC-FDA model, beginning in 2017 there is 32% chance of a damaging flood event in any given year. Figure 11 shows how likely it is to have one or more damaging flood events over different periods of time.

Table 4: Performance Statistics for Existing Dike-Pond System at 2017

Stream Name	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
		Median	Expected	10	30	50	10%	4%	2%	1%	.4%	.2%
SSFS - Alviso	levee	0.3203	0.3211	0.9792	1.0000	1.0000	0.5272	0.3810	0.3528	0.2954	0.1950	0.1331

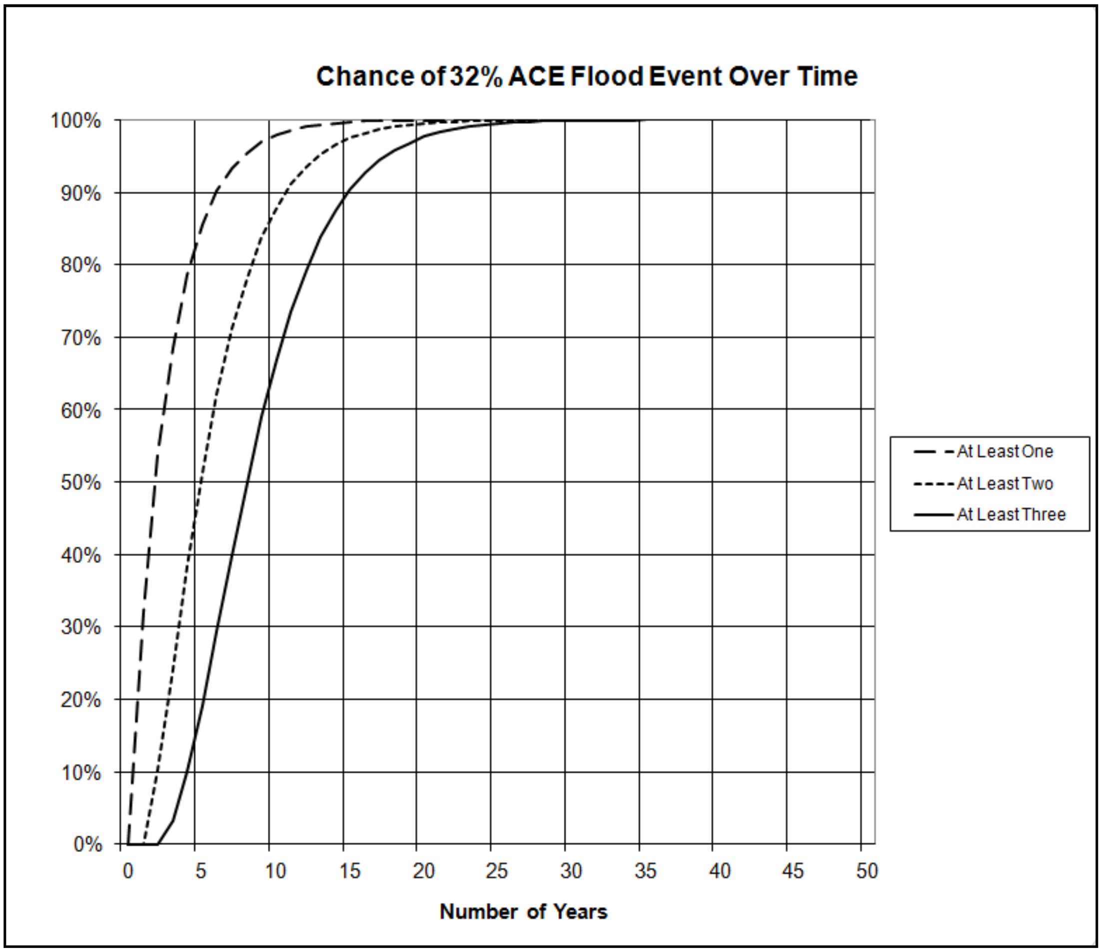


Figure 11: Binomial Distribution of Multiple Flood Events over Time Beginning 2017

The without-project performance of the dike-pond system changes over time with SLC, and the performance varies by scenario. Table 5 below shows the performance statistics at the end of the period of analysis (2067) under the USACE Intermediate SLC scenario. Under any of the future scenarios considered, the risk increases in the future. Table 5 shows that, according to the flood damage analysis, by 2067 the annual likelihood of a damaging flood event is essentially a coin flip, and over a ten-year period the chance of a damaging flood event is a virtual certainty (i.e., long-term risk over a ten-year period is 0.9995).

Table 5: Performance Statistics for Existing Dike-Pond System at 2067 (Intermediate SLC Scenario)

		Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
Stream Name	Target Stage	Median	Expected	10	30	50	10%	4%	2%	1%	.4%	.2%
SSFS - Alviso	levee	0.5339	0.5325	0.9995	1.0000	1.0000	0.2960	0.1283	0.0734	0.0541	0.0363	0.0246

2.8 ECONOMIC DAMAGES & BENEFITS SUMMARY

As described in more detail in the Economics Technical Summary Section 5.0 of this report, the future without-project damages in the study area are estimated to be significant. Because of the low elevation, essentially any flood event in the developed area would be expected to cause millions of dollars in damage to homes, businesses, and infrastructure. Because some flood events could result in several or more feet of water in the community, human health and safety are also at risk from a coastal flood event. As Tables 3 and 4 above show, the likelihood of future flooding in the absence of a project is high.

The USACE typically reports economic flood damage in "expected annual" or "equivalent annual" terms. This is done because of the probabilistic nature of flooding. The average annual damage estimates should not be interpreted as actual damages expected in a given year, but the annual damage if the total flood damage over a very long time horizon were averaged to an annual value. The without-project expected annual flood damage between 2017 and 2067 is between $10M and $28M, depending on the year. The flood risk increases over time due to sea level change, and any structure relocations would decrease the consequences of future flood events in the area. The total equivalent annual damage over the fifty-year period of analysis under the USACE Low, Intermediate, and High SLC scenarios is $18.2M, $22.6M, and $40.2M, respectively. These values include the consideration of the cost of possible structure relocations over time. Under any of the scenarios, a large flood event could cause more than $100M in damage.

The with-project analysis of the different levee heights shows that levees higher than 12' reduce nearly all of the expected future flood damage through the fifty-year period of analysis, and thus greatly reduce the risk to the community from flooding over that time. The levee height with the greatest net benefits (difference between benefits and costs) differs depending on the SLC scenario. A 12.5' levee has the greatest net benefits under the USACE Low and Intermediate scenarios, while a 13.5' levee has the greatest net benefits under the USACE High scenario. The larger 13.5' levee height has the lowest overall residual flood risk, effectively eliminating expected annual damage for any of the three SLC scenarios. With a 12.5' levee in place, under the USACE High SLC scenario the HEC-FDA model results indicate that there would be approximately $1.5M in expected annual damage, which is equivalent to about 4% of the total future without-project annual damage expected under the High SLC scenario. Both the 12.5' and 13.5' levee are strongly economically justified under each of the three SLC scenarios. The benefit-cost ratios range from between about 6 and 12, depending on the levee height and SLC scenario.

According to the HEC-FDA modeling results, with either of the levee heights in place the probability of a damaging coastal flood event in 2017 is extremely low. As sea level rises over time, the likelihood of a damaging event will increase. Just considering the probabilities associated with storm-generated water surface elevations and the project levee elevations, under the USACE Low and Intermediate SLC scenarios each of the levees has a greater than 99% chance of containing a 1% annual chance of exceedance coastal storm event in the year 2067. However, under the USACE High SLC scenario, the 12.5' levee only has about a 1% chance of containing the 1% annual chance exceedance storm event in 2067, while the 13.5' levee has an 88% chance of containing that same storm event. In the year 2067 there is an 8% annual chance of a damaging flood event with 12.5' levee in place, and less than a 1% annual chance with the 13.5' levee. Thus, the 13.5' levee provides a much greater level of performance through the entire fifty-year period of analysis compared to the lower 12.5' levee. More details on the damages and benefits can be found in the Economics Technical Summary.

3.0 COASTAL ENGINEERING TECHNICAL SUMMARY

3.1 SEA LEVEL CHANGE AND TIDES

3.1.1 SEA LEVEL CHANGE PROJECTIONS

Projections developed for this SSFBS tidal flood risk analysis summary report are based on procedures prescribed by ER 1100-2-8162. The geographically closest, suitable NOAA tide gage to the project area is the San Francisco, CA, NOAA tide gage, Station ID: 9414290 (Figure 12). The San Francisco tide gage has a long record length (110 years) and has been referenced to NAVD88. Sea Level Rise projections for the project area in South San Francisco Bay will use the current Relative Sea Level Rise (RSLR) rate for the San Francisco tide gage, 2.06 mm/year, based on 1983-2001 National Tidal Datum Epoch (NTDE). The NOAA tide gage at Coyote Creek, CA, Station ID: 9414575, is located within 5 miles of the project are and has been intermittently operated to collect observed data. The gage does have an established tidal datums based on the last NTDE, and has predicted tide data available.

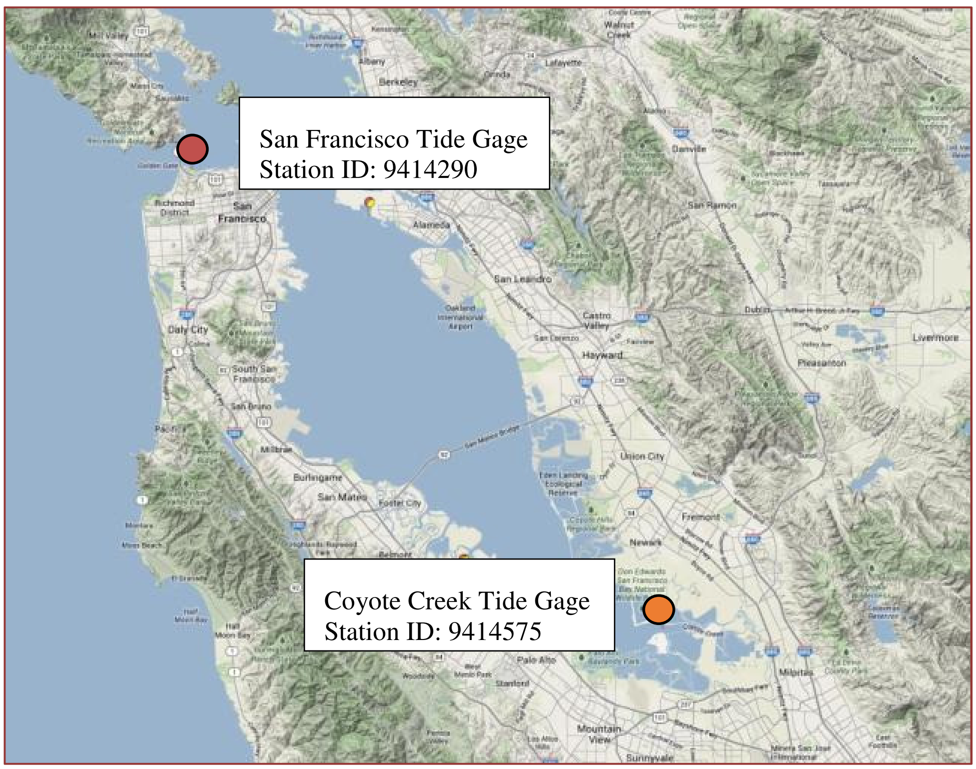


Figure 12: Vicinity Map showing location of Tide Gages used in SSFBS feasibility study.

The planning, design, and construction of a large water resources infrastructure project can take decades. Though initially justified over a 50-year economic period of analysis, USACE projects can remain in service much longer. The climate for which the project was designed can change over the full lifetime of a project to the extent that stability, maintenance, and operation may be impacted, possibly with serious consequences, but also potentially with beneficial consequences. Given these factors, the project planning horizon (not to be confused with the economic period of analysis) should be 100 years, consistent with ER 1110-2-8159 (USACE, 1997).

Water level changes have been developed for the end of the SSFBS 50-year and 100-year economic and planning analysis periods using the current (RSLR) for the San Francisco NOAA tide gage, 2.06 mm/yr (Table 6). Projections made to the year 2100.

Table 6: 50 Year RSLR Low, Intermediate, and High Estimates for SSFBS Economic and Planning Analysis Period

South San Francisco Bay	**2017-2067 Change (ft.)**		
Scenario	Low	Intermediate	High
Coyote Creek Tide Gage /Alviso	0.51	1.01	2.59
	2017 – 2100 Change (ft.)		
Coyote Creek Tide Gage /Alviso	0.73	1.77	5.05

3.1.2 TIDAL DATUM

A temporary NOAA tide gage was deployed at Coyote Creek, Station ID 9414575. Water surface measurements archived between March and August of 2011 were used to update the tidal datum. The MLLW datum plane for the Coyote Creek tide gage was referenced to NAVD88, with some uncertainty due to difficulty in obtaining low water readings from the water level gages surveyed (Table 7).

Table 7: Coyote Creek Tidal Datums (Based on NTDE 1983-2001)

Coyote Creek, CA, Station ID: 9414575		
Tidal Datum	**(feet above NAVD88)**	**(feet above MLLW)**
Mean Higher-High Water	7.64	8.99
Mean High Water	6.99	8.33
Mean Tide Level	3.48	4.82
NAVD88	0.00	1.35
Mean Low Water	-0.07	1.28
Mean Lower-Low Water	-1.35	0.00

The uncertainty in water surface flood elevations due to the Coyote Creek tidal datum conversion to NAVD88 has been recognized and accounted for in the water surface elevations developed for existing conditions. The project vertical datum must be the latest vertical reference frame of the National Spatial Reference System, currently NAVD88, to be held as constant for tide station comparisons, and a project datum diagram (Figure 13) must be prepared per EM 1110-2-6056 (USACE, 2010).

Tidal datums are used throughout all USACE coastal areas and are based on long-term water level averages of a phase of the tide. Mean Sea Level (MSL) datum (or more precisely Local Mean Sea Level--LMSL) is commonly used as a base reference for hydrodynamic modeling, wind and wave surge modeling, high water mark observations, stillwater surge elevations, and design of coastal storm protection structure elevations. The hydraulic/tidal and geodetic vertical datum relationships must be assessed, developed and/or verified during the Feasibility and Preconstruction Engineering and Design (PED) phases, during construction, and periodically monitored after construction to account for subsidence, settlement, NOAA reference datum redefinitions and readjustments, SLC, and other factors. The Coyote Creek tide gage datum adjustment to NAVD88 will be reassessed in the PED phase, and adjustments will be made to design and other key information accordingly.

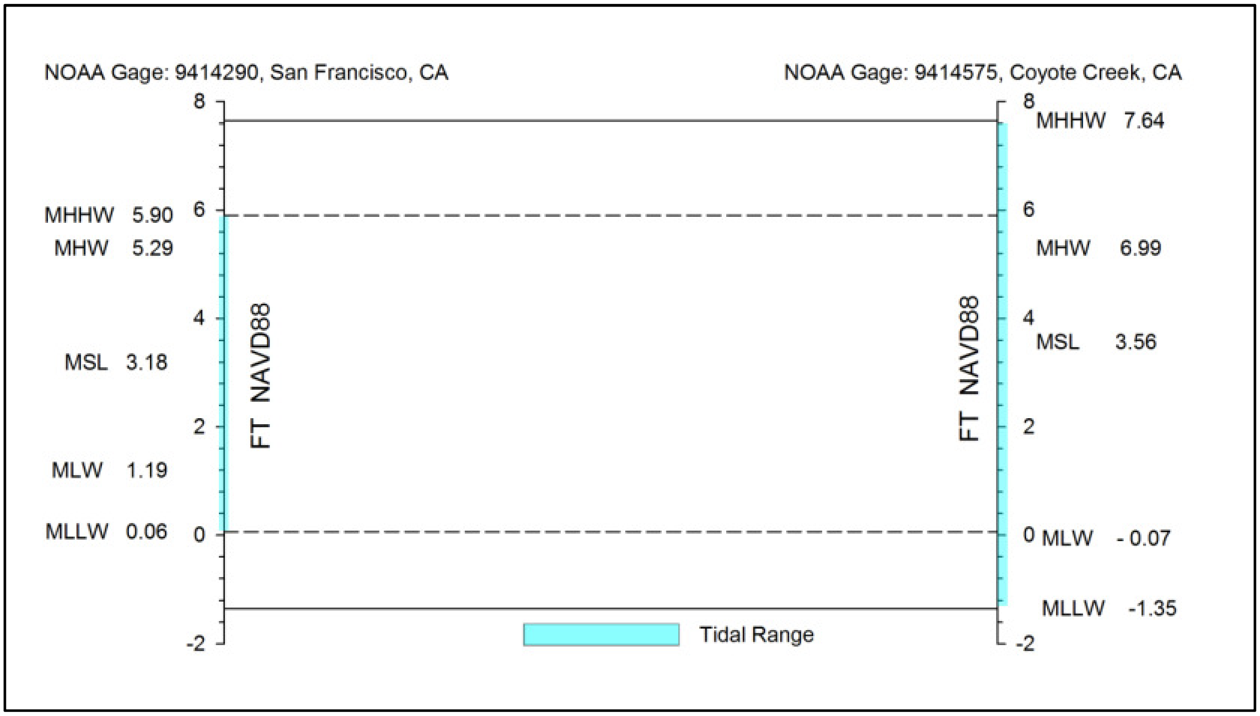


Figure 13: Project Datum Diagram, SSFBS, San Francisco and Coyote Creek Tide Gages.

3.1.3 TIDAL HYDRODYNAMICS AND VARIABILITY IN SAN FRANCISCO BAY

Tides and tide ranges are highly variable through the length of San Francisco Bay. The South Bay area has elevated tides relative to the Pacific Ocean and the rest of San Francisco Bay. The maximum tide levels generally increase with distance southward. As the tides propagate from the Pacific Ocean into 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, Cheng, & Conomos, 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 as shown in Figure 14. The tide range increases from 5.84 feet at the San Francisco tide gage to 9.01 feet at the project area tide gage, Coyote Creek.

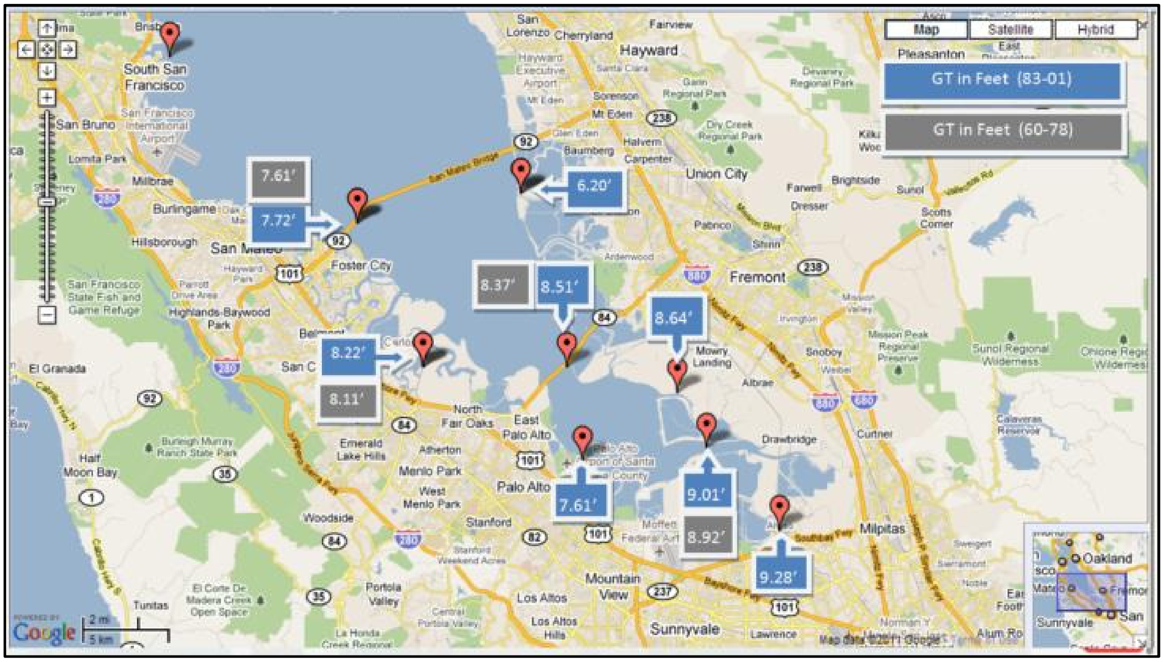


Figure 14: Tidal Ranges in South San Francisco Bay, Last Two Complete NTDE.

