SAN FRANCISCO WATERFRONT COASTAL FLOOD STUDY, CA

APPENDIX B.1 – HYDROLOGY, HYDRAULICS, AND COASTAL [DRAFT]

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USACE TULSA DISTRICT | THE PORT OF SAN FRANCISCO

US Army Corps of Engineers

PORT OF SAN FRANCISCO

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Acronyms and Abbreviations

Acronym	Definition
°F	Degree(s) Fahrenheit
2D	two-dimensional
AEP	Annual Exceedance Probability
CCSF	City and County of San Francisco
cfs	cubic feet per second
CSD	Combined Sewer Discharge
DEM	Digital Elevation Model
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
FWP	Future With Project
FWOP	Future Without Project
HEC-RAS	Hydrologic Engineering Center-River Analysis System
LiDAR	Light Detection and Ranging
LOD	Line of Defense
MGD	Million Gallons Per Day
MHHW	Mean Higher High Water
MLLW	Mean Lower Low Water
NAD83	North American Datum of 1983
NAVD88	North American Vertical Datum of 1988
NOAA	National Oceanic and Atmospheric Administration
OPC	California Ocean Protection Council
PED	Preconstruction Engineering and Design

POSF	Port of San Francisco
SFPUC	San Francisco Public Utilities Commission
SFPW	San Francisco Public Works
SFWCFS	San Francisco Waterfront Coastal Flood Study
SLC	Sea Level Change
SWEL	Stillwater Elevation
TNBP	Total Net Benefits Plan
TSP	Tentatively Selected Plan
TWL	Total Water Level
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
WCE	Wave Crest Elevation
WRDA	Water Resource Development Act

Section B.1-1 Introduction

This report is written as an appendix to the San Francisco Waterfront Coastal Flood Study (SFWCFS). The text summarizes the water resource engineering analyses required to support the planning and Federal interest determination of a coastal resilience civil works project in San Francisco, California along San Francisco Bay. The water resource engineering analysis span over two decades of effort and some analyses have been previously released to the public. Where analyses have been previously released to the public, they are referenced in this report as appropriate. Analyses that have not been previously released to the public are included in the main text of this report or in a Sub-Appendix where applicable.

This report is organized into sections that provide the water resources engineering analyses in a logical order to support the project planning process. The major sections are organized by the preliminary engineering evaluations for the hydrology, hydraulic and coastal components. This includes the evaluation of coastal hazards, presentation of inundation maps based on flooding from coastal hazards, sensitivity assessment of wave overtopping assumptions, assessments of interior drainage and shallow groundwater impacts, and overview of induced Baywide conditions. Within each major section, a discussion of Existing Condition, Future Without Project (FWOP) Condition, and Future With Project (FWP) Condition is presented. Additional detail has been included in sub-appendices to this report. These sub-appendices are referenced in this report and provided under their own separate covers due to their size.

Discussions of considerations related to use of nature and natural based features and to climate change are presented in separate appendices.

Section B.1-2 Current Watershed Conditions

San Francisco is located on the Central California coastline, on the northern tip of a peninsula, just south of the Golden Gate – the connection between the Pacific Ocean and the Bay (**Figure B.1-1**). The Bay Area has a variable climate that is dominated by many large-scale atmospheric and oceanic processes. Although generally characterized by a mild Mediterranean climate with dry summers and cool, wet winters, the Bay Area is also a region that experiences volatile storms that can cause widespread flooding in low-lying coastal areas.