3.2 EXTREME WATER LEVEL STATISTICS IN PROJECT AREA, EXISTING AND WITHOUT-PROJECT CONDITIONS

3.2.1 METHODOLOGY

Extreme water statistics representative of coastal flood risk from high water levels in the South Bay area near the community of Alviso were developed by computing the tidal amplification factor between the predicted (astronomical) tide at the San Francisco tide gage and the Coyote Creek tide gage. Tidal residuals (observed – predicted tide) represent storm surge, and are assumed to transfer directly to the South Bay. This method is referred to as the Direct Transfer Method (DTM).

3.2.2 DIRECT TRANSFER METHOD

Factors used to amplify the predicted tide at San Francisco are assumed to be linear and were computed by comparing predicted tide at the San Francisco tide station to predicted tide at Coyote Creek (Figure 15). The comparison indicated tidal amplification at Coyote Creek varied with predicted tide water surface elevation at the San Francisco tide station. Four amplification factors were developed to account for the range of predicted tides, with a focus on the daily higher-high tide. Comparison of the derived water levels at Coyote Creek and predicted daily higher-high tides showed good agreement at Coyote Creek.

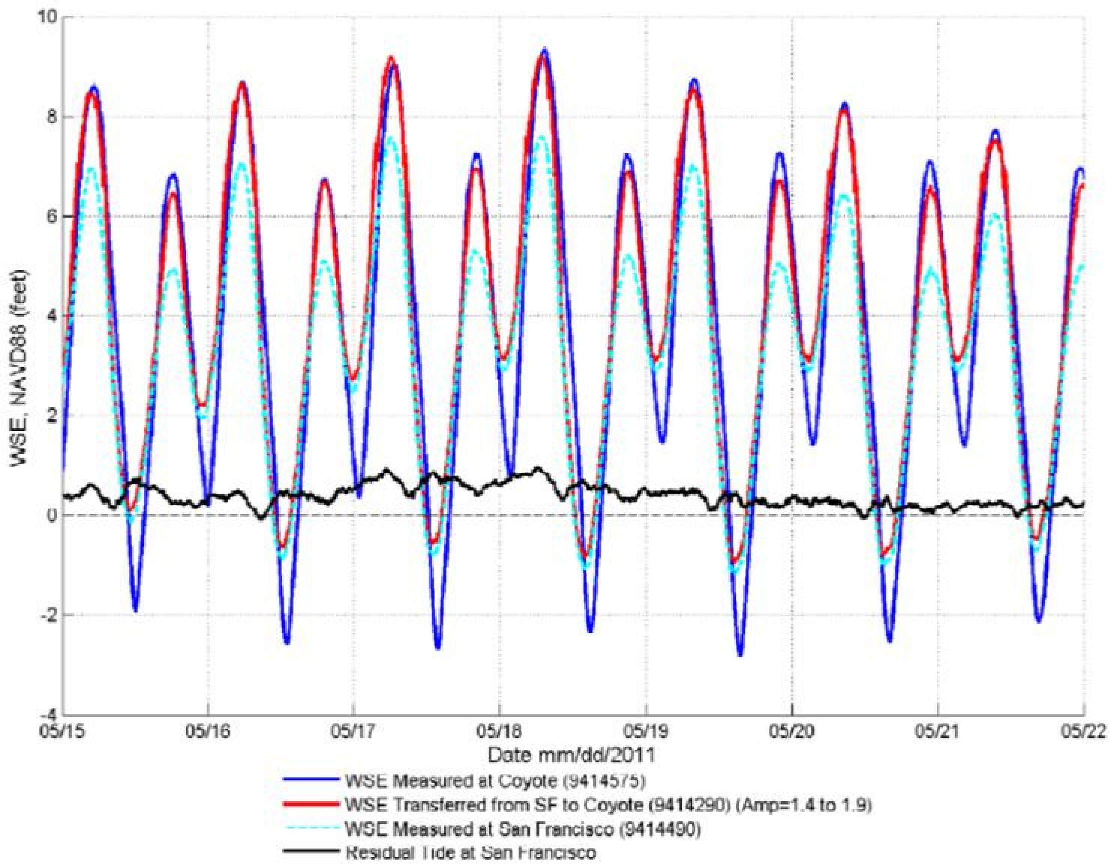


Figure 15: Comparison of Amplified Tides at San Francisco and Measured Tides at Coyote Creek.

Table 8 shows the amplification factors used in the DTM, based on the San Francisco tide gage MLLW tidal datum. The DTM is an appropriate surrogate method for developing accurate water levels and developing extreme water level statistics in areas where local mean sea level and tidal datum have been defined. Multidimensional hydrodynamic modeling may add precision, and is appropriate when decoupling of the tidal residual addition to the tide into components; wind, wave run-up, and surge is desired to aid design of coastal structures.

Table 8: Tidal Amplification Factor - San Francisco to Coyote Creek

Direct Transfer Method - Amplification Factor (San Francisco to Coyote Creek)	
Predicted Tide Range at San Francisco (feet MLLW)	**Amplification Factor at Coyote Creek**
Less Than 4.94	1.9
4.94 to 5.52	1.6
5.52 to 6.15	1.5
Greater Than 6.15	1.4

Amplification factors developed with the DTM were applied to extreme water statistics developed for the San Francisco tide gage and used to derive the extreme water statistics for the Coyote Creek tide gage, representing the study area's existing risk from coastal flooding.

3.2.3 EXTREME WATER LEVEL STATISTICS, SAN FRANCISCO TIDE GAGE

Extreme Water Level Statistics were developed for the San Francisco tide gage. Tide gage monthly highs or monthly extreme high water (MEHW) from 1 January 1901 through 31 December 2011 were converted to NAVD88. The bias due to RSLR was removed by detrending the data to the mid-year of the last complete NTDE (1983-2001), 1992. Figure 16 compares the detrended data (blue) with the biased data (red).

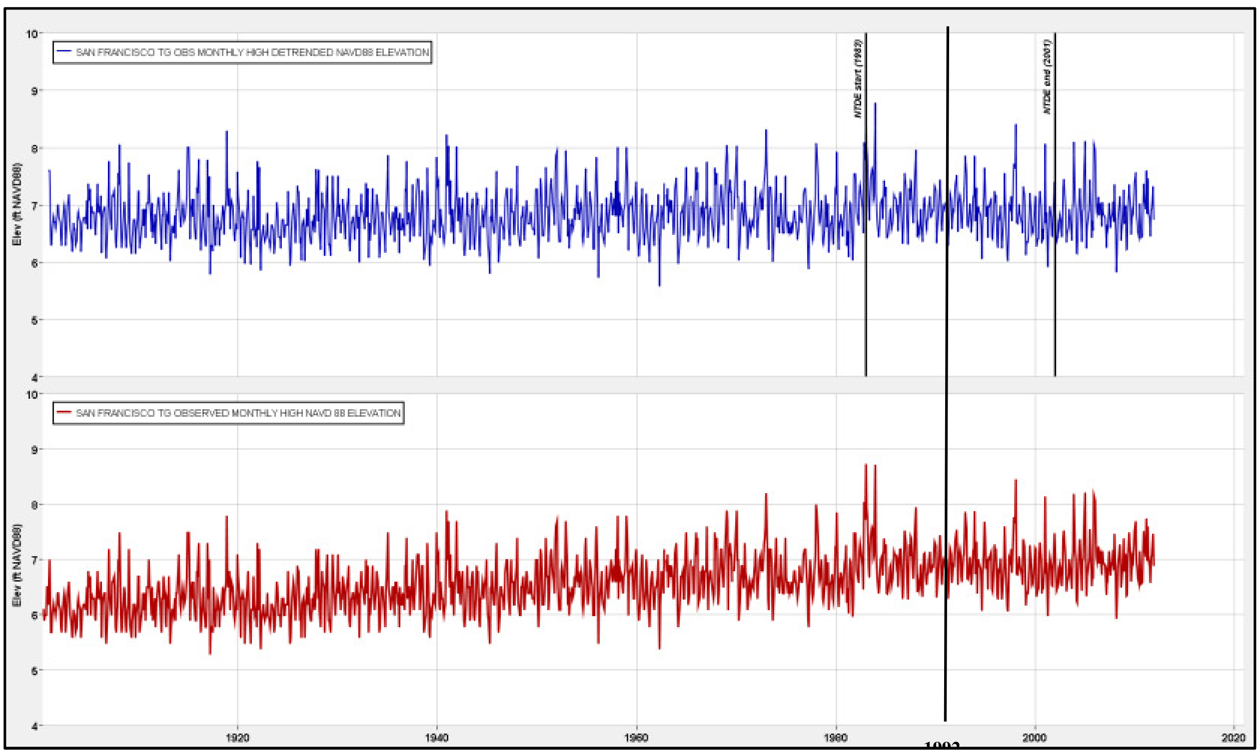


Figure 16: Observed Monthly MEHW levels, San Francisco Tide Gage (1901-2011).

Detrending the MEHW tide gage data creates a homogenous data set with regard to relative sea level rise. The year 1992 is used in the USACE SLC scenario equations, and is a base from which extreme water level statistics developed from the MEHW detrended data are projected to the project base year and future years by applying the 3 SLC rates related to the San Francisco tide gage.

The MEHW detrended time series was sampled to create an annual extreme high water level (AEHW) time series, for use in developing ACE statistics representing extreme water levels (Figure 17). The peak water level of record occurred January 23, 1983, while the second-highest water level occurred eleven months later on December 3, 1983. The annual series was adjusted slightly by moving the December 1983 high water level into the 1984 monthly series and recognizing it as the 1984 annual peak; had a water year division been used to develop the annual series, this adjustment would not have been necessary.

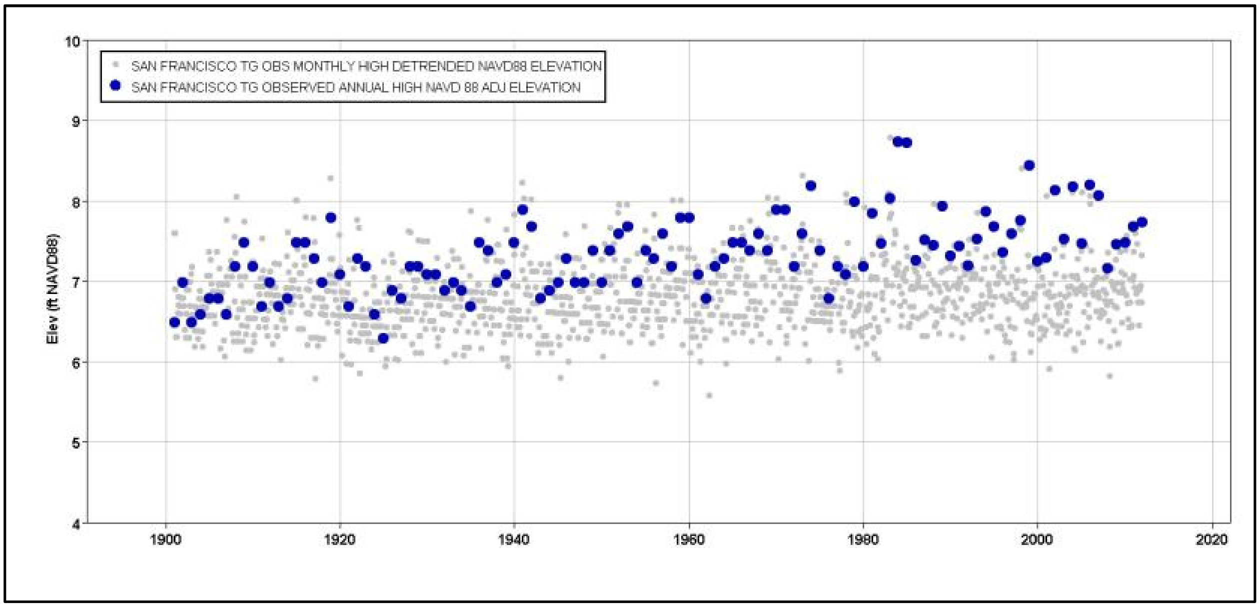


Figure 17: San Francisco Tide Gage AEHW Data (Blue) Developed from MEHW Data (Gray).

The 110 year AEHW annual series for the San Francisco tide gage was fit to a generalized extreme value distribution (GEV). The GEV distribution is a three parameter distribution (Table 9). The GEV probability distribution functions are defined by a location parameter (mean), a scale parameter (variance), and a shape parameter. If the shape parameter is zero, the distribution is known as a Gumbel distribution. If the shape parameter is positive, the distribution is called a Frechet distribution; if the shape parameter is negative, the distribution is called the Weibull distribution. The Frechet distribution has a thicker positive tail indicating a higher probability of extreme positive outliers. In contrast, the Weibull distribution actually goes to zero above some limiting positive value (Zervas C. E., 2005). Table 9 presents the GEV parameters from the 110 year annual series data.

Table 9: GEV Distribution Parameters for San Francisco Tide Gage Adjusted AEHW Record (1901-2011)

Parameter	Estimate	Standard Error
ξ (shape)	-0.091	0.06
σ (scale)	0.32	0.023
μ (location)	7.42	0.34

NOAA has used the GEV distribution to compute extreme water level statistics for 117 NOAA/National Ocean Service water level stations. The statistics show regional trends, and when the shape parameters are negative (Weibull distribution); there are relatively small differences in the levels of the four or five most extreme events. However, when the shape parameters are positive (Frechet distribution), there can be large differences in the levels of the top four or five extreme events. For the larger NOAA data set, the shape factors were negative for almost all of the high extreme levels at the Pacific Coast, Alaskan, and

Pacific Island stations. In contrast, most of the GEV shape parameters for high extreme levels are positive (Frechet distribution) ranging from 0 to 0.5 at the Atlantic Coast, Gulf of Mexico, and Atlantic Island stations. This is usually due to the interaction of a few powerful hurricanes with a wide, shallow, continental shelf at these stations, resulting in a handful of extreme values significantly higher than the levels of the most powerful winter storms (Zervas C. E., 2005). Figure 18 shows the relative uncertainty of the 1% annual chance probability water level versus shape parameter; for the San Francisco tide gage, this value corresponds roughly to 0.28 m or 0.9 feet. The record at San Francisco is very long, which reduces the confidence interval, however there is significant inter-annual variation observed primarily due to the effects of El Niño–Southern Oscillation (ENSO).

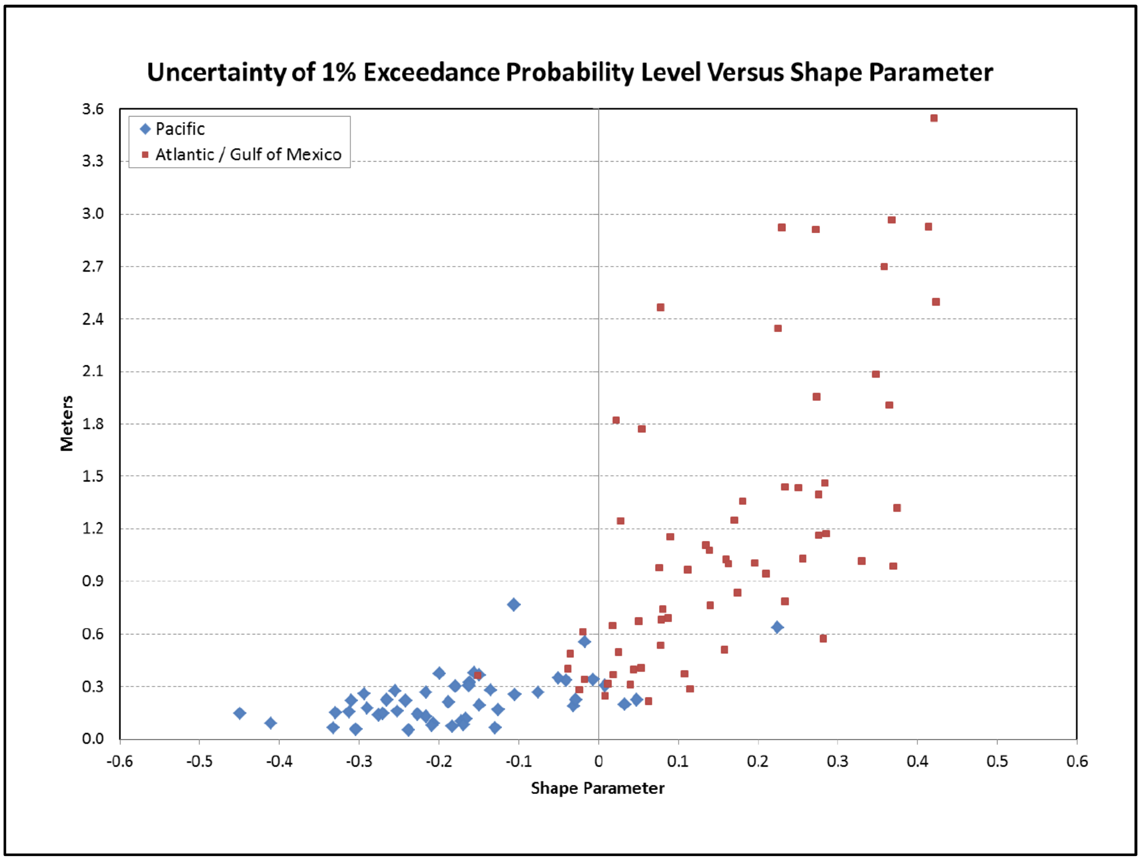


Figure 18: GEV Shape Factors for 117 NOAA Stations showing uncertainty and regional trends [adapted from (Zervas & Sweet, 2014)]

The GEV expected probability function was used to compute annual percent chance exceedance water level statistics for the San Francisco tide gage, which would be transferred to the Coyote Creek gage using the DTM (Figure 19). Statistics developed with the detrended data represent the midpoint year of 1992, the last complete NTDE. The statistics are then adjusted to the current year, project base year, or future years using the USACE SLC scenario equations and RSLR rate from the San Francisco tide gage as defined in EC 1165-2-212 and ER 1100-2-8162. Table 10 shows the annual percent chance exceedance water level statistics computed for the San Francisco tide gage. The low or observed RSLR (2.06 mm/year) was added to the 1992 statistics to project the base year (2017) conditions.

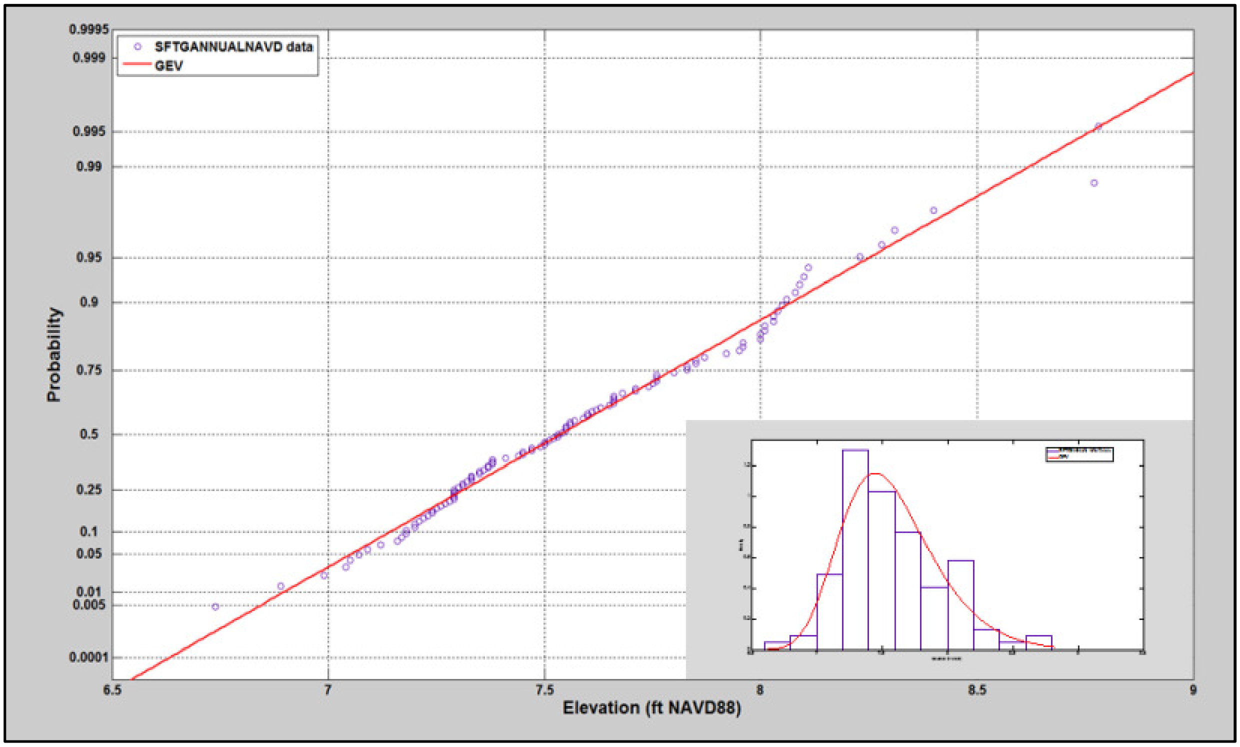


Figure 19: Annual Series of Peak Water Levels Fit to GEV, 1901-2011.

Table 10: Annual Chance of Exceedance (ACE) Water Levels, San Francisco Tide Gage 1992, 2017

	1992	**1992**	**RSLR Low Rate 1992-2017**	**2017**
FREQ (%)	feet MLLW	feet NAVD88	feet	feet NAVD88
99.99	6.89	6.95	0.17	7.12
50	7.48	7.54	0.17	7.71
20	7.81	7.87	0.17	8.04
10	8.01	8.07	0.17	8.24
4	8.25	8.31	0.17	8.48
2	8.41	8.47	0.17	8.64
1	8.56	8.62	0.17	8.79
0.4	8.75	8.81	0.17	8.98
0.2	8.88	8.94	0.17	9.11

3.2.4 COYOTE CREEK EXTREME WATER LEVEL STATISTICS FROM DTM

The DTM separates predicted tide and residual tide, amplifying predicted tide by an amplification factor, of 1.4 to 1.9 (Table 8), and adding the residual tide back to the amplified tide and adjusting for the local Coyote Creek datum.

Hydrodynamic model simulations were conducted to evaluate the change in residual tide recorded at the San Francisco tide station as it propagates into South San Francisco Bay. The simulation indicates that residual tide varied minimally (MacWilliams, Kilham, & Bever, 2012). This implies that it is a reasonable assumption that residual tide at San Francisco is additive to the amplified predicted tide when transferred to Coyote Creek. These results confirm the DTM assumption that only the predicted tide is amplified, with the residual tide remaining constant. The DTM equations are;

$$MT_{CC} = PT_{CC} + RT_{SF} \qquad \text{Equation 1.1}$$

$$PT_{CC} = (PT_{SF} - MTL_{SF})\ x\ A + MTL_{CC} \qquad \text{Equation 1.2}$$

$$RT_{SF} = MT_{SF} - PT_{SF} \qquad \text{Equation 1.3}$$

where:

MT_{CC} = Estimated Measured WSE at Coyote Creek (NAVD88)

RTSF = Residual Tide at San Francisco

PTCC = Predicted Tide at Coyote Creek

PTSF = Predicted Tide at San Francisco

MTLSF = Mean Tide Level at San Francisco (3.24’, MLLW)

A = Amplification Factor, Table 3

MTLCC = Mean Tide Level at Coyote Creek (3.48’, NAVD88)

MTSF = Measured WSE at San Francisco (MLLW)

The tidal residual component contained in the extreme water level statistic represents what is commonly referred to as storm surge. Storm surge refers to the increased elevation of water levels due to meteorological conditions such as increase in water elevation due to low barometric pressure and wave setup to a limited extent. The ACE water levels are comparable to FEMA still-water surface elevations and base flood elevations.

The most likely or 50% ACE for Coyote Creek was computed by using the 50% residual tide statistic developed from analysis of 47 historical storm events, with residuals greater than 0.5 ft. Figure 20 shows the cumulative distribution function (CDF) of the tidal residuals from the San Francisco tide gage. The 50% residual value of 0.85 feet, 5 and 95 percent values (1.55 and 0.55 feet) respectively were selected from the CDF.

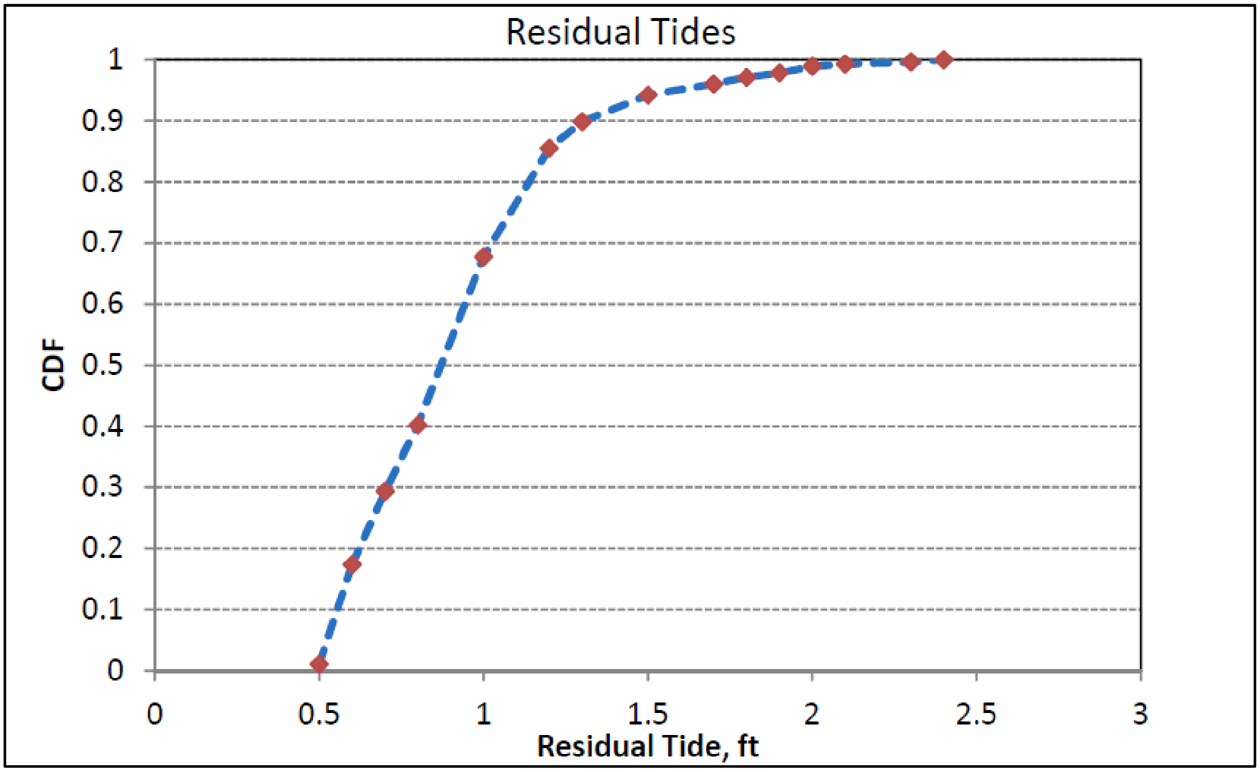


Figure 20: Cumulative Distribution Function for Tidal Residuals in feet developed from 47 Historical Storm Events at the San Francisco Tide Gage

The DTM was applied to the 1992 San Francisco ACE results in Table 11to produce the derived Coyote Creek 50% ACE (Table 11). Apparent RSLR was recognized from 1992 to 2017 in the amount of 0.17 feet based on the rate at the San Francisco tide gage (2.06 mm/year) and added to the 1992 detrended statistics to arrive at the 2017 existing and without-project conditions in South San Francisco Bay at the Coyote Creek tide gage.

Table 11: ACE Water Levels for San Francisco and Coyote Creek Tide Gages, 1992 and 2017

	San Francisco Tide Gage Station ID: 9414290		**Coyote Creek Tide Gage Station ID: 9414575**	
FREQ (%)	**1992**	**2017**	**1992**	**2017**
	feet MLLW	**feet NAVD88**	**feet NAVD88**	**feet NAVD88**
99.99	6.89	7.12	8.25	8.42
50	7.48	7.71	9.08	9.25
20	7.81	8.04	9.54	9.71
10	8.01	8.24	9.82	9.99
4	8.25	8.48	10.15	10.32
2	8.41	8.64	10.38	10.55
1	8.56	8.79	10.59	10.76
0.4	8.75	8.98	10.85	11.02
0.2	8.88	9.11	11.04	11.21

Note: San Francisco based on gage record of 110 years, Coyote Creek derived from San Francisco using DTM and 50% tidal residual value.

As the relative components of the extreme water levels are primarily tidal with small residuals, 1% exceedance levels can be reached only by a combination of a storm, a spring tide and an El Nino event (Zervas & Sweet, 2014). These thresholds will be easier to reach and more frequent under future sea level change scenarios. The 0.4 and 0.2 ACE values at both stations are expected values from the GEV distribution. With RSLR increasing the base water levels, the ENSO impacts are more likely to push extreme water levels into the higher ACE values under the same conditions.

3.2.5 COMPARISON OF 1 PERCENT ACE WATER LEVEL WITH PRIOR STUDIES.

The 1% ACE or 100-year values for San Francisco are compared with results from other studies in Table 12.

Table 12: Comparison of 1% ACE Water Levels for San Francisco and Coyote Creek Tide Gages to Prior Studies

	USACE (2014)[1]	(USACE, 1984)[2]	(Knuuti, 1995)[3]	(PWA, 2007)[4]
San Francisco Gage	8.79	8.69	8.89	8.72
Coyote Creek Gage	10.76	10.99	-	11.02

[1] Value represents record (1901-2011), detrended to 1992, projection to 2017
[2] Value represents record (1855-1983), adjustment of 0.53 ft. to the mean
[3] Value represents record (1897-1995), projection to 2000, detrended to 2000
[4] Value represents record (1897-2004), detrended to 2005

Variation in the 1% ACE water levels may be attributed to many factors, such as methodology, record length and statistical methods. Accounting for these differences, the results are very consistent. The USACE 2014 water level, representing the results of the current analysis, is based on an additional 7 to 31 years of data at the San Francisco tide gage. Interannual variations primarily due to ENSO may influence statistics if an extreme is appended to the end of the record. Apparent SLC rates have been lower in the recent 5 to 10 years due to a neutral ENSO phase, and will account for some of the difference in the PWA 2007 and USACE 2014 result. Current SLC rates and coefficients used in the other studies have been updated and are reflected in the USACE 2014 result, and account for some of the difference in results. The last reference (PWA, 2007) contains a more in-depth discussion of the methods behind the other the other results cited.

3.2.6 NATURAL VARIABILITY, UNCERTAINTY IN COYOTE CREEK EXTREME WATER LEVEL STATISTICS

ACE statistics presented in Table 12 represent the most likely or 50% occurrence. The bulk of natural variability is captured in the CDF of tidal residuals (Figure 20). The 5 and 95 percent ACE water surface elevation estimates were computed using the DTM function and assume tidal residuals of 1.55 and 0.55 respectively. In the DTM formula, the residual is not amplified so the result is that the higher residual (1.55 feet) is used to compute the lower 5 percent and the lower residual (0.55 feet) is used to compute the upper 95 percent confidence interval (Table 13). The higher number is achieved due to a larger component of the tide is predicted or astronomical and thus subject to the amplification factor. The natural variability assumptions and computation are recognized to be a simplifying, coarse assumption, but accurate. Combinations of water level components occurring concurrently such as high astronomical tide, storm surge residual, and extreme wind generated waves are possible, but would occur in the 95 to 99.99 percentile. The confidence interval range of the water surface elevation used in the HEC-FDA model to estimate flood damage is slightly greater than that shown in Table 13. The FDA model uses order statistics to derive the confidence limit when using what is termed the "graphical method." As an example, the difference for the 50% ACE water surface elevation is about .1 feet, and the difference for the .2% ACE elevation is about .5 feet. Because of the small difference for the more likely events, and because the absolute value of the difference is generally symmetrical above and below the mean, this small difference in uncertainty parameters should have very little impact on the overall estimate of flood damage.

Table 13: Coyote Creek Tide Gage 2017

	Coyote Creek Tide Gage Station ID: 9414575		
	2017 (5%)	**2017 (50%)**	**2017(95%)**
FREQ (%)	**feet NAVD88**	**feet NAVD88**	**feet NAVD88**
99.99	8.14	8.42	8.54
50	8.97	9.25	9.37
20	9.43	9.71	9.83
10	9.71	9.99	10.11
4	10.04	10.32	10.44
2	10.27	10.55	10.67
1	10.48	10.76	10.88
0.4	10.74	11.02	11.14
0.2	10.93	11.21	11.33

The El Niño-Southern Oscillation (ENSO) is a quasi-periodic climate pattern that occurs across the tropical Pacific Ocean about every two to seven years. It is characterized by variations in the sea-surface temperature of the tropical eastern Pacific Ocean (NRC, 2012). ENSO is the dominant cause of sea-level variability in the northeast Pacific Ocean on interannual timescales (Zervas C. , 2009). Sea level rises off the west coast of the United States during El Niño events and falls during La Niña events. The highest sea levels recorded along the west coast and at the San Francisco tide gage were associated with El Niño events. On January 27, 1983, during one of the largest El Niños in half a century, seven tide gages along the west coast recorded their highest water levels. This event produced a water level 2.82 feet above Mean Higher High Water (MHHW) at the San Francisco gage. Figure 21 and Figure 22 show the impact of ENSO on relative sea levels (NRC, 2012).

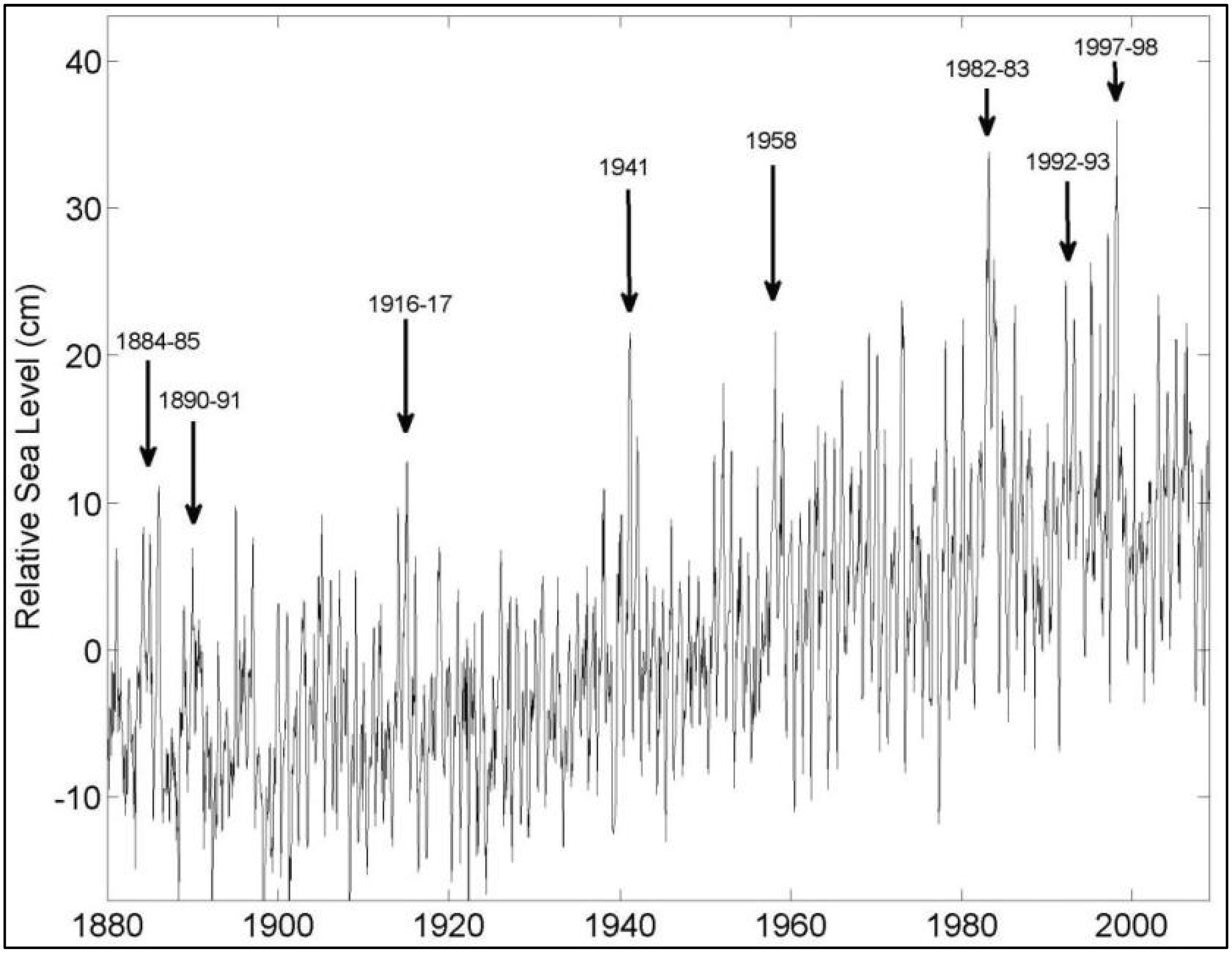


Figure 21: San Francisco Tide Gage Record Showing Relative Sea Level Rise Increases during Major El Niño Events [From (NRC, 2012)]

Most recent work on the impact of ENSO on west coast sea levels estimate the variability due to ENSO to be in the range of 10 to 30 cm (0.32 to 0.98 feet) , with 20 cm 0.66 feet the consensus. This estimate is visible by examination of

Figure 22, which shows variability of the ENSO pattern imposed on the MEHW by a seven-month moving average shown in red.

Decadal and longer variability in sea level off the United States West Coast often corresponds to forcing by regional and basin scale winds associated with climate patterns such as the Pacific Decadal Oscillation (PDO) (NRC, 2012).

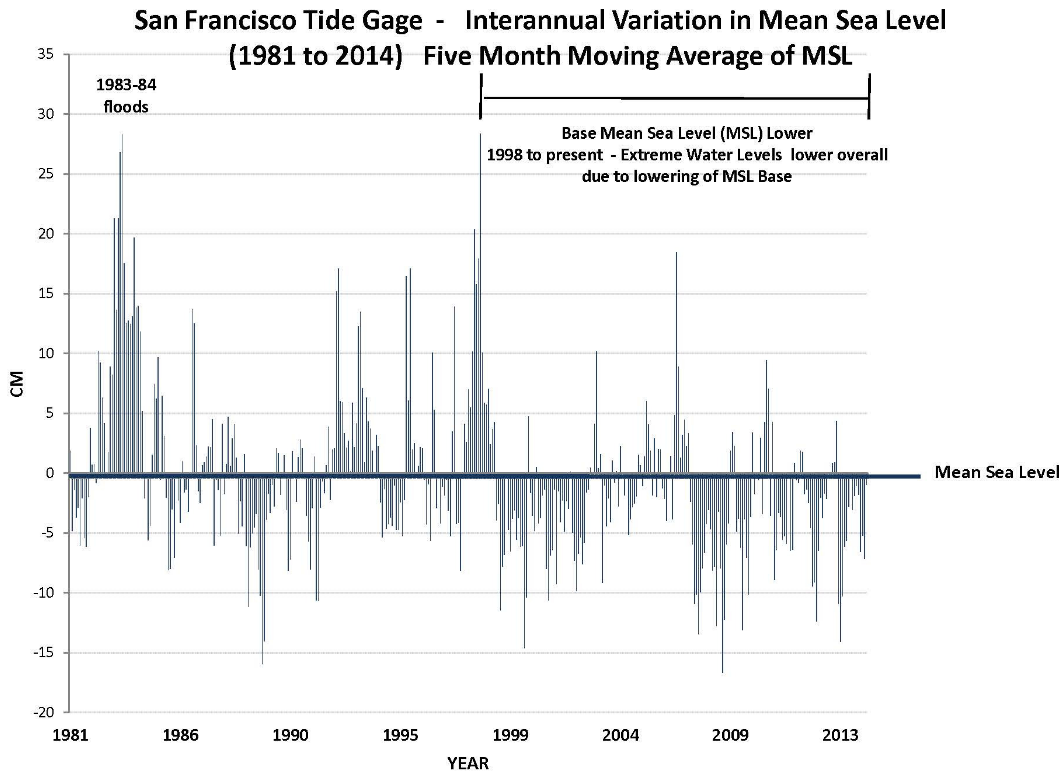


Figure 22: Detrended San Francisco Tide Gage MEHW, Moving Average Showing Range Interannual Variability Due to ENSO.