B.1-2.1 San Francisco Shoreline

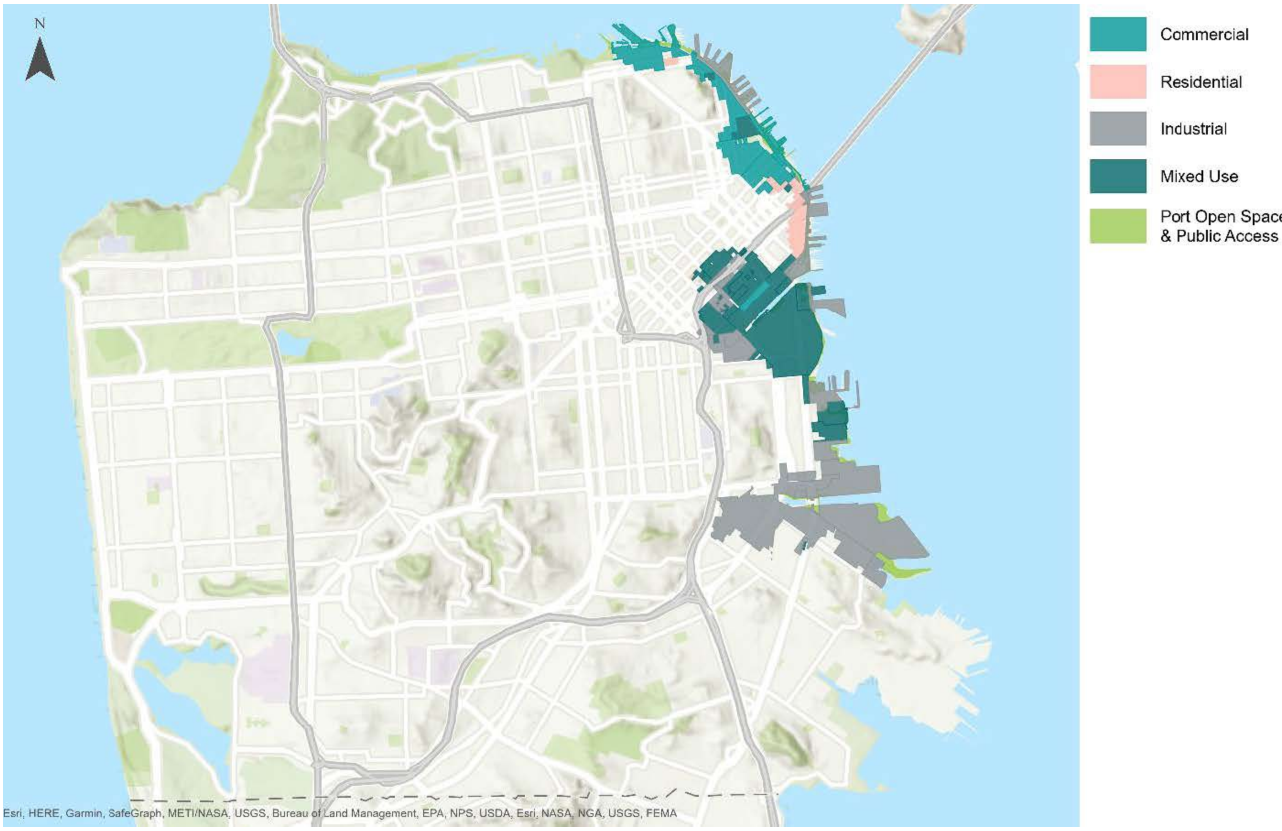
The Port of San Francisco (POSF) manages 7.5 miles of shoreline along the San Francisco waterfront from Aquatic Park near the Golden Gate to Heron’s Head Park (**Figure B.1-2**). Much of the northern shoreline (i.e., north of the San Francisco Giants ballpark) is engineered with bulkhead wharves and finger piers, while the southern shoreline includes two inlets (Mission Creek and Islais Creek), working piers (Piers 80 –

96), and areas with sensitive habitat such as the Pier 94 wetlands and Heron's Head Park. Much of the areas inland from the shoreline are built on reclaimed land (Bay fill) that was filled over time to support the construction of the historic Embarcadero seawall in the late 1800s, and the ship building industries that supported the World Wars in the early 1900s (**Figure B.1-3**). This man-made shoreline is relatively flat, with a mean elevation of approximately 11.8 feet North American Vertical Datum of 1988 (NAVD88) (**Figure B.1-4**). Therefore, when Bay waters overtop the shoreline, the entire shoreline can quickly be overtopped.