The daily, monthly and annual tidal cycles account for some of the natural variability in water levels and may contribute to an extreme water level when combined with other contributing factors. The Earth-Moon-Sun orbital geometry results in heightened high tides twice monthly (spring tides, near the times of the full and new moon) and every 4.4 years and 18.6 years (NRC, 2012). The largest tidal amplitudes of the year impacting San Francisco Bay occur in the winter and in summer are often more than 20 cm (0.66 feet) higher than tides in the spring and fall months. The peaks in the 4.4-year and 18.6-year cycles produce monthly high tides that are about 15 cm and 8 cm (0.49 feet and 0.26 feet), respectively, higher than they are in the intervening years (Flick, 2000). Table 14 summaries the various factors impacting extreme water levels.

Table 14: Summary of Extreme Water Level Natural Variability

Variability due to Single Event and Seasonal Climate Trends			Variability due to Tidal Cycles (added to peak)		
	Storm Surge	**ENSO**	**Seasonal**	**1 in 4.4 years**	**1 in 18.6 years**
feet	0.55 – 1.55	0.32 – 0.98	0.66	0.49	0.26
cm	17 – 47	10 – 30	20	15	8
Mean (feet)	0.85	0.66	0.66	0.49	0.26
S (feet)	0.54	0.33			

The water level component variability discussed in this section and summarized in Table 14 is reflected in the overall statistics developed for the San Francisco tide gage and DTM function for Coyote Creek. Uncertainty in the ACE for the Coyote Creek tide gage is estimated by a simple uncertainty model created through estimates of two of the major factors identified in Table 14. The total uncertainty in extreme water levels for the Coyote Creek tide gage is developed using Equation 1-4, adapted from EM 1110-2-1619:

$$S_{Z,total} = \sqrt{S^2_{Z,natural} + S^2_{Z,\mathrm{mod}el} + S^2_{Z,datum}} \qquad \text{Equation 1.4}$$

where

$S_{Z,\ total}$ = total standard deviation of error representing uncertainty in extreme water levels

$S_{Z,\ natural}$ = the standard deviation associated with uncertainty in extreme water levels due to natural variability

$S_{Z,\ model}$ = the standard deviation associated with uncertainty in extreme water levels due to application and assumptions in the Direct Transfer Function (DTM)

$S_{Z,\ datum}$ = the standard deviation associated with uncertainty in extreme water levels due to tidal datum to geodetic datum gage conversion

The factors comprising the total uncertainty (Table 15) are assumed to occur independently of each other, and determine the confidence interval applied to the ACE elevations for Coyote Creek tide gage. The ACE elevations and associated confidence interval represent the coastal elevation-probability function which describes exposure in the economics model, HEC-FDA. The approximate confidence interval estimated by equation 1-4, 0.76 feet, is input as an "equivalent gage record" value in HEC-FDA. The equivalent gage record was estimated by a sensitivity analysis using HEC-SSP software in which gage record lengths in years were input into a graphical frequency analysis model created with the San Francisco tide gage AEHW values and run to produce confidence intervals roughly equivalent to the value developed by equation 1.4 (Deering, 2014), in effect "backing into an equivalent gage value" which approximates the uncertainty estimate developed by equation 1.4. The HEC-SSP sensitivity analysis yielded an equivalent gage value of approximately 35 to 40 years.

Table 15: Uncertainty given by Equation 1.4 to create Confidence Intervals for Coyote Creek Tide Gage ACE Values

	Source/Type of Uncertainty				
	Natural		**Model**	**Datum**	**Total**
	Storm Surge	**ENSO**	**DTM function**	**Datum**	
S (feet)	0.54	0.33	0.33	0.25	
S^2 (feet)^2	0.29	0.11	0.11	0.06	0.57
S (feet)					0.76

3.2.7 ALVISO ECONOMIC IMPACT AREA, EXISTING WITHOUT-PROJECT CONDITION FLOOD RISK

The Alviso EIA identified in the existing without-project condition roughly comprises the community of Alviso, CA. The area has been subjected to high rates of subsidence from groundwater withdrawal for agriculture for the first half of last century, causing lands adjacent to San Francisco Bay to sink 2 to 8 feet by 1969, with 4 to 6 feet occurring in Alviso (USGS). Figure 23 shows the dramatic change in elevation at the South Bay Yacht Club in Alviso last century, with Alviso now several feet below sea level.

Figure 23 -South Bay Yacht Club, Alviso, CA. Top – 1914, Bottom – 1978 (Source: USBS, Santa Clara Valley Water District)

The degree of subsidence locally and across the South Bay effectively rendered the area vulnerable to flooding at high tides. The system of dikes and ponds, which was constructed and operated strictly for the purpose of harvesting salt, does provide incidental tidal flood risk reduction, demonstrated by the fact that there is no history of tidal flooding in Alviso. The existing without-project condition, under which the overall planning effort is being conducted, recognizes and accounts for this performance. An extensive geotechnical investigation of the dike system was conducted to assess current and future risk of flooding though failure or overtopping of the dike systems surrounding the salt ponds. Flood risk to Alviso from riverine flooding and localized rainfall flooding have been mitigated by levees and stormwater drainage systems aided by pumping to offset the loss of elevation from subsidence, which makes gravity drainage to the South Bay ineffective.

Under the existing without-project condition, water levels due to coastal or tidal flood risk for Alviso are defined by several assumptions;

1. Coyote Creek tide gage, ACE base year 2017 water levels represent the flood aspect of the existing dike-pond system

2. Two pathways for flooding in Alviso from outer dike breaching have been identified, as two discreet dike-pond systems exist, separated by Artesian Slough. The eastern path flows through Pond A18 and the western path flows through Ponds A9 through A16. The most likely path under the existing without-project condition is the western path, as dike elevations are lower, and geotechnical risk is higher. Figure 24 shows the assumed flood pathways to Alviso.
3. Hydrologic risk, represented by the Coyote Creek tide gage ACE water levels, is equivalent through both potential flood pathways.
4. The magnitude and breadth of predicted outboard dike breaches increase as the ACE water levels increase.

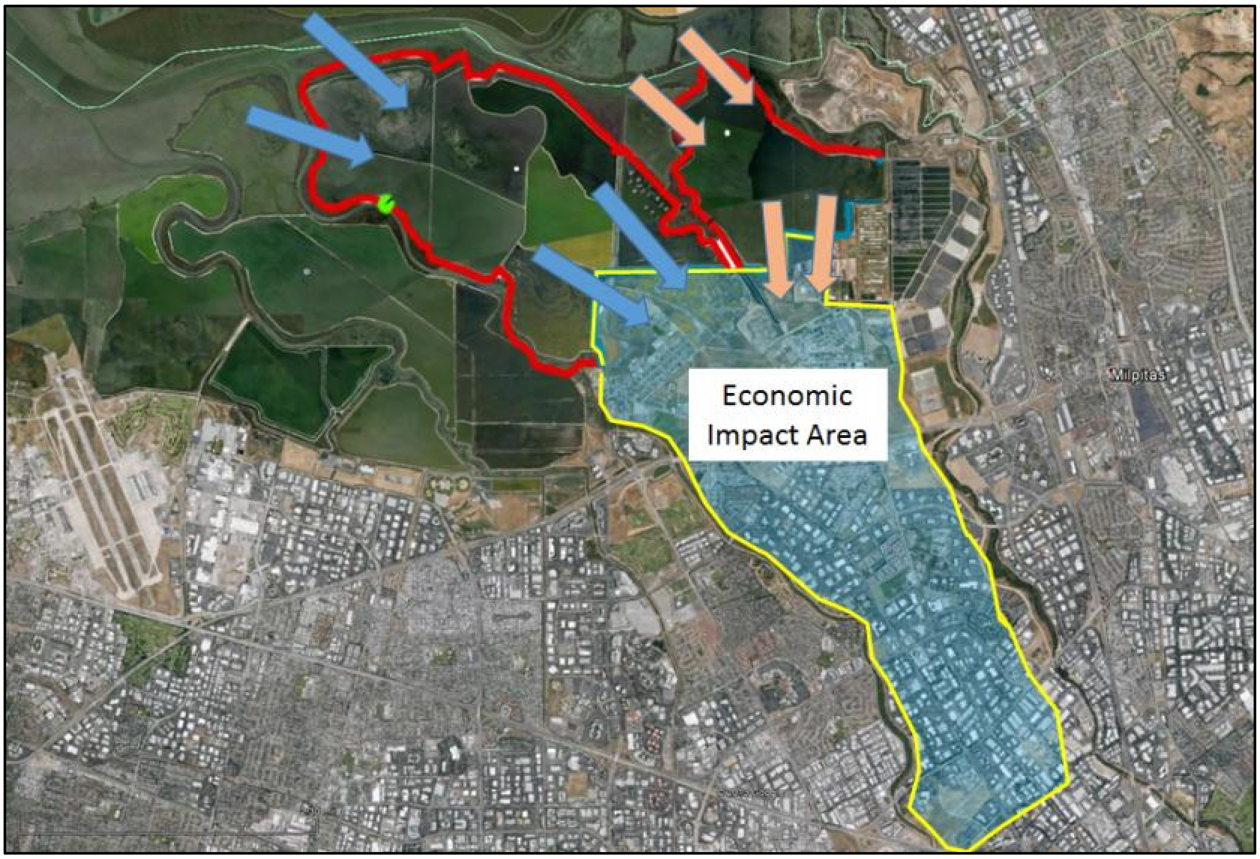


Figure 24 - South San Francisco Bay Area Showing the Outboard Dike System in Red and Potential Tidal Flood Pathways to the Alviso Economic Impact Area

5. The outboard and inboard dikes create a closed system, which is assumed to be at mean sea level. The available pond storage to mitigate initial flood levels is controlled by the average inboard dike elevation minus mean sea level.
6. Coyote Creek tide gage ACE levels are transferred to Alviso through breaching of outboard dikes. An exterior-interior ACE water level relationship was created between Coyote Creek and Alviso, which reflects performance of the dike-pond system resulting in some reduction of

potential flood levels in Alviso. The major controlling factors in the exterior-interior relationship are the duration and elevation of extreme water levels and available pond storage.

7. Performance for the western flood pathway through Ponds A9 through A16 is defined by a curve defining probability of unsatisfactory performance (Pu), also referred to as probability of failure, for a typical outboard dike elevation of 12 feet NAVD88 and represents the controlling geotechnical risk. This elevation represents the predominant elevation for most of the outboard dikes enclosing the pond and would represent a systemic failure resulting in flooding to Alviso immediately or within one or two tidal cycles.
8. Any outboard dike breach will result in an inboard dike breach and flooding to Alviso (see Section 6.3.2).
9. The minimal flood level occurring in Alviso with any outboard breach will be MHHW or highest high tide. This assumption is based on the fact that outboard and inboard breaches created in the flood event will continue to expand during recession of the flood tide, and fill to MHHW on subsequent tides. The water levels in the pond will quickly reach phase and equilibrium with the bay transferring a flood level to MHHW to areas of the Alviso, which are 6 to 8 feet below mean sea level as a result of subsidence from the cycle of groundwater withdrawals last century.
10. Water volume transferred to Alviso though inboard dike breaches will be conveyed to the lowest elevations and remain there. Flooding will progress to higher elevation areas once areas at lower elevations are flooded. Water volume transferred to the EIA though interior levee breaching is assumed to pond and remain until removed by existing drainage system aided by pumping or a targeted dewatering effort.

The topography of the EIA, which was influenced by rapid subsidence last century, may be described by an elevation storage curve. A critical performance element of the dike-pond system providing incidental tidal flood risk reduction for Alviso is the available storage in the ponds, which is defined by a critical elevation for inboard dike failure and the ambient water surface in the pond. While overtopping may start at elevations as low as 6.5 feet, the critical elevation for inboard dike failure is 7.5 feet. The ambient water surface on the pond is mean sea level and changes with time due to sea level rise; at 2017 it is 3.71 feet. The dike-pond system volume differential ameliorates the tidal flood potential in the bay, which is limited in duration as the bulk of the water surface elevation is due to astronomical tide. Performance of the closed system intact provides significant flood risk reduction. The open system performance with predicted failure due to the combination of hydrologic risk and geotechnical risk still provides some flood reduction as defined by the net elevation difference between the exterior and interior ACE water surfaces under the more frequent occurrence intervals.

Elevation-volume curves for the closed ponds bordering the EIA, and the EIA are given in Table 16. Elevation 7.5 feet has been established as a critical elevation for the inboard dikes and geotechnical failure criteria (Table 22) predicts that between 0.75 and 1.0 feet of overtopping for 1 to 3 hours will cause breaches to occur. At 7.5 feet NAVD88, approximately 20 percent of the inboard dike system would be likely to breach, a number representing 500 lineal feet (Figure 25).

Table 16: Elevation-Volume data for Alviso EIA, Western and Eastern Dike-Pond Systems

		Elevation-Volume Curves		
Elevation	Alviso EIA	Western Ponds[1]	Volume needed to reach Critical Elevation	Eastern Ponds[2]
(feet NAVD88)	(acre-feet)	(acre-feet)	(acre-feet)	(acre-feet)
-2.0	5	-		-
-1.0	29	-		-
0.0	168	980		102
1.0	465	1,864		160
2.0	919	3,268		252
3.0	1,741	5,858		643
3.73[3]	-	6,891	8,793[5]	-
4.3	2,547	8,104	7,580	1,351
5.2	3,492	10,364	5,320	2,156
6.6	4,953	13,395	2,289	3,237
7.5[4]	6,200	15,684	0	4,050
8.5	7,586	17,985		4,865
9.8	9,710	21,071		5,957
10.8	11,529	23,393		6,778
11.8	13,526			7,603
13.1	16,465			8,708
14.1	18,806			9,537

[1] Ponds A9 though A16
[2] Ponds A17, A18
[3] Mean Sea Level in South San Francisco Bay (2017), Coyote Creek tidal datum
[4] Critical elevation where inboard dikes will breach after overtopping for 1 to 3 hours
[5] Critical Volume needed to raise pond elevation to 7.5 feet NAVD88 and fail Inboard Dikes

The Alviso EIA ACE water level elevations are based on an exterior-interior relationship that was developed from a simple breach analysis, which transfers flood volume from the South Bay through a sequence of dike failures into the Alviso EIA (Table 17). Pertinent information for the breach analysis is listed below:

1. Critical Overtopping Elevation for inboard dikes = 7.5 feet NAVD88
2. Critical Overtopping Duration for inboard dike failure = 1 to 3 hours
3. V = Velocity through breach (6.0 to 6.5 feet/sec)
4. W = Cumulative Breach Width (200 to 755 feet) or (1.4 to 5.5 % of total outboard dike length)
5. D = Depth of Breach – Assume -1.0 feet NAVD88 mud line, equals (3HR WSE – (-1.0)) feet
6. Area = Assume rectangular breach W x D
7. 3HR Breach Volume = W x D x V x 3 hours
8. Ambient Pond Volume – Volume at MSL (3.73 feet) base year 2017 = 6891 acre-feet
9. Critical Overtopping Volume – Volume entering pond through outboard dike breach needed to raise pond elevation to 7.5 feet NAVD88, 8,793 acre-feet, assumes starting pond water level at MSL.
10. 3HR Breach Volume = Flood Volume to Alviso EIA
11. MHHW Elevation/Volume = 7.81 feet/6620 acre-feet, base year 2017

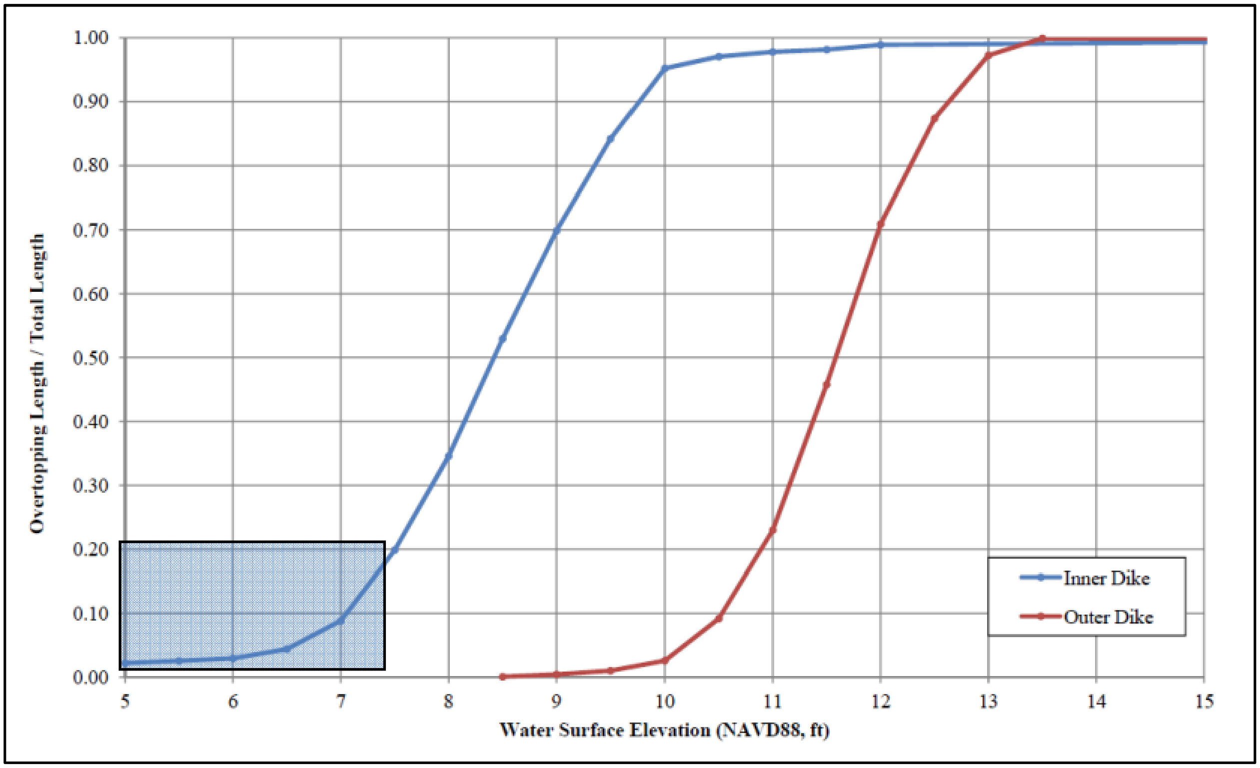


Figure 25: Inboard Dike Length (Blue) and Critical Overtopping Elevation of 7.5 feet NAVD88; Blue Box represents portion of total dike length likely to fail by overtopping (20 percent, approximately 500 lineal feet).

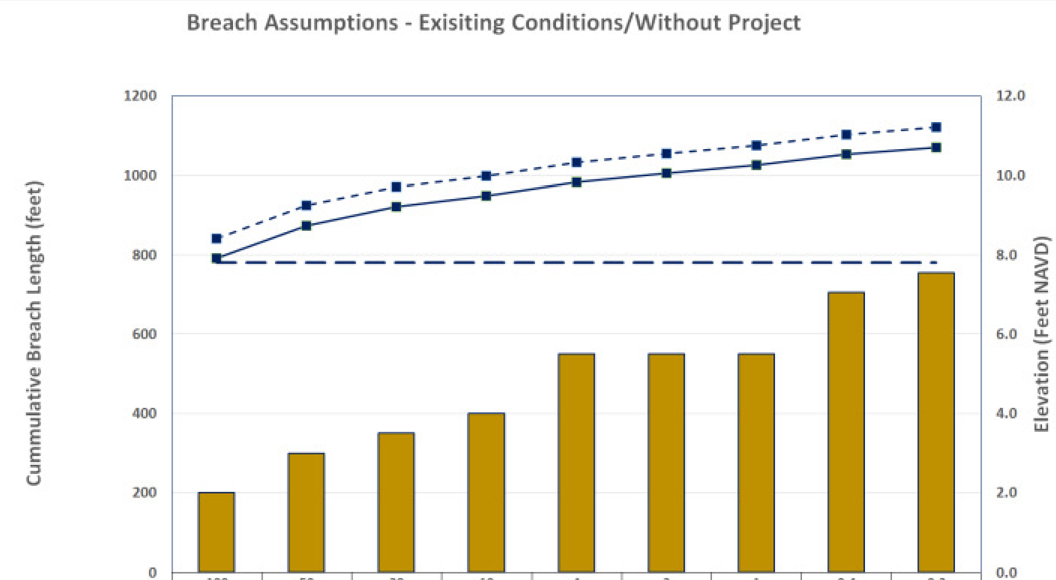
Table 17: Computation of Interior Water Surface Elevation for Alviso EIA from Breach Analysis

	2017 Exterior WSE[1]	**2017 3 hour WSE**	**Assumed Breach Width**	**Breach Area [2]**	**Velocity through breach**	**3 hour Breach Volume[3]**	**Critical Inboard Dike Overtopping Volume[4]**	**Flood Volume to Alviso[5]**	**2017 Interior WSEL[6]**
FREQ (%)	**feet NAVD88**	**feet NAVD88**		**feet^2**	**feet/sec**	**acre-feet**	**acre-feet**	**acre-feet**	**feet NAVD88**
99.99	8.42	7.92	200	1,784	6.00	2,654	8,793	6,620	7.81
50	9.25	8.75	300	2,924	6.00	4,349	8,793	6,620	7.81
20	9.71	9.21	350	3,573	6.00	5,315	8,793	6,620	7.81
10	9.99	9.49	400	4,195	6.00	6,241	8,793	6,620	7.81
4	10.32	9.82	525	5,412	6.25	8,806	8,793	8,806	9.34
2	10.55	10.05	530	5,524	6.25	9,073	8,793	9,073	9.49
1	10.76	10.26	540	5,629	6.25	9,420	8,793	9,420	9.63
0.4	11.02	10.52	705	8,124	6.45	12,922	8,793	12,922	11.02
0.2	11.21	10.71	755	8,838	6.50	14,243	8,793	14,243	11.21

[1] Coyote Creek tide gage
[2] Breach Width x Depth
[3] (Breach Width x Depth x Velocity) x 3 Hour
[4] Volume added to starting pond water surface elevation (MSL) volume (8793+6891) acre-feet
[5] Flood Volume to Alviso, less than 8,703 acre-feet, Flood Volume equal to MHHW – 7.81 feet
[6] Water Surface Elevation in Alviso EIA, from elevation-volume table (Table 16)

For the existing without-project base year scenario, a potential flood event impacting the Alviso EIA would occur as a result of an outboard dike breach though the western ponds. Any outboard dike breach will cause overtopping to occur at the inboard dikes protecting the Alviso EIA, either on the initial storm tide or from a subsequent MHHW tide. The dikes on the eastern pond system are higher, and carry less geotechnical risk overall, so the controlling failure mode describing hypothetical flood events is based on the western pond system.

A three-hour duration ACE water surface elevation for the 2017 Coyote Creek tide gage was developed to compute water volumes to be transferred into the western ponds during an outboard dike breach. The controlling factor in the transfer of tidal flood water volume into the Alviso EIA is the ambient water level in the pond, assumed to be at mean sea level. The ambient water level is the starting water level in the pond for the potential flood event and determines the volume of flood water needed to bring the pond water level to 7.5 feet NAVD88, the water level at which a significant amount overtopping occurs over 20 percent (500 feet) of inboard dikes, causing them to fail and transferring a substantial volume of water into the Alviso EIA during either the initial storm tidal cycle or subsequent tidal cycles. The potential flood event and simplified breach analysis describe the performance of the dike-pond system as it transitions from a closed system to a partially open system to finally a fully open system. The exterior-interior relationship between the Coyote Creek tide gage and the Alviso EIA reflects the performance and transitions between closed and open systems. Figure 26 shows the range of breach assumptions used to describe performance under the annual recurrence intervals.

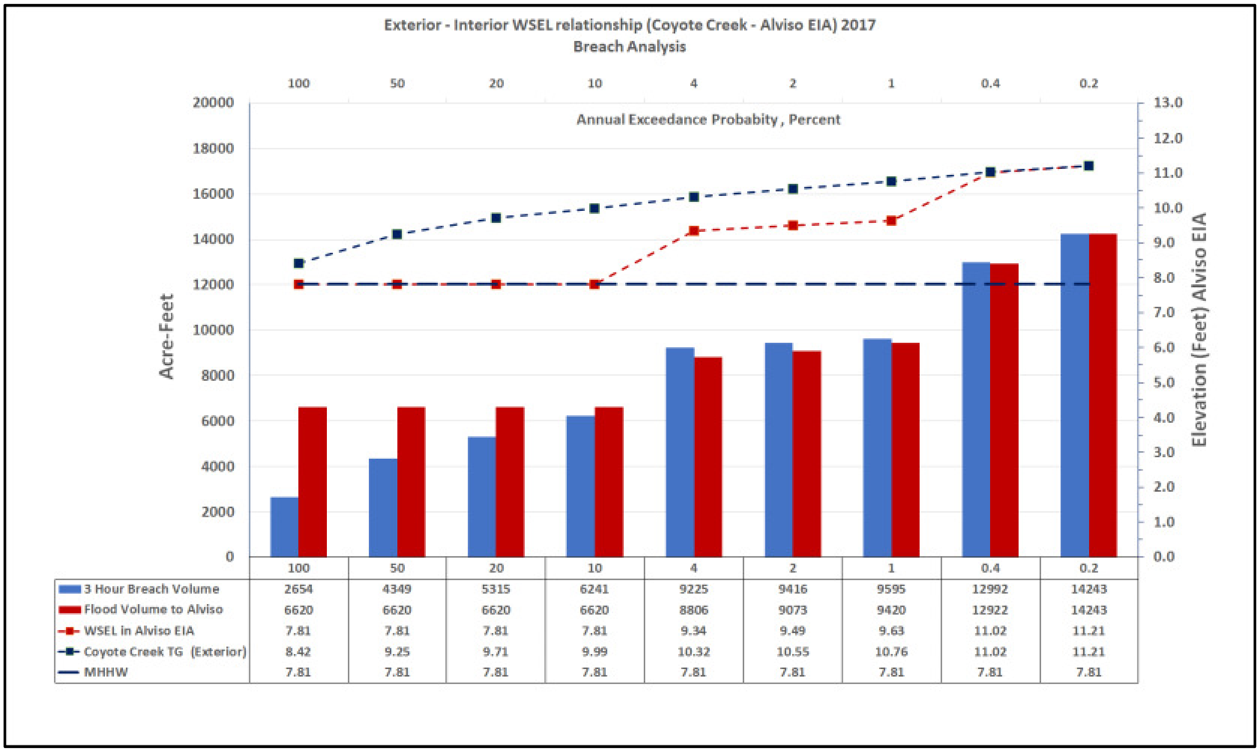


	100	50	20	10	4	2	1	0.4	0.2
Cummulative Length of Individual Breaches	200	300	350	400	550	550	550	705	755
Coyote Creek 3 Hour WSEL	7.92	8.75	9.21	9.49	9.82	10.05	10.26	10.52	10.71
MHHW	7.81	7.81	7.81	7.81	7.81	7.81	7.81	7.81	7.81
Coyote Creek AEP WSEL	8.42	9.25	9.71	9.99	10.32	10.55	10.76	11.02	11.21

Figure 26: Breach Assumptions for 3-hour ACE Water Surface Elevations for the Coyote Creek Tide Gage.

The ability of the dike pond system to transfer flood volume into the Alviso EIA is constrained by the ability of the pond to reach the critical overtopping elevation of 7.5 feet NAVD88 within a very short time. There are three flooding conditions possible:

1. *For the more frequent events, it is not possible to transfer enough volume into the pond to achieve the critical overtopping elevation of 7.5 feet under the initial tidal cycle containing the storm tide.* As the initial storm tide recedes, and water drains out of the pond through the outboard dike breaches, which will continue to expand on the falling tide and on the subsequent rising tides, the pond will refill to the MHHW elevation of 7.81 feet NAVD88, failing a section of inboard dikes and transferring flood volume into the Alviso EIA, the water surface in the Alviso EIA will reach equilibrium with the MHHW tide. This partially closed system will become completely open over subsequent tidal cycles, with Alviso at risk from flooding from daily high tides.
2. *Storm tides that are able to transfer enough volume to raise the pond elevation past the overtopping failure threshold while transferring a significant volume into the Alviso EIA during the initial storm tidal cycle.* This results in an internal water surface elevation at Alviso lower than the external water surface elevation at Coyote Creek. This level of performance reflects the fact that the breaches created are not sufficient to bring the pond into phase or equilibrium with the South Bay. This partially closed system will become completely open over subsequent tidal cycles, with Alviso at risk from flooding from daily high tides.
3. *Storm tides that are able to transfer enough volume to raise the pond elevation past the overtopping failure threshold during the initial storm tidal cycle by transferring a volume, which results in an internal water surface elevation in the Alviso EIA equal to the external water surface elevation at Coyote Creek.* This condition will occur when breaches and failures on both inboard and outboard dikes are substantial enough to create enough volume transfer for the pond to be in phase with the South Bay, creating an open system. This condition will occur more quickly if the ambient pond levels are raised creating a condition requiring less volume to fill the pond to the critical inboard dike overtopping elevation at 7.5 feet.



	100	50	20	10	4	2	1	0.4	0.2
3 Hour Breach Volume	2654	4349	5315	6241	9225	9416	9595	12992	14243
Flood Volume to Alviso	6620	6620	6620	6620	8806	9073	9420	12922	14243
WSEL in Alviso EIA	7.81	7.81	7.81	7.81	9.34	9.49	9.63	11.02	11.21
Coyote Creek TG (Exterior)	8.42	9.25	9.71	9.99	10.32	10.55	10.76	11.02	11.21
MHHW	7.81	7.81	7.81	7.81	7.81	7.81	7.81	7.81	7.81

Figure 27: Exterior-Interior Water Surface Relationship between Coyote Creek Tide Gage and Alviso assuming Outboard Dike Breaching.

3.2.8 ALVISO ECONOMIC IMPACT AREA, FUTURE WITHOUT-PROJECT CONDITION FLOOD RISK

Future conditions in the project are impacted by SLC, which in turn further reduces the performance and reliability of the existing west and east dike pond systems currently preventing tidal flooding in the Alviso EIA.

Under the three SLC scenarios, the assumption is that the tidal ranges in San Francisco Bay remain unchanged, but shift to higher levels and inland. The ACE statistics are projected forward under the three SLC rates. The ability of the existing dike-pond systems to prevent tidal flooding declines significantly and rapidly under the USACE High SLC scenario. Figure 27 illustrates the transfer in volume under an assumed failure of the dike-pond system that defines the exterior-interior relationship between Coyote Creek and Alviso in the base year of 2017.

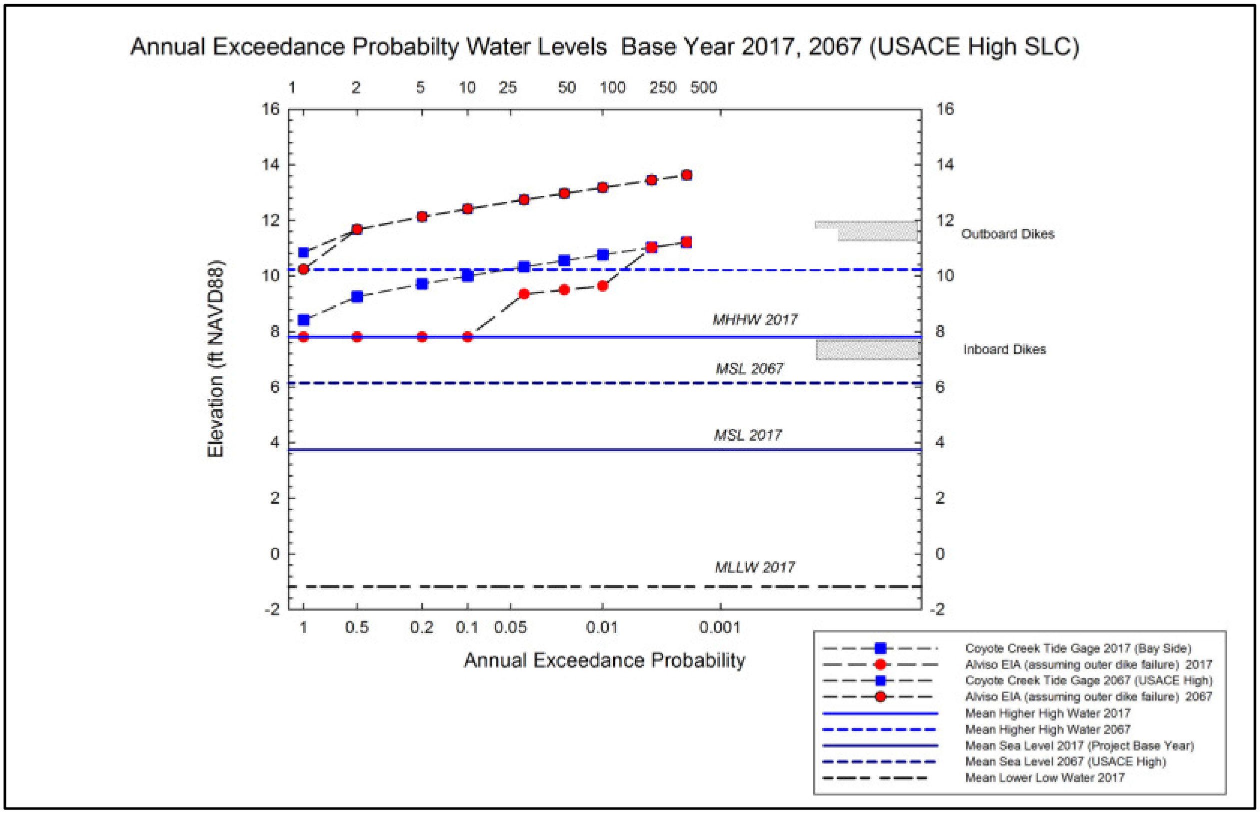


Figure 28: ACE for Coyote Creek Tide Gage and Alviso EIA for 2017 and 2067 under USACE High SLC Scenario.

The impact of SLC on the performance of the dike-pond system and the change in exterior-interior water surface elevation relationship can be seen in Figure 28. The change in mean sea level, potentially several feet higher under the USACE High SLC scenario effectively eliminates any flood risk reduction benefit by the dike-pond system through storage. Water would only need to rise by 1 to 1.5 feet for the inboard dikes to be overtopped and fail. The transition to a completely open system now occurs at the 50 percent ACE, and the exterior-interior relationship is no longer in effect. ACE water surface elevations are developed in 10-year increments for the base year 2017 through 2067 using the web tool at https://corpsclimate.us/ccaceslcurves.cfm. The low rate is used for all 2017 scenarios since the base year of 2017 is so close to the current year. Exterior-interior relationships between the Coyote Creek tide gage and Alviso EIA based on breach analysis developed for the existing without-project condition are estimated for the future SLC scenarios, accounting for changes impacting performance.

Table 18, Table 19, and Table 20 contain ACE water surface elevations for the three SLC scenarios, USACE Low, Intermediate, and High.

Table 18: USACE Low SLC Scenario - ACE Water Surface Elevations, Ext - Coyote Creek Gage, Int - Alviso

	2017		2027		2037		2047		2057		2067	
ACE (%)	Ext (ft.)	Int (ft.)	Ext (ft.)	Int (ft.)	Ext (ft.)	Int (ft.)	Ext (ft.)	Int (ft.)	Ext (ft.)	Int (ft.)	Ext (ft.)	Int (ft.)
99.99	8.42	7.81[1]	8.49	7.88[1]	8.55	7.94[1]	8.62	8.01[1]	8.69	8.08[1]	8.76	8.15[1]
50	9.25	7.81[1]	9.32	7.88[1]	9.38	7.94[1]	9.45	8.01[1]	9.52	8.08[1]	9.59	8.15[1]
20	9.71	7.81[1]	9.78	7.88[1]	9.84	8.50	9.91	8.45	9.98	8.65	10.05	9.20
10	9.99	7.81[1]	10.06	8.30	10.12	8.70	10.19	8.90	10.26	9.15	10.33	9.45
4	10.32	9.34	10.39	9.36	10.45	9.65	10.52	9.80	10.59	9.99	10.66	10.20
2	10.55	9.49	10.62	9.57	10.68	9.75	10.75	9.92	10.82	10.70	10.89	10.80
1	10.76	9.63	10.83	9.75	10.89	9.85	10.96	10.80	11.03	11.03	11.10	11.10
0.4	11.02	11.02	11.09	11.09	11.15	11.15	11.22	11.22	11.29	11.66	11.36	11.36
0.2	11.21	11.21	11.28	11.28	11.34	11.37	11.41	11.41	11.48	11.85	11.85	11.85

[1] MHHW

Table 19: USACE Intermediate SLC scenario - ACE Water Surface Elevations, Ext - Coyote Creek Gage, Int - Alviso

	2017		2027		2037		2047		2057		2067	
ACE (%)	Ext (ft.)	Int (ft.)	Ext (ft.)	Int (ft.)	Ext (ft.)	Int (ft.)	Ext (ft.)	Int (ft.)	Ext (ft.)	Int (ft.)	Ext (ft.)	Int (ft.)
99.99	8.42	7.81[1]	8.60	7.99[1]	8.73	8.12[1]	8.89	8.28[1]	9.06	8.45[1]	9.26	8.65[1]
50	9.25	7.81[1]	9.43	7.99[1]	9.56	8.12[1]	9.72	8.28[1]	9.89	8.45[1]	10.09	8.65[1]
20	9.71	7.81[1]	9.89	7.99[1]	10.02	8.50	10.18	9.45	10.35	9.78	10.55	10.55
10	9.99	7.81[1]	10.17	8.50	10.30	9.50	10.46	9.65	10.63	10.49	10.83	10.83
4	10.32	9.34	10.50	9.40	10.63	9.80	10.79	10.40	10.96	10.96	11.16	11.16
2	10.55	9.49	10.73	9.68	10.86	10.60	11.02	11.02	11.19	11.19	11.39	11.39
1	10.76	9.63	10.94	10.55	11.07	11.07	11.23	11.23	11.40	11.40	11.60	11.60
0.4	11.02	11.02	11.20	11.20	11.33	11.33	11.49	11.49	11.66	11.66	11.86	11.86
0.2	11.21	11.21	11.39	11.39	11.52	11.52	11.68	11.68	11.85	11.85	12.05	12.05

[1] MHHW

Table 20: USACE High SLC Scenario - ACE Water Surface Elevations, Ext - Coyote Creek Gage, Int - Alviso

	2017		2027		2037		2047		2057		2067	
ACE (%)	Ext (ft.)	Int (ft.)	Ext (ft.)	Int (ft.)	Ext (ft.)	Int (ft.)	Ext (ft.)	Int (ft.)	Ext (ft.)	Int (ft.)	Ext (ft.)	Int (ft.)
99.99	8.42	7.81[1]	8.94	8.33[1]	9.30	8.69[1]	9.74	9.13[1]	10.26	9.65[1]	10.84	10.23[1]
50	9.25	7.81[1]	9.77	8.33[1]	10.13	8.69[1]	10.57	9.85	11.09	11.09	11.67	11.67
20	9.71	7.81[1]	10.23	8.75	10.59	9.70	11.03	11.03	11.55	11.55	12.13	12.13
10	9.99	7.81[1]	10.51	9.50	10.87	10.10	11.31	11.31	11.83	11.83	12.41	12.41
4	10.32	9.34	10.84	9.80	11.20	11.20	11.64	11.64	12.16	12.16	12.74	12.74
2	10.55	9.49	11.07	11.07	11.43	11.43	11.87	11.87	12.39	12.39	12.97	12.97
1	10.76	9.63	11.28	11.28	11.64	11.64	12.08	12.08	12.60	12.60	13.18	13.18
0.4	11.02	11.02	11.54	11.54	11.90	11.90	12.34	12.34	12.86	12.86	13.44	13.44
0.2	11.21	11.21	11.73	11.73	12.09	12.90	12.53	12.53	13.05	13.05	13.63	13.63

[1] MHHW

4.0 GEOTECHNICAL ENGINEERING TECHNICAL SUMMARY

4.1 INTRODUCTION

This section summarizes the assumptions for geotechnical performance for the existing pond dikes within the South San Francisco Bay Shoreline (SSFBS) study area. The proposed SSFBS project includes ecosystem restoration in retired salt production ponds and the construction of flood risk management features along an existing inboard dike on the west and east side of Artesian Slough. No existing dikes or berms are engineered structures. The geotechnical recommendations are focused on the outboard and inboard dike system west of Artesian Slough (Figure 29). By comparison, the existing condition of the west side of the project is consistently at lower elevations (i.e., > 2 ft) on both inboard and outboard dikes. Therefore, the likely source of initial flooding under more frequent flood events is through the dike-pond system that is west of Artesian Slough.