Figure B.1-5 presents the existing Federal Emergency Management Agency (FEMA) floodplain, which was analyzed and mapped relative to 2008 ocean and Bay water levels. The POSF shoreline area is currently mapped as Zone D, which indicates an existing but unquantified flood risk. The areas inland of the shoreline are high-density urban and industrial areas. Businesses and residents are located within the existing FEMA floodplain along Islais Creek, and substantially more structures and infrastructure are located within areas that could be flooded if sea level rise trends along the higher projections. The City and County of San Francisco (CCSF) requires that all capital projects within the Sea Level Rise Vulnerability Zone (**Figure B.1-6**), an area that could be inundated by a 1% annual chance coastal flood coupled with 66 inches of sea level rise (relative to the year 2000 water levels), consider sea level rise adaptation as part of the project planning and design process (CPC 2020). The CCSF also completed a comprehensive Sea Level Rise and Consequence Assessment which includes exposure, vulnerability, and consequence information for transportation, wastewater and stormwater, water, energy, parks, and open space, and POSF assets within the Sea Level Rise Vulnerability Zone (CCSF 2020).

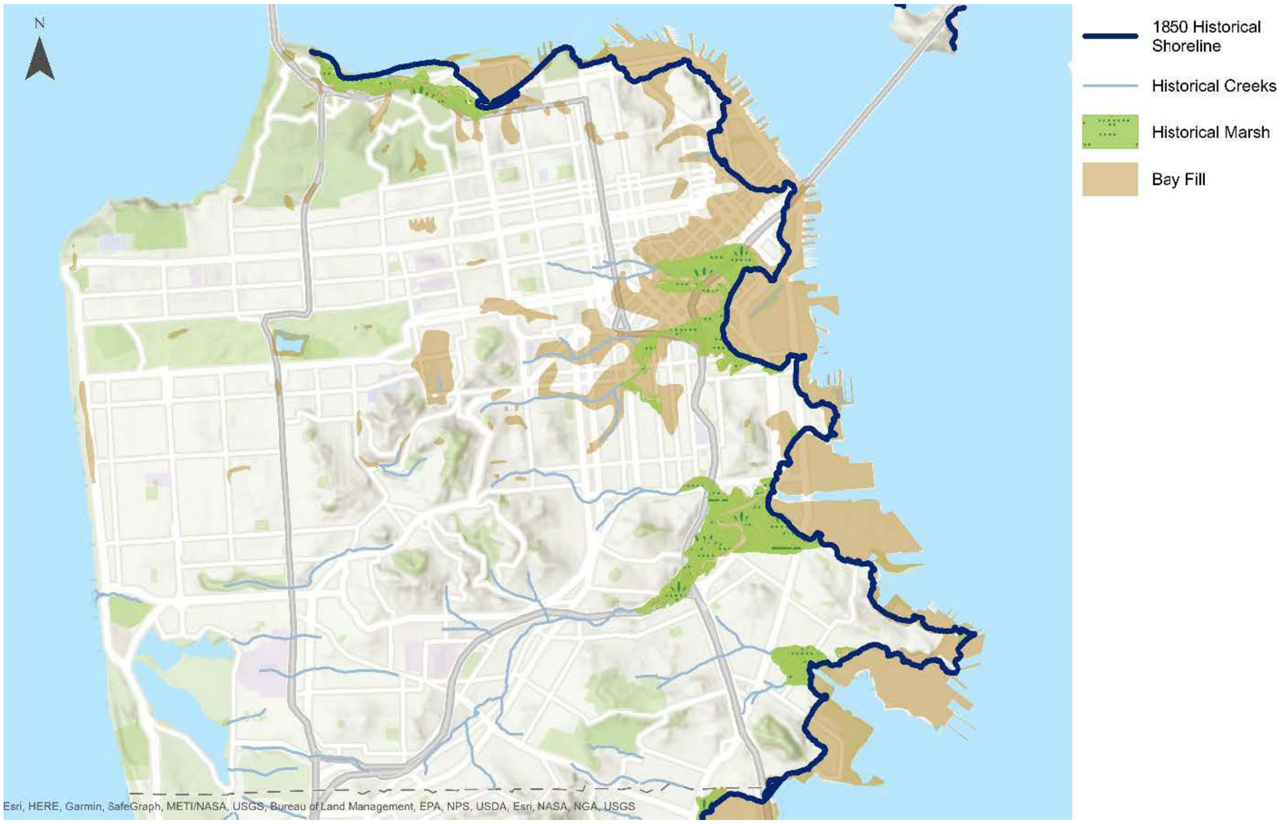
Source: (May et al. 2016b)