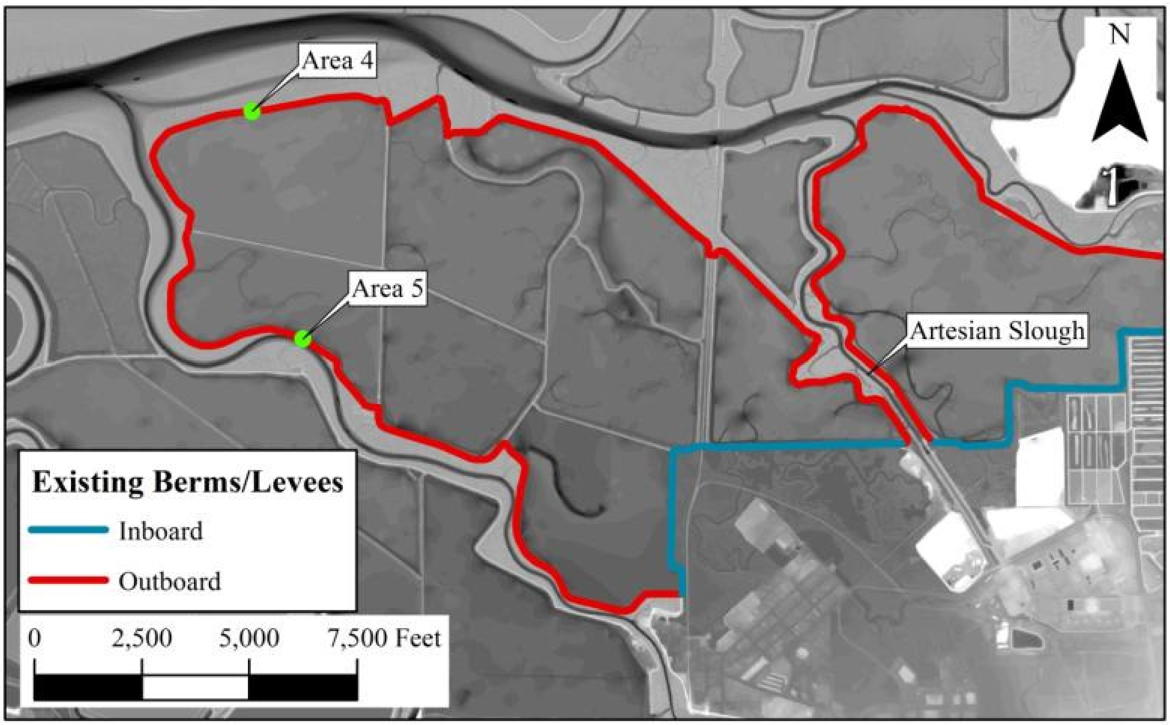


Figure 29: Project Map of Existing Dikes and Berms.

There are no new geotechnical analyses relative to what was completed for the USACE Feasibility Scoping Meeting milestone (USACE, 2009a)and Alternative Formulation Briefing milestone (USACE, 2013b) to support the current effort to identify the Federal interest and determine whether a potential project is economically justified. Instead, research of existing sources of geotechnical information and analyses were used to revise the geotechnical assumptions that have been applied for the reevaluation of Federal interest and economic justification for a future project. Through this effort, the failure mechanisms that form the geotechnical fragility curve of the outboard dike were reevaluated. The revised geotechnical fragility curve was judged to be more appropriate in light of observed performed at the project site for water levels that are lower than those characteristic of a higher SLC curve.

The geotechnical performance assumes that the outboard dike is the only line of protection. This approach assumes that a breach failure at the outboard dike will result in a subsequent breach from overtopping at the inboard dike above a specific threshold loading.

4.2 GEOTECHNICAL PERFORMANCE

4.2.1 OUTBOARD DIKE PERFORMANCE (FRAGILITY CURVE)

Geotechnical fragility curves for the entire SSFBS project were developed for the FSM milestone (USACE, 2009a) to characterize the without-project condition of the existing pond dikes. This effort leveraged data from existing (650 SPT and 43 CPT soundings), as well as new (34 SPT and 102 CPT soundings), geotechnical exploration locations along the existing inboard and outboard dikes. This data was used to create a total of 14 index points; six on the outboard dikes and eight on the inboard dikes. Two of the index points developed, Area 4 and Area 5, are along the outboard dike that is west of Artesian Slough (Figure 29). Probability of unsatisfactory performance (P_u), also referred to as probability of failure, was reported as a function of water surface elevation from the crest (i.e., crest elevation minus water surface elevation).

The fragility curve developed for the Feasibility Scoping Meeting milestone was based upon seepage and rapid drawdown and judged incompatible with the short duration (hours) loading of flood events. Erosion and overtopping erosion were identified as the mechanisms critical to determining the likelihood of failure/breach of the outboard dike. In addition, newer and higher resolution survey information in the study area had been collected. An additional fragility curve was developed to more accurately represent loading (i.e. erosion and overtopping) and updated dike dimensions (i.e. elevation and crest width) known to exist in the study area.

An additional fragility curve was developed for combined erosion and overtopping mechanisms. No new geotechnical analysis was performed to quantitatively support the current effort. However, existing analysis for erosion and overtopping as well as empirical observations of dike performance were leveraged to support the revised fragility curve. The primary factors supporting the revised fragility curve were (i.) typical conditions along the outboard dike, (ii.) hydraulic and breach modeling already performed for the without project condition in the study area, and (iii.) observed performance relative to maintenance performed.

A 2010 USGS LiDAR survey of the study area was used to identify the typical configuration of the outboard dike. The cross-section geometry was sampled at 21 representative locations (Figure 30). Cross sections were purposely concentrated in areas where overtopping is likely to occur first (i.e., saddles) and/or erosion is more likely (i.e., proximity to sloughs). Plotted cross sections are shown in Figure 31. Crest widths were estimated by measured the section width 1 ft below the peak crest elevation. This method was used to avoid underestimating crest widths due to irregular topography. Factors that contribute to functionally narrower crests, such as rodent holes, were not considered in the estimate of the crest width. Average crest elevation and width of the selected cross sections was 10.8 ft NAVD88 and 18 ft, respectively.

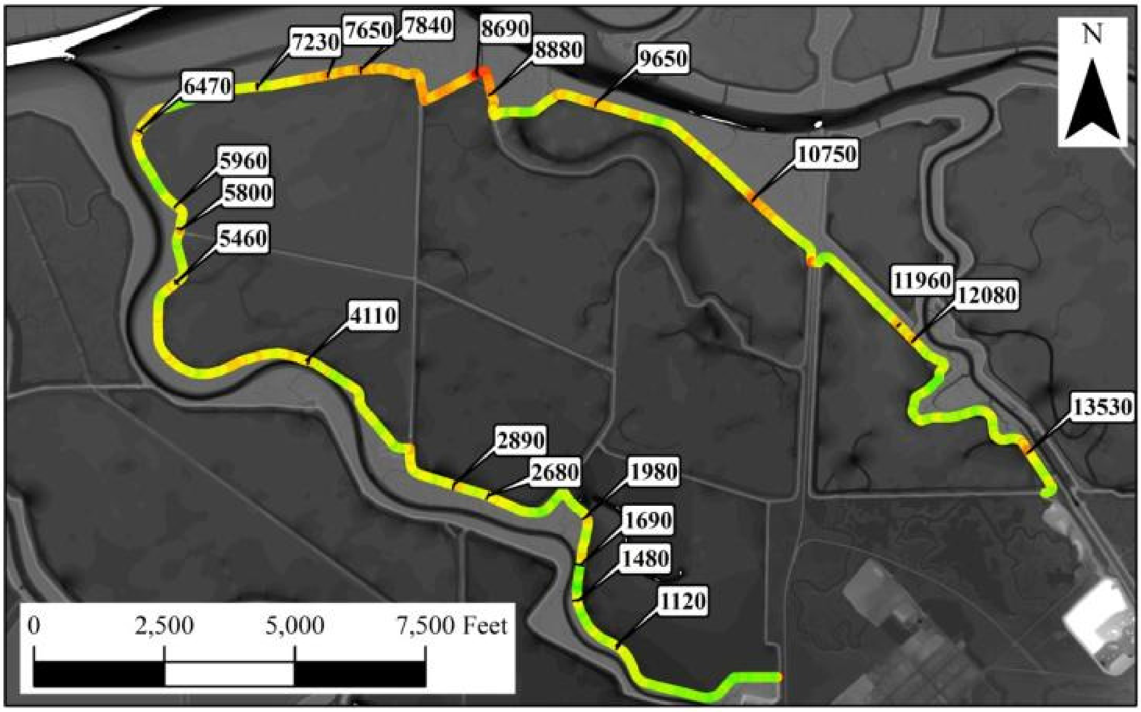


Figure 30: Locations of Select Cross sections Along the Outboard Dike.

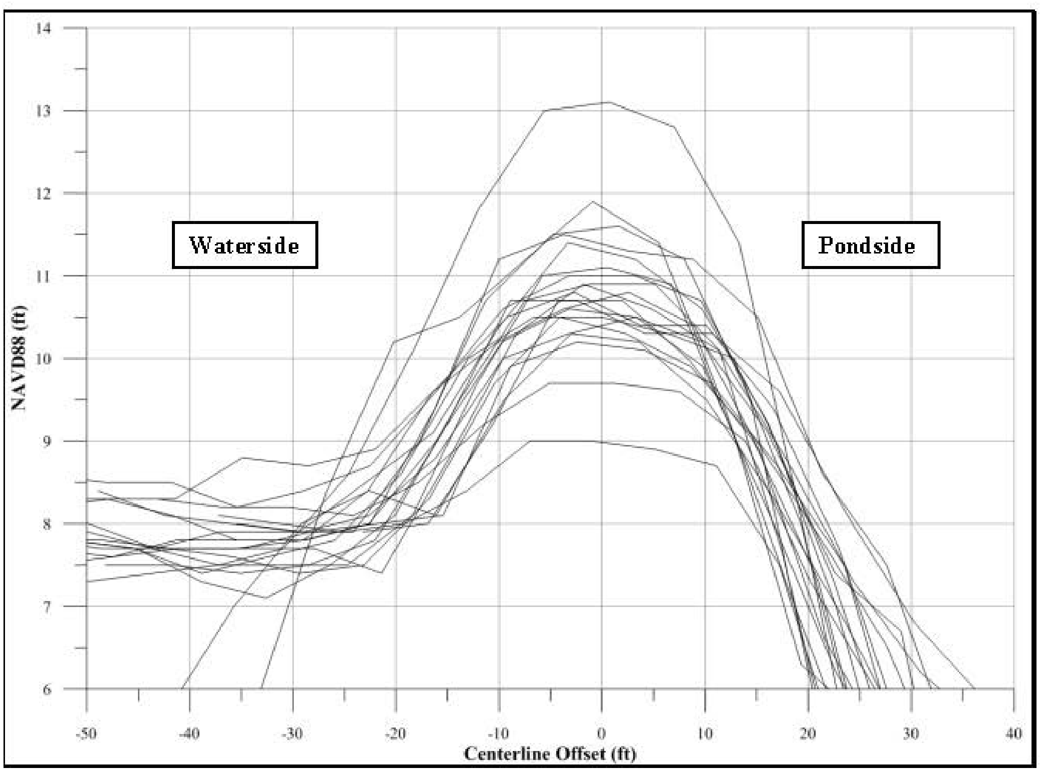


Figure 31: Cross sections along the Outboard Dike

Overtopping and erosion are critical to the performance of the outboard dike. Existing information duration of tidal flood events and the results of breach modeling efforts in the study area were used to estimate the thresholds at which the likelihood of breach along the outboard dike will occur. The following section discusses the basis for estimated loading duration and respective performance impacts to the outboard dike with respect to the peak water surface elevation (WSE) experience during a flood event.

The duration of flood loading was estimated using the tidal signal (i.e., shape) from the San Francisco Golden Gate tide gauge. The peak of the signal was set equal to a given WSE and the duration above lower elevations was recorded. Table 21 shows the approximate durations of loading above elevations incrementally lower than the peak WSE.

Table 21: Summary of Durations Exceeding Elevations Lower than the Peak WSE

Peak Water Level (NAVD88, ft)	WSE above (NAVD88, ft)	Duration Above WSE (hr)
12	11	4.5
	10	7
	9	9
	8	> 10
11	10	4.5
	9	7
	8	9
10	9	4.5
	8	7

(USACE, 2008) details the investigation and modeling effort to establish likely times to breach from wave attack, overtopping erosion, or both. Table 22 summarizes the overtopping scenarios likely to induce a breach at the outboard dike between Alviso and the ponds west of Artesian Slough. The table was adapted from (USACE, 2008) and shows the expected time to breach for overtopping scour only.

Table 22: Estimated Time to Breach versus Dike Crest Width

		Expected critical time to breach (hr) for respective crest width (ft)					
q (ft^3/s) per foot of dike	Height (ft) of overtopping	W = 25*	W = 20*	W = 15	W = 11	W = 7	W = 5
0.5	0.30	--	--	42.86	31.43	19.43	14.04
1	0.47	--	--	9.19	6.7	4.33	2.98
2	0.75	--	--	4.46	3.32	2.08	1.49
3	0.98	5.50	4.40	3.29	2.42	1.53	1.09
4	1.19	4.60	3.70	2.75	2.02	1.27	0.91

1. Overtopping flow rate from the Feasibility Scoping Meeting Geotechnical Appendix (USACE, 2009b)

2. Overtopping height determined from broad crested weir equation (Henderson, 1966).

3. (*) Indicates time to breach estimated from linear fit of data for dikes with W from 5 to 15 ft.

The cross-section geometry, anticipated loading duration, loading required for overtopping breach, and past performance were considered to identify possible breach locations. Figure 32 shows potential overtopping breaches that can be expected to occur from a given peak WSE. Point labels represent crest elevation and width at respective outboard dike station (Figure 30). Lines draw indicate the approximate threshold (i.e. overtopping duration vs. crest width) to which overtopping breaches are likely to occur. Of the 21 cross sections evaluated, three locations are at risk of an overtopping breach for a peak WSE of 11 ft. The number of potential overtopping breaches increases to 12 for a peak WSE of 12 ft.

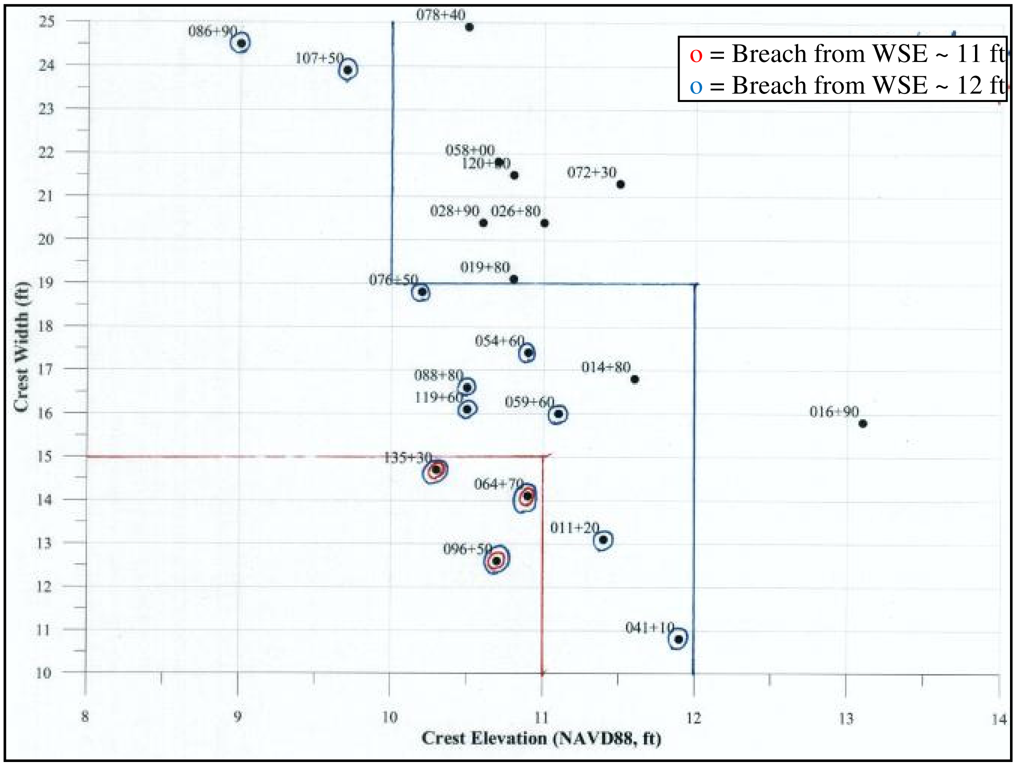


Figure 32: Potential Overtopping Breach Locations for Given Peak WSE.

The impact of wave attack and erosion on the waterside of the outboard contribute to the performance of the outboard dike. USACE (2008) modeled wave attack, however, wave height (i.e. 3 ft height or greater) was judged to be overestimated by at least 2 ft in the study area. Past performance along the outboard dike during frequent (i.e. non-overtopping) events was inferred from maintenance records for the period 1995 to 2005 (Geomatrix, 2006). These records provide a generally coarse interpretation of distress along the outboard dike. Figure 33 shows the number of repair episodes along the outboard dike in the period of record. Figure 34 shows the summed extent of repairs in the period of record when such records were available. The extent of repairs was typically described in terms of linear feet and/or cubic yards. A review of the storm frequency and annual maximum water levels showed a positive correlation between "stormier years" and increased maintenance (i.e. 1997 and 2003).

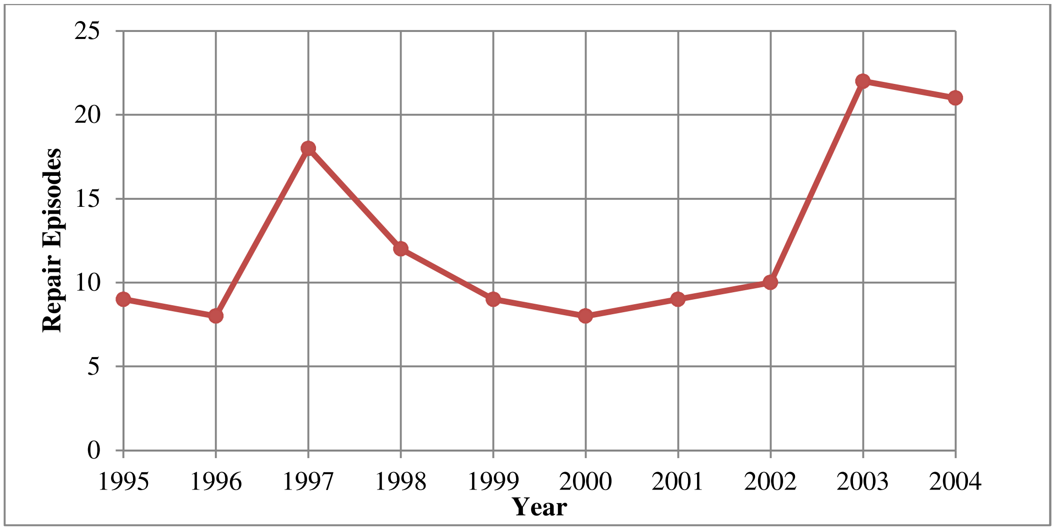


Figure 33: Number of Maintenance Episodes by Year along the Outboard Dike.