Figure B.1-1: San Francisco Bay Coastal and Estuarine System



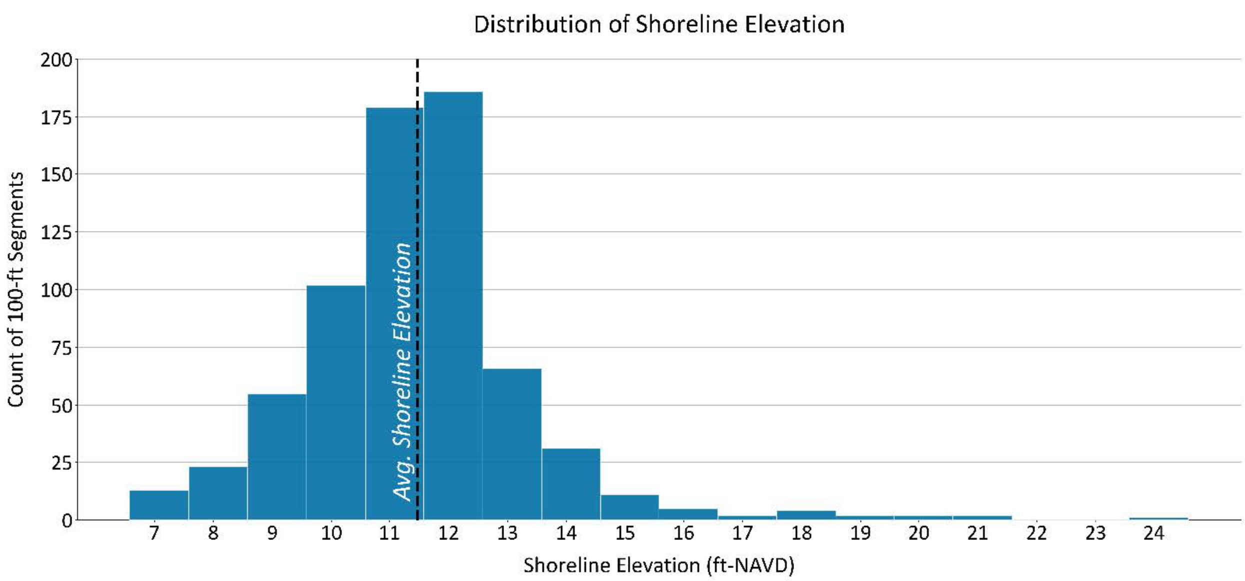
Source: (SF Planning 2019)

Figure B.1-2: Port of San Francisco Land Use



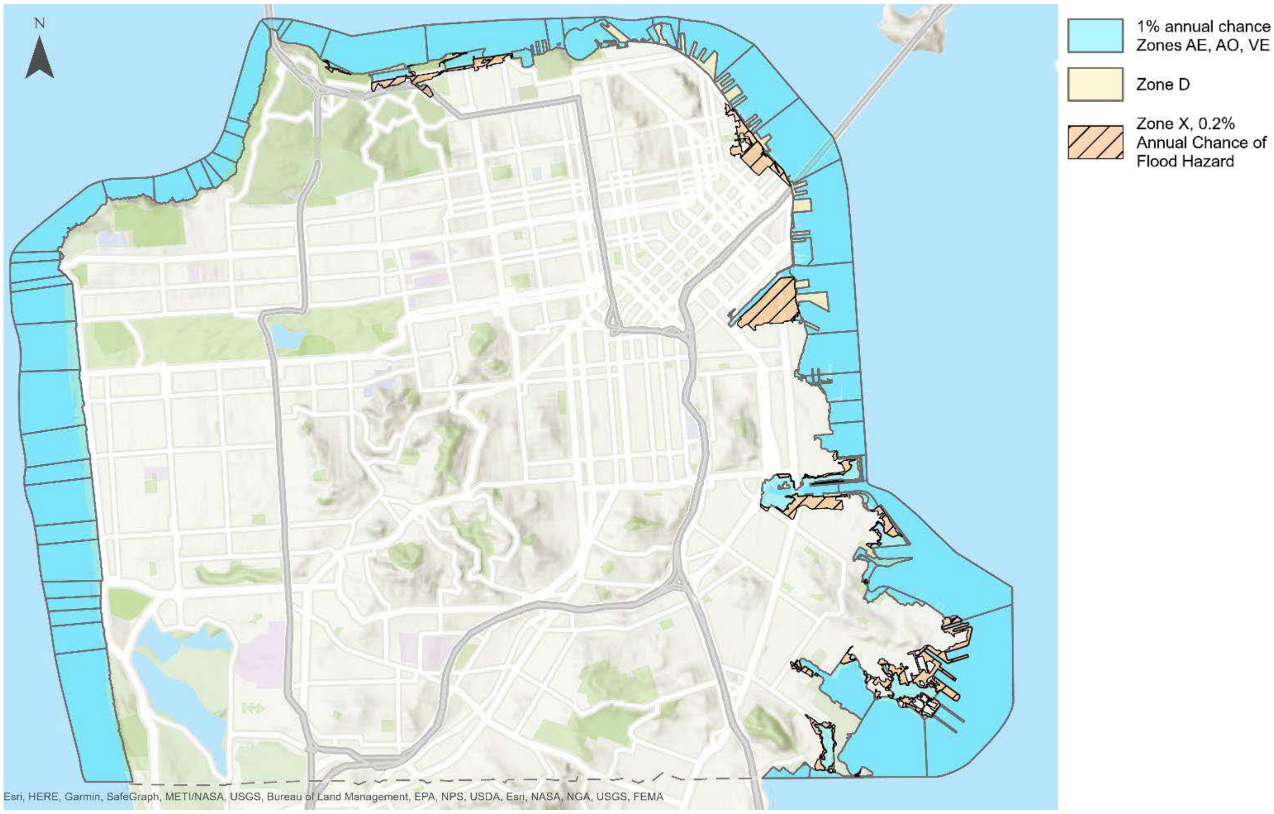
Source: (SFEI 1998)

Figure B.1-3: Historic Shoreline and Area of Reclaimed Land Built on Bay Fill



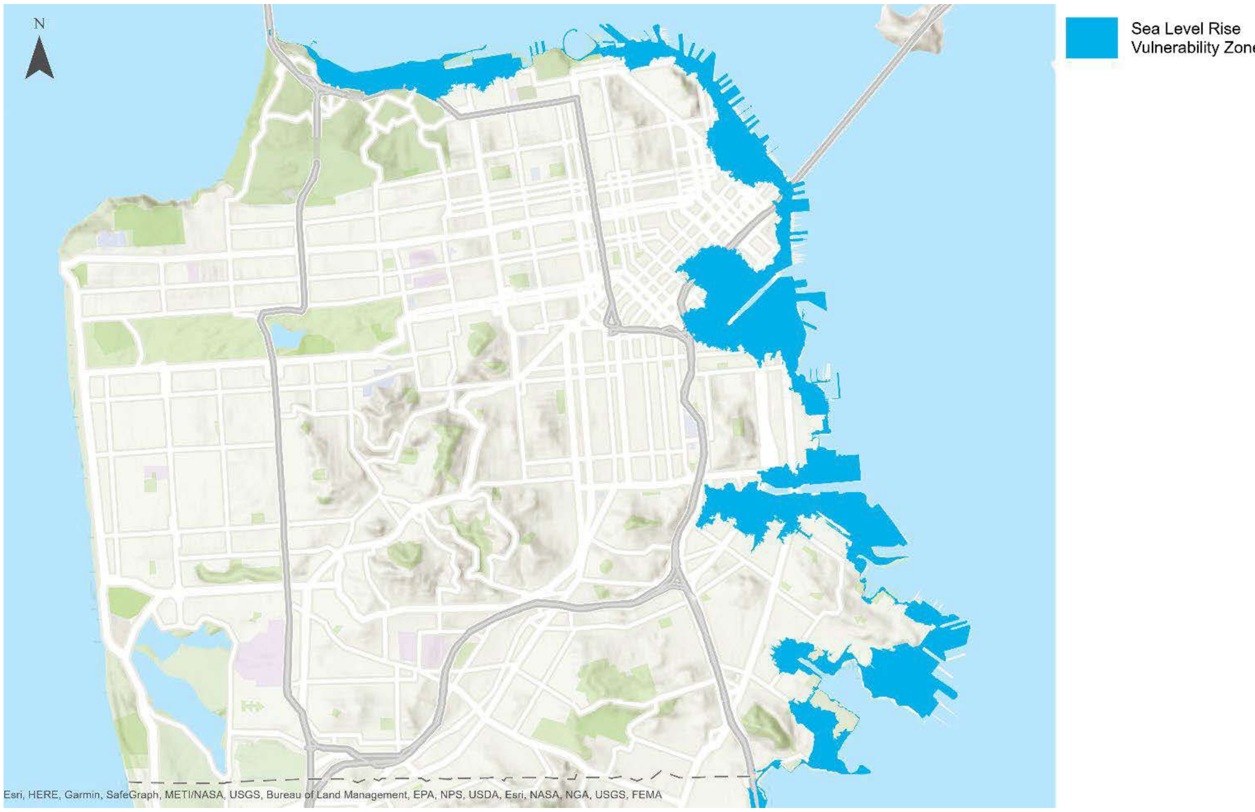
Source: (Port of San Francisco 2021a)