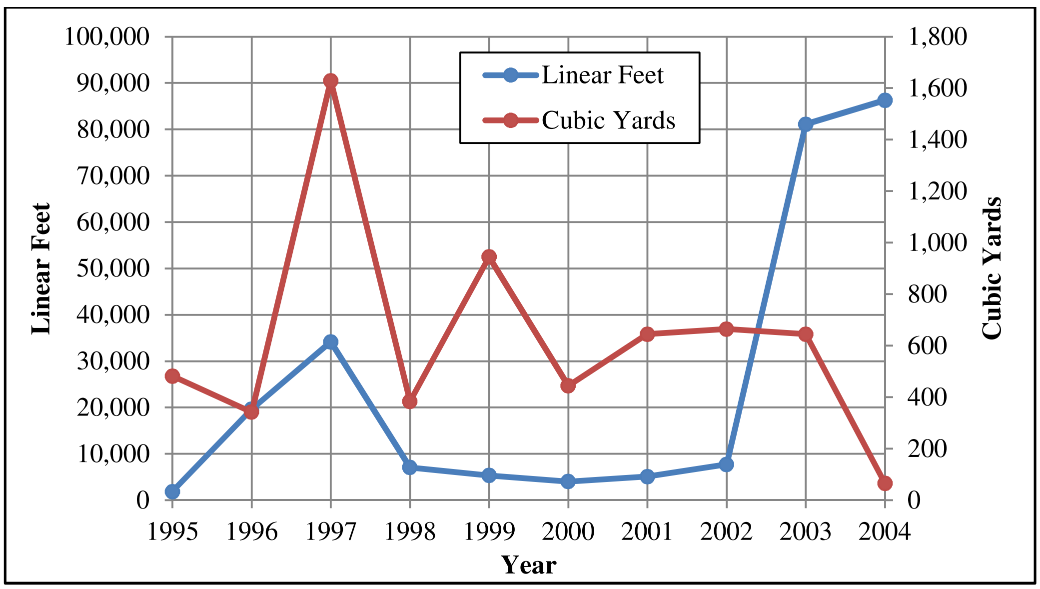


Figure 34: Summed Total Extent of Repairs by Year along the Outboard Dike.

The fragility curve for outboard dike combined geotechnical investigation, numerical modeling, and maintenance record datasets to capture the primary mechanisms critical to performance along the outboard dike; overtopping and erosion. The key assumptions used to construct the fragility curve are as follows:

- Time to overtopping breach is quantitatively supported in the geotechnical analyses performed in USACE (2009a).
- Maintenance records demonstrate distress and/or damage occurring in "stormier years" with presumably higher than typical water surface elevations. Maintenance was generally ad-hoc when the ponds and associated dikes were owned by Cargill, Incorporated; however, the U.S.

Fish and Wildlife Service (FWS) performs maintenance annually in the period following the wet season.

- Wave height in the project area is limited to 0.5 to 1 ft above the static WSE and does not increase with increasing static WSE. The outboard dike is assumed partially exposed to wave attack above elevation 8 ft and fully exposed above elevation 9 ft (USACE 2008).
- The extent of resources (e.g., funding and staff) for FWS to maintain the outboard dike into the future is uncertain. To date, repairs have been prioritized to the areas of highest need and is not comprehensive to all needs (USACE, 2014a)(USACE, 2014).

Figure 35 shows the fragility curves developed for the SSFBS study. Table 23 shows the estimated probability of unsatisfactory performance for the two mechanisms and the combined probabilities for respective elevations. Commentary is provided below as justification to support the engineering judgment applied and to describe the typical conditions anticipated during specific loading.

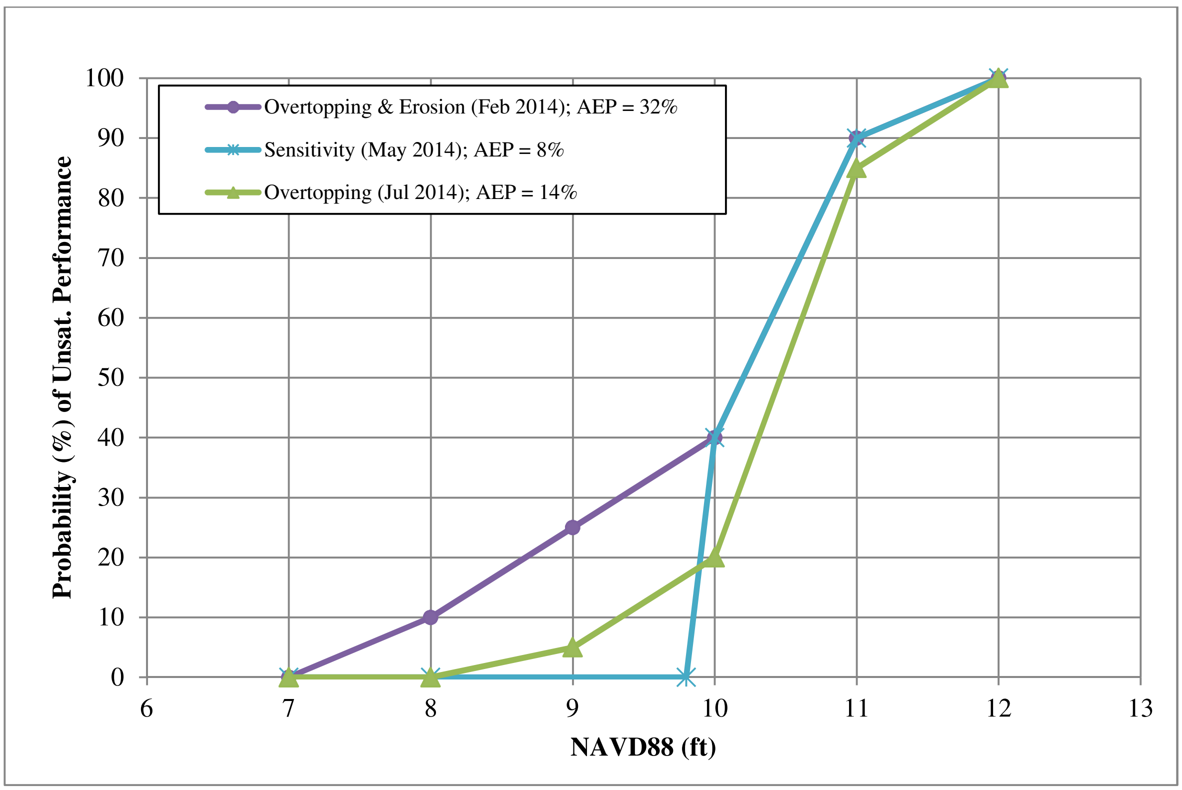


Figure 35: Outboard Dike Fragility Curves Developed.

Two additional fragility curves were developed and implemented in HEC-FDA to the risks assumed during the reanalysis of SLC scenarios (Figure 35). The first added curve was constructed to disallow failure below elevation 10 ft and the second is the probability of failure induced by overtopping only. Both additional curves represent conditions that ignore mechanisms/elevations that are documented to have negative impacts to outboard dike performance. However, there is concern that implementation of a geotechnically appropriate fragility curve in HEC-FDA and the application of a specific failure mode scenario may have led to the overestimation of flood risks in the immediate future. The sensitivity analysis addressing the impacts on the economic results when adjusting the fragility curve is discussed in Section 5.0

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Table 23: Updated Probability of Unsatisfactory Performance (Breach) Based on Erosion & Overtopping Only

Static WSE (NAVD88, ft)	Probability of Failure (P_u)			Comments
	Erosion	Overtopping	Combined	
12	0.3	1.0	1.0	1. 32,000 ft of outboard dike (70% of length) overtops. About 21,000 ft overtops over elevation 11 ft for 4hrs, possibly inducing up to 3 overtopping breaches. 2. Overtopping of crest elevations at 10 ft for 6.5 hours, possibly inducing 9 additional overtopping breaches (Figure 32).
11	0.3	0.85	0.90	1. 9,250 ft of outboard dike (25% of length) overtops above elevation 10 ft for 4 hrs. Potential overtopping breaches at three locations. 2. Overtopping height is transient and the duration required to induce breaching may not occur. 3. Breach from combined erosion and overtopping increases the likelihood of breach at the three locations (Figure 32).
10	0.25	0.20	0.40	1. Overtopping at a limited number of locations. These locations have wide sections and sustain overtopping erosion for proportionally longer durations than narrow (< 15 feet) sections. 2. The dike crest in several reaches is composed of loose highly erodible silt with organics (USACE, 2014a). Time to overtopping breach may be substantially shorter in these reaches. 3. Rodent activity in the uppermost 1 to 3 feet of the dike section may contribute to internal erosion (USACE, 2014a) or effectively "narrower" crest width available during overtopping. 4. Increased size and frequency of maintenance can be expected based on maintenance records (Geomatrix 2006). 5. The difference between the 2010 site survey and current conditions in 2014 is uncertain (e.g. potential for lower and thinner than measured crest elevations). 6. Repairs/Action to restore crest elevation from subsidence is recognized only after overtopping occurs (i.e., no periodic surveys/measurements of dikes). 7. Dike vulnerability to combined erosion and overtopping in low spots is very minor or incipient overtopping.
9	0.2	0.05	0.25	1. WSE in the range observed to have increased frequency and scope of repairs. 2. Lower WSE more frequent in a single wet season with maintenance performed annually and not ad-hoc. 3. Prioritization of repairs/maintenance relative to available resources can allow "semi-vulnerable" locations to become increasing vulnerable to loading. 4. Loss of section height and width due to normal coastal processes.
8	0.1	0	0.10	
7	0	0	0.0	1. Water levels experienced frequently (daily to weekly) with no noteworthy distress.

Notes:
1. Calculated per ETL 1110-2-547; (1 - Erosion) * (1 - Overtopping) = 1 - Combined.

4.2.2 INBOARD DIKE PERFORMANCE

The inboard dike was assumed to fail due to overtopping. The inboard dike crest width is variable in the reach west of Artesian Slough. Crest widths are typically between 10 and 15 ft wide but can be as little as 8 ft along the alignment. Crest elevations vary from 6 to 11 feet suggesting substantial overtopping length (i.e. 1,000 ft) if the dike was exposed to normal high tides (i.e. MHHW = 7 ft NAVD88) or greater than one mile of overtopping length for WSEs that cause an overtopping breach of the outboard dike. It can be inferred from Table 23 that an overtopping height of 1 ft for the duration of 3 to 4 hrs is likely to induce a breach through the inboard dike. An accumulation of overtopping high tide cycles in the days following a non-overtopping outboard dike breach, or an overtopping induced breach of the outboard dike would result in subsequent failure of the inboard dike.

Static failures prior to overtopping were not considered credible during the current effort. Water levels have been sustained for significant periods near mean tide elevation (i.e., 3.5 ft) without failure. If the outboard dike experienced a breach, normal high tide water levels (i.e., MHHW ~ 7 ft) would overtop the lowest reaches (elevation 6 to 6.5 ft) of the inboard dike. Therefore, sustained water levels that are appreciably above elevation 3 ft and do not overtop the inboard dike are highly unlikely.

4.2.3 FAILURE MODE SEQUENCE

The geotechnical performance of the outboard dike is critical to the performance of the entire dike-pond system. The failure at the outboard dike will result in overtopping and subsequent failure at the inboard dike. Overtopping is likely to occur at as low as elevation 6.5 ft for the inboard dike. Overtopping, or a breach before overtopping, of the outboard dike will likely result in at least 2 feet of overtopping at the inboard dike. In addition, a breach of the inboard dike is assumed to occur shortly after breach of the outboard.

5.0 ECONOMICS TECHNICAL SUMMARY

5.1 WITHOUT-PROJECT FLOOD DAMAGE

Table 24 below shows the economic damages estimated in the HEC-FDA model for the USACE Intermediate SLC scenario. The decrease in damages between consecutive years is due to the assumption that relocations are occurring in response to damaging floods. For example, the decrease in damages from 2026 to 2027 is a result of the structure relocations that were assumed to take place in 2027.

Table 24: Without-Project Structure & Content EAD, Intermediate SLC Scenario (1,000s)

Year	Commercial	Displacement	Industrial	Public	Residential	Total
2017	$4,845	$471	$2,542	$553	$2,945	$11,356
2026	$7,181	$617	$3,109	$675	$3,691	$15,273
2027	$6,799	$373	$418	$255	$2,230	$10,075
2036	$10,383	$515	$565	$292	$2,712	$14,467
2037	$9,421	$419	$568	$46	$2,200	$12,654
2046	$12,716	$527	$662	$52	$2,564	$16,521
2047	$12,189	$388	$608	$52	$1,763	$15,000
2056	$21,343	$848	$887	$85	$3,262	$26,425
2057	$14,363	$680	$44	$17	$1,948	$17,052
2067	$23,421	$1,234	$69	$33	$3,466	$28,223

As noted previously, in addition to the structure and content damage calculated in the HEC-FDA models, the without-project damage analysis considers the cost of relocations out of the floodplain and the cost to protect the San Jose/Santa Clara Water Pollution Control Plant. The plant serves 1.4 million people and thousands of businesses, and is the largest treatment plant in the region. In the absence of a structural project to keep coastal storm water from reaching the basin, it is assumed that, because of its economic and environmental importance, actions would be taken to reduce the likelihood of damage to the plant. A ring levee surrounding the plant was estimated to cost $25M to construct. It was assumed that in the face of increased coastal flood risk the ring levee would be constructed by 2027.

Table 25 below shows an example of how the total EAD is calculated for each year of the period of analysis. The table only shows eleven years of the period of analysis, for illustrative purposes. For each year, the damages from all of the damage categories are summed and the present value is calculated using the applicable discount rate. The values for the intervening years between the beginning and end of each FDA model's 10-year periods of analysis were calculated by interpolation. The sum total of the annual present values is then annualized to calculate an equivalent annual damage.

Table 25: Example of Without-Project Total EAD Calculation - Intermediate SLC Scenario (1,000s)

Year of Project	Year	Present Value Factor	EAD From FDA Model	Relocation Cost	Ring Levee Construction	Total	Present Value
0	2017	1.000	$11,356	$0	$0	$11,356	$11,356
1	2018	0.966	$11,791	$0	$0	$11,791	$11,392
2	2019	0.934	$12,226	$0	$0	$12,226	$11,413
3	2020	0.902	$12,661	$0	$0	$12,661	$11,420
4	2021	0.871	$13,096	$0	$0	$13,096	$11,413
5	2022	0.842	$13,532	$0	$0	$13,532	$11,393
6	2023	0.814	$13,967	$0	$0	$13,967	$11,362
7	2024	0.786	$14,402	$0	$0	$14,402	$11,320
8	2025	0.759	$14,837	$0	$0	$14,837	$11,268
9	2026	0.734	$15,273	$0	$0	$15,273	$11,206
10	2027	0.709	$10,075	$168,484	$25,000	$203,559	$144,306

The total equivalent annual damage for the fifty-year period of analysis under the USACE Low, Intermediate, and High SLC scenarios is $18.2M, $22.6M, and $40.2M, respectively.

5.2 WITH-PROJECT RESULTS

5.2.1 DAMAGES REDUCED

The with-project economic analysis was conducted by inputting levees of various heights into the HEC-FDA models. As tables further below show, a non-structural plan was also analyzed.

For all plans involving levee construction, an assumption was made that no relocations would occur, and that for levee heights above the elevation of the water pollution control plant no ring levee would be constructed. Table 26 below shows the first eleven years (for comparison's sake with the without-project results) of the with-project analysis for a 12' levee height.

Table 26: Example of With-Project Total EAD Calculation – 12' Levee, USACE Intermediate SLC Scenario (1,000s)

Year of Project	Year	Present Value Factor	EAD From FDA Model	Relocation Cost	Ring Levee Construction	Total	Present Value
0	2017	1.000	$33	$0	$0	$33	$33
1	2018	0.966	$37	$0	$0	$37	$36
2	2019	0.934	$41	$0	$0	$41	$38
3	2020	0.902	$45	$0	$0	$45	$40
4	2021	0.871	$49	$0	$0	$49	$42
5	2022	0.842	$53	$0	$0	$53	$44
6	2023	0.814	$56	$0	$0	$56	$46
7	2024	0.786	$60	$0	$0	$60	$47
8	2025	0.759	$64	$0	$0	$64	$49
9	2026	0.734	$68	$0	$0	$68	$50
10	2027	0.709	$72	$0	$0	$72	$51

5.2.2 BENEFIT TO COST RATIO, NET BENEFITS, & RESIDUAL RISK

Table 27, Table 28, and Table 29 below show the results for each levee height under each of the three USACE SLC scenarios. As the results tables show, a levee project is economically justified under any of the three SLC scenarios. A 12.5' levee is the height with the greatest net benefits under the USACE Low and Intermediate scenarios, and a 13.5' levee is the greatest net benefit plan under the USACE High SLC scenario. The small difference in net benefits between many of the levee heights is in large part a reflection of the small difference in the construction cost. The non-structural plan, which involves the relocation of several hundred homes and businesses, while highly effective at reducing flood damage is the least efficient option, consistently having the lowest benefit to cost ratio for all of the options.

Table 27: With-Project Results – USACE Low SLC Scenario

Without-Project Equivalent Annual Flood Damage (1,000s)										
Structure & Content Damage	$11,478									
Relocation Cost	$6,691									
Total	$18,170									
With-Project Equivalent Annual Damages & Damages Reduced (1,000s)										
	No Action	11' Levee	11.5' Levee	12' Levee	12.5' Levee	13' Levee	13.5' Levee	14' Levee	15' Levee	Non-Structural
With-Project Avg Annual Flood Damage	$18,170	$2,418	$1,123	$84	$17	$0	$0	$0	$0	$0
Annual Damages Reduced	$0	$15,752	$17,047	$18,086	$18,153	$18,170	$18,170	$18,170	$18,170	$18,170
Project Costs (1,000s)										
Project First Cost	$0	$58,186	$59,761	$61,336	$62,486	$63,636	$65,536	$67,436	$71,536	$425,000
Interest During Construction	$0	$3,021	$3,102	$3,184	$3,244	$3,304	$3,402	$3,501	$3,714	$0
Total Investment Costs	$0	$61,207	$62,863	$64,520	$65,730	$66,940	$68,938	$70,937	$75,250	$425,000
Capital Recovery Factor	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426
Average Annual Costs	$0	$2,607	$2,678	$2,749	$2,800	$2,852	$2,937	$3,022	$3,206	$18,105
Annual O&M Costs	$0	$387	$387	$387	$387	$387	$387	$387	$387	$0
Total Average Annual Costs	$0	$2,994	$3,065	$3,136	$3,187	$3,239	$3,324	$3,409	$3,593	$18,105
Results										
Annual Net Benefits	**$0**	**$12,758**	**$13,982**	**$14,951**	**$14,966**	**$14,931**	**$14,846**	**$14,761**	**$14,577**	**$65**
Benefit-to-Cost Ratio	**N/A**	**5.26**	**5.56**	**5.77**	**5.70**	**5.61**	**5.47**	**5.33**	**5.06**	**1.00**

Table 28: With-Project Results - USACE Intermediate SLC Scenario

Without-Project Equivalent Annual Flood Damage (1,000s)										
Structure & Content Damage	$15,391									
Relocation Cost	$7,153									
Total	$22,545									
With-Project Equivalent Annual Damages & Damages Reduced (1,000s)										
	No Action	11' Levee	11.5' Levee	12' Levee	12.5' Levee	13' Levee	13.5' Levee	14' Levee	15' Levee	Non-Structural
With-Project Avg Annual Flood Damage	$22,545	$3,894	$1,534	$131	$21	$1	$0	$0	$0	$0
Annual Damages Reduced	$0	$18,650	$21,011	$22,414	$22,524	$22,544	$22,545	$22,545	$22,545	$22,545
Project Costs (1,000s)										
Project Cost	$0	$58,186	$59,761	$61,336	$62,486	$63,636	$65,536	$67,436	$71,536	$425,000
Interest During Construction	$0	$3,021	$3,102	$3,184	$3,244	$3,304	$3,402	$3,501	$3,714	$0
Total Investment Costs	$0	$61,207	$62,863	$64,520	$65,730	$66,940	$68,938	$70,937	$75,250	$425,000
Capital Recovery Factor (CRF)	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426
Average Annual Costs	$0	$2,607	$2,678	$2,749	$2,800	$2,852	$2,937	$3,022	$3,206	$18,105
Annual O&M Costs	$0	$387	$387	$387	$387	$387	$387	$387	$387	$0
Total Average Annual Costs	$0	$2,994	$3,065	$3,136	$3,187	$3,239	$3,324	$3,409	$3,593	$18,105
Results										
Annual Net Benefits	**$0**	**$15,656**	**$17,946**	**$19,278**	**$19,337**	**$19,305**	**$19,221**	**$19,136**	**$18,952**	**$4,440**
Benefit-to-Cost Ratio	**N/A**	**6.23**	**6.86**	**7.15**	**7.07**	**6.96**	**6.78**	**6.61**	**6.28**	**1.25**

Table 29: With-Project Results - USACE High SLC Scenario

Without-Project Equivalent Annual Flood Damage (1,000s)												
Structure & Content Damage	$31,902											
Relocation Cost	$8,293											
Total	$40,195											
With-Project Equivalent Annual Damages & Damages Reduced (1,000s)												
	No Action	10' Levee	10.5' Levee	11' Levee	11.5' Levee	12' Levee	12.5' Levee	13' Levee	13.5' Levee	14' Levee	15' Levee	Non-Structural
With-Project Avg Annual Flood Damage	$40,195	$72,421	$49,111	$29,154	$14,490	$5,071	$1,575	$419	$92	$16	$0	$0
Annual Damages Reduced	$0	-$32,226	-$8,916	$11,040	$25,704	$35,123	$38,619	$39,776	$40,103	$40,178	$40,195	$40,195
Project Costs (1,000s)												
Project Cost	$0	$55,036	$56,611	$58,186	$59,761	$61,336	$62,486	$63,636	$65,536	$67,436	$71,536	$425,000
IDC	$0	$2,857	$2,939	$3,021	$3,102	$3,184	$3,244	$3,304	$3,402	$3,501	$3,942	$0
Total Investment Costs	$0	$57,893	$59,550	$61,207	$62,863	$64,520	$65,730	$66,940	$68,938	$70,937	$75,478	$425,000
Capital Recovery Factor (CRF)	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426
Average Annual Costs	$0	$2,466	$2,537	$2,607	$2,678	$2,749	$2,800	$2,852	$2,937	$3,022	$3,215	$18,105
Annual O&M Costs	$0	$387	$387	$387	$387	$387	$387	$387	$387	$387	$387	$0
Total Average Annual Costs	$0	$2,853	$2,924	$2,994	$3,065	$3,136	$3,187	$3,239	$3,324	$3,409	$3,602	$18,105
Results												
Annual Net Benefits	**$0**	**-$35,079**	**-$11,840**	**$8,046**	**$22,639**	**$31,988**	**$35,432**	**$36,537**	**$36,779**	**$36,770**	**$36,592**	**$22,090**
Benefit-to-Cost Ratio	**N/A**	**-11.29**	**-3.05**	**3.69**	**8.39**	**11.20**	**12.12**	**12.28**	**12.07**	**11.79**	**11.16**	**2.22**

5.2.3 WITH-PROJECT PERFORMANCE STATISTICS

When engineered levees are assumed not to fail before overtopping as they were for this analysis (no geotechnical failure function entered into HEC-FDA), the HEC-FDA program uses the top of levee elevation as the performance criteria. Table 30 below shows the performance statistics for the two levee heights that have been identified as having the greatest net benefits under the various SLC scenarios – 12.5' under the USACE Low and Intermediate scenarios, and 13.5' under the USACE High scenario. The annual exceedance probability (AEP) is the likelihood that the levee will be overtopped, the long-term risk is the risk of overtopping over some defined period of time for a particular water surface profile, and the conditional non-exceedance Probability (CNP, also referred to as "assurance") is the likelihood that the levee would contain a particular exceedance probability event. As an example, Table 30 reports that with a 12.5 ft levee under the USACE Low SLC scenario, the likelihood of overtopping is extremely low. There is just a 2.6% chance over a period of thirty years that the levee would be overtopped once, and the levee has a 99.9% chance of containing the 1% ACE event at the end of the period of analysis. The primary difference in performance statistics can be seen under the USACE High SLC scenario. Table 30 shows that the 13.5 ft levee has substantially lower residual risk by the end of the period of analysis under this scenario than the 12.5 ft levee. For example, the 12.5 ft levee has an AEP of 8.5%, while the 13.5 ft levee has an AEP of only 0.5%. Similarly, the CNP for the 1% ACE event is less than 1% for the 12.5 ft levee, but over 88% for the 13.5ft levee. Since under any of the scenarios sea level is expected to continue rising beyond 2067, these results should be viewed as a single snapshot in time of the risk and not a permanent reality.

Table 30: Project Performance Statistics - 12.5 ft and 13.5 ft Levee, USACE Low, Intermediate, and High SLC Scenario, 2067

SLC Scenario	FRM Option	Mean Annual Exceedence Probability	Long-Term Risk (30 Years)	Conditional Non-Exceedence Probability by Event			
				10%	2%	1%	0.20%
Low	No Action	39.5%	99.9%	36.9%	24.7%	16.2%	3.9%
	12.5' Levee	0.08%	2.6%	99.9%	99.9%	99.9%	94.9%
	13.5' Levee	0.02%	0.7%	99.9%	99.9%	99.9%	99.9%
Intermediate	No Action	53.2%	99.9%	29.6%	7.3%	5.4%	2.5%
	12.5' Levee	0.08%	2.0%	99.9%	99.9%	99.9%	92.6%
	13.5' Levee	0.02%	0.6%	99.9%	99.9%	99.9%	99.9%
High	No Action	94.9%	99.9%	0.3%	<.01%	<.01%	<.01%
	12.5' Levee	8.51%	93.0%	66.7%	3.2%	0.7%	0.0%
	13.5' Levee	0.48%	13.4%	99.9%	98.3%	88.2%	33.5%

5.2.4 SENSITIVITY ANALYSIS – ECONOMIC JUSTIFICATION & LEVEE FAILURE PROBABILITY

As the previous sections show, there is strong economic justification for the construction of a levee to reduce the risk of flooding in the study area. The strong justification is in large part the result of the finding that there is currently a high annual likelihood of flooding in the study area. The most uncertain of the inputs to the estimation of the likelihood of flooding in the study area is the likelihood of failure of the outer dike, which is incorporated in the HEC-FDA model as the without-project levee failure function. Because of the uncertainty associated with the likelihood of outer dike failure, an obvious sensitivity analysis that can be performed is to determine how changes to the levee failure function affect project economic justification. The uncertainty in the levee failure function is greater at the lower elevations, and so for this sensitivity analysis an adjustment was made to the probability of failure near the bottom of the levee – between 7’ and 10’. The probability of failure was set to zero between above 7’ and below 10’. At and above 10’ the probability of failure was unchanged from the expected value curve as described previously. Altering the failure curve at the lower elevation to this degree is simply an adjustment that was made for purposes of understanding the sensitivity of the economic justification to the changes in the failure curve; the adjusted curve is not an alternative to what the USACE engineers consider is the most likely relationship between water elevation and probability of failure. The two curves are shown in Figure 36 below for comparative purposes.

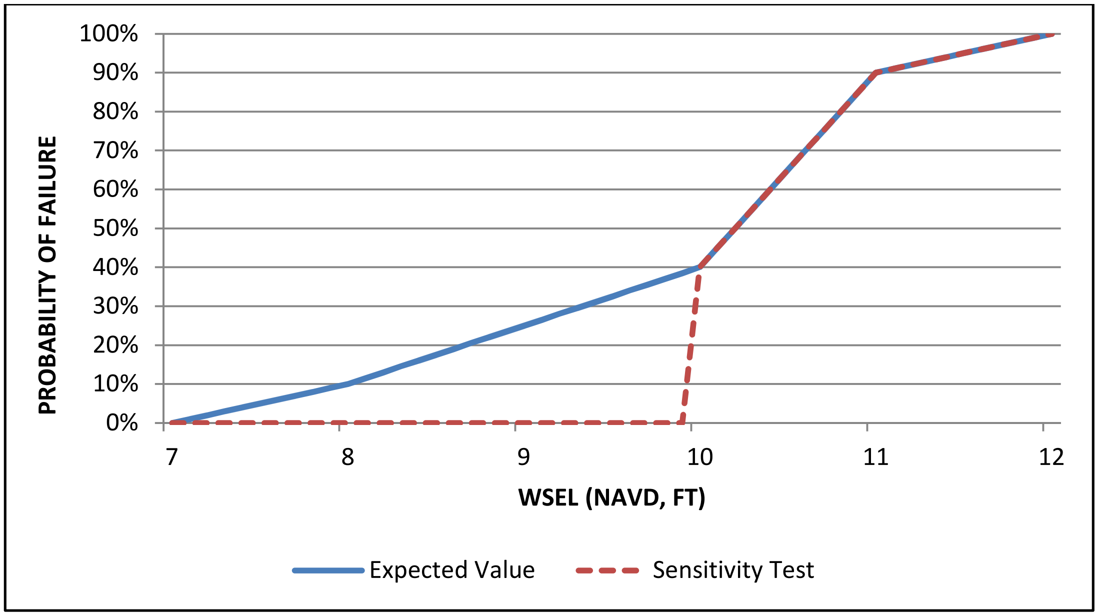

Figure 36: Levee Failure Function Comparison – Economic Justification Sensitivity Test

For the sensitivity analysis, the without-project HEC-FDA models for the USACE Low SLC scenario were altered to include the adjusted levee failure function. As a result of the significant decrease in the risk of flooding, the sensitivity analysis assumed no relocations would occur. The tables below show the results of this sensitivity analysis.

As Table 31 below shows, the change to the failure function reduces the annual likelihood of damage from 32% (see Table 4) at 2017 to 7%. However, an AEP of 7% is still significant and would almost certainly lead to failure over the fifty-year period of analysis.

Table 31: Without-Project Performance with Adjusted Failure Function

Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
	Median	Expected	10	30	50	10%	4%	2%	1%	.4%	.2%
levee	0.0689	0.0727	0.5298	0.8960	0.9770	0.6618	0.3999	0.3567	0.2929	0.1939	0.1311

Table 32: Sensitivity Test Results - Economic Justification

Without-Project Equivalent Annual Flood Damage (1,000s)											
Structure & Content Damage	$8,688										
Relocation Cost	$756										
Total	$9,443										
With-Project Equivalent Annual Damages & Damages Reduced (1,000s)											
	No Action	10.5' Levee	11' Levee	11.5' Levee	12' Levee	12.5' Levee	13' Levee	13.5' Levee	14' Levee	15' Levee	Non-Structural
With-Project Avg Annual Flood Damage	$9,443	$6,244	$2,418	$1,123	$84	$17	$6	$3	$1	$0	$0
Annual Damages Reduced	$0	$3,199	$7,026	$8,320	$9,359	$9,427	$9,438	$9,441	$9,442	$9,443	$9,443
Project Costs (1,000s)											
Project Cost	$0	$56,611	$58,186	$59,761	$61,336	$62,486	$63,636	$65,536	$67,436	$71,536	$425,000
IDC	$0	$2,939	$3,021	$3,102	$3,184	$3,244	$3,304	$3,402	$3,501	$3,714	$0
Total Investment Costs	$0	$59,550	$61,207	$62,863	$64,520	$65,730	$66,940	$68,938	$70,937	$75,250	$425,000
Capital Recovery Factor (CRF)	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426	0.0426
Average Annual Costs	$0	$2,537	$2,607	$2,678	$2,749	$2,800	$2,852	$2,937	$3,022	$3,206	$18,105
Annual O&M Costs	$0	$387	$387	$387	$387	$387	$387	$387	$387	$387	$0
Total Average Annual Costs	$0	$2,924	$2,994	$3,065	$3,136	$3,187	$3,239	$3,324	$3,409	$3,593	$18,105
Results											
Annual Net Benefits	**$0**	**$275**	**$4,031**	**$5,255**	**$6,224**	**$6,240**	**$6,199**	**$6,117**	**$6,033**	**$5,851**	**-$8,662**
Benefit-to-Cost Ratio	**N/A**	**1.09**	**2.35**	**2.71**	**2.98**	**2.96**	**2.91**	**2.84**	**2.77**	**2.63**	**0.52**

As Table 32 above shows, the without-project damages decreased by 50% compared to the damages under the USACE Low SLC scenario when using the expected probabilities for outer dike failure. However, as the results table shows, even under the USACE Low SLC scenario, the levee project is still economically justified with the adjusted failure function. Including some consideration of structure relocations would not materially change the results because of the offsetting effects of repeated flood damages or high relocation cost. These results reflect a lower bound as far as economic justification with a modification to the levee failure function, since they are based upon the low SLC scenario. Since without project damages and with project benefits are higher under the intermediate and high SLC scenarios, conducting this same sensitivity analysis on those scenarios would yield higher net benefits and benefit/cost ratios than those shown in the table above.

6.0 SUMMARY

The existing pond-dike system fronting the community of Alviso, while not composed of engineered structures, has incidentally provided a measure of coastal flood risk reduction to the area. Initial analyses performed for the SSFBS study indicated a Federal interest for coastal flood risk reduction largely based on the USACE High SLC scenario; however, additional work was deemed necessary to demonstrate Federal interest under all three SLC scenarios that must be analyzed in USACE studies. In order to comply with EC 1165-2-212 (USACE, 2011) and ER 1100-2-8162 (USACE, 2013a) Federal interest, as determined solely by NED outputs, was also demonstrated for the USACE Low and Intermediate SLC scenarios in subsequent analyses (summarized in this report). The current flood risk analysis is policy compliant and provides results useful for the SSFBS study that can be incorporated into the Integrated Interim Feasibility Study and Environmental Impact Statement/Report. Without-project analysis results indicate that there currently is a high probability of failure of the existing dike-pond system, and that the risk increases over time with a rise in relative sea level. In 2017 the annual chance of a damaging flood event is estimated to be 32%, and by 2067 the annual chance is estimated to be as high as 53% for the USACE Intermediate SLC scenario. The with-project results for the three USACE SLC scenarios show positive net benefits under each scenario, ranging from approximately $15 million to $37 million in annual net benefits. All structural projects considered have strong economic justification under each of the three SLC scenarios considered (the benefit to cost ratios range from approximately 4 to 12). The optimum levee heights based on annual net benefits for the three SLC scenarios are 12.5 feet under the USACE Low and Intermediate SLC scenarios, and 13.5 feet under the USACE High SLC scenario.

The current probability of exceeding the existing the dike-pond system's capacity to alleviate flood risk (as described above) may seem high given the fact that there have been no historical coastal flood events in the study area. Given the water elevations in the bay near the existing outboard dike, and given the low elevation of the community of Alviso and surrounding area, it is clear that the dike-pond system has been providing the community of Alviso with some level of coastal flood risk reduction. The San Francisco District believes that the updated Coastal Engineering and Geotechnical Engineering analyses presented in this report have made reasonable assumptions and changes from previous analyses conducted under the SSFBS study. These changes include the use of extreme water level statistics from the DTM for the Coyote Creek gage, uncertainty estimate for the extreme water level statistics, interior-exterior relationship for the outboard and inboard dikes, outboard dike failure mechanisms, levee fragility curve, and inboard dike performance. Perhaps the most significant of the changed assumptions is for the inner dike, where in the initial analysis reported at the USACE Alternative Formulation Briefing milestone for this study it was assumed that the inner dike would not breach, but only be overtopped. The current analysis assumes the inner dike will breach due to overtopping. These assumptions and changes are based on USACE policy, accepted standard practices, and best engineering judgment and represent the best estimates for these parameters. A sensitivity analysis on the dike fragility curve indicates that positive net benefits are still obtained even under more stout assumptions about dike performance.

In conclusion, the flood risk analysis presented in this report meets USACE policy, follows accepted practices, and represent reasonable best estimates. These results, combined with results from other analyses and criteria, can be used to determine Federal interest, evaluate plans, and select a recommended plan. The project delivery team has moved forward with the existing analysis and has acknowledged the risks that remain in communicating flood risk via the flood damage analysis for the existing without and FWOP condition. The reduced level of confidence in the reported AEP can be attributed to multiple factors that define the complexity of the dike-pond system and floodplain in the study area. While an AEP of 32% may appear to substantially overstate the flood risk for the study period, there is reason to believe that existing flood risk to the Alviso economic impact area is fundamentally different than what past performance would otherwise indicate.

7.0 REFERENCES

Deering. (2014). Personal communication.

FEMA. (2005). Full Data Module for Benefit-Cost Analysis of Riverine Hazard Mitigation Projects, Mitigation BCA Toolkit CD, Version 2.0, January 2005'.

Flick, R. E. (2000). Time-of-day of peak tides in a mixed-tide regime. *Shore & Beach*, 15-17.

Geomatrix. (2006). *South Bay Salt Pond Restoration Project Levee Assessment.* Oakland, CA: Geomatrix.

Henderson, F. M. (1966). *Open channel flow.* New York, NY: MacMillian Publishing Comoany.

Knuuti, K. (1995). *Extreme tide prediction for San Francisco Bay.* Berkeley, CA: University of California.

MacWilliams, M., Kilham, N. E., & Bever, A. J. (2012). *South San Francisco Bay Shoreline Study, South San Francisco Bay Long Wave Modeling Draft Report.* San Francisco, CA: Delta Modeling Associates.

NRC. (2012). *Sea-Level Rise for the Coasts of California, Oregon, and Washington: Past, Present, and Future.* Washington, DC: National Research Council, The National Acadamies Press.

PWA. (2007). *South Bay Salt Pond Restoration Project Flood Analyses Report.* San Francisco, CA: Phillip Williams and Associates.

USACE. (1984). *Tidal Stage versus Frequency Study.* San Francisco, CA: San Francisco District.

USACE. (1997). *Life Cycle Design and Performance, ER 1110-2-8159.* Washington, DC: Headquarters.

USACE. (2008). *Erosion-induced Breaching: Reliability Assessment of San Francisco South Bay Salt Pond Levee.* Geotechnical and Structures Laboratory.

USACE. (2009a). *Appendix C: Geotechnical Investigation and Analysis South San Francisco Bay Shoreline Study F3 Milestone Without Project.* San Francisco, CA: San Francisco District.

USACE. (2009b). *Reliability Assessment of San Francisco South Bay Salt Pond Inboard Levees.* Vicksburg, MS: Geotechnical and Structures Laboratory.

USACE. (2010). *Standards and Procedures for Referencing Project Evaluation Grades to Nationwide Vertical Datums, EM 1110-2-6056.* Washington, DC: Headquarters.

USACE. (2011). *SEA-LEVEL CHANGE CONSIDERATIONS FOR CIVIL WORKS PROGRAMS, EC 1165-2-212.* Washington, DC: Headquarters.

USACE. (2013a). *INCORPORATING SEA LEVEL CHANGE IN CIVIL WORKS PROGRAMS, ER 1100-2-8162.* Washington, DC: Headquarters.

USACE. (2013b). *Geotechnical Appendix to South San Francisco Bay Shoreline Study Alternative Formulation Briefing.* San Francisco, CA: San Francisco District.

USACE. (2014). Memorandum for Record: South San Francisco Bay Shoreline Study Supplemental Analyses on Sea Level Change and flood risk associated with US Fish & Wildlife Service's (FWS) Refuge Lands, dated 12 Mar 2014. San Francisco, CA: San Francisco District.

USACE. (2014a). *Geotechnical field assessment of the San Francisco South Bay dike system.* Richard Olsen.

Walters, R. A., Cheng, R. T., & Conomos, T. J. (1985). Time scales of circulation and mixing processes of San Francisco Bay waters. *Hydrobiologia*, 13-36.

Zervas, C. (2009). *Sea Level Variations of the United States 1854–2006, NOAA Technical Report NOS CO-OPS 053.* Silver Spring, MD: NOAA.

Zervas, C. E. (2005). Response of Extreme Storm Tide Levels. *Oceans, Proceedings of the MTS/IEEE* (pp. 2501-2506). Wasnington DC: MTS/IEEE.

Zervas, C., & Sweet, W. (2014). Calculation of Exceedance Probability Statistics Using NOAA's Historical Water Levels . New York City : USACE Workshop on Extreme Water Levels .

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