Figure B.1-4: Elevation of the Port Shoreline



Source: FEMA 2021

Figure B.1-5: FEMA 1% Annual Chance Coastal Floodplain (relative to 2008)



Source: (CPC 2020)

Figure B.1-6: San Francisco Sea Level Rise Vulnerability Zone (relative to 2000)

The Sea Level Rise Vulnerability Zone encompasses the area that could be inundated by a 1% annual chance coastal flood coupled with 66 inches of sea level rise (relative to the year 2000 water levels).

B.1-2.2 San Francisco Watershed

The City of San Francisco is divided into two primary drainage basins, the Westside Basin, which drains to the Pacific Ocean, and the Bayside basin which drains into San Francisco Bay. The two drainage basins are divided into eight urban watersheds, five of which are on the Bayside. The watershed for San Francisco area is shown on **Figure B.1-7**.

Figure B.1-7: San Francisco Watershed and Urban Drainage Areas

Section B.1-3 Physical Data and Numerical Models

B.1-3.1 Climate

San Francisco has Mediterranean climate with mild rainy winters and warm dry summers. San-Francisco is located on the peninsula of same and surrounded with water on three sides thus weather is influenced by cold currents. Average temperature and precipitation data are shown on **Figure B.1-8** and in **Table B.1-1**.

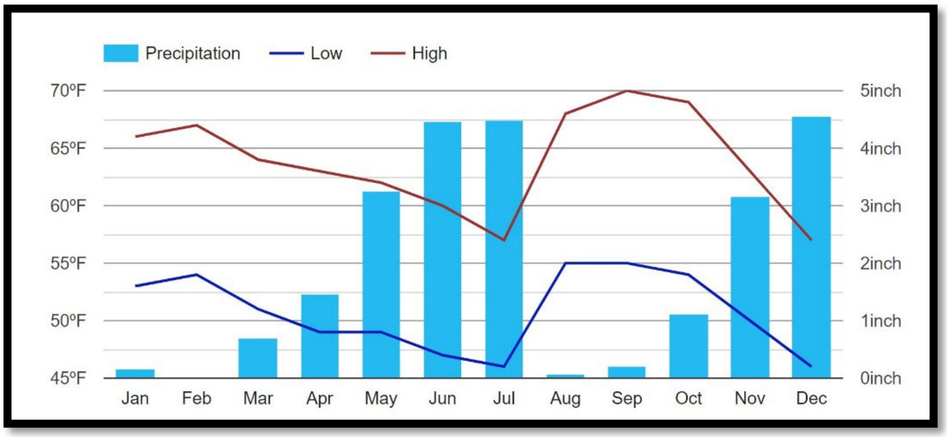


Figure B.1-8: San Francisco Annual Average Temperature and Precipitation Values

Table B.1-1: San Francisco Temperature and Precipitation Values

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Average high in ºF:	66	67	64	63	62	60	57	68	70	69	63	57
Average low in ºF:	53	54	51	49	49	47	46	55	55	54	50	46
Days with precipitation	2	1	4	6	10	11	11	1	1	4	7	10
Hours of sunshine	330	300	314	281	251	182	165	272	267	243	189	156
Av. precipitation in inch	0.16	0.01	0.70	1.46	3.26	4.46	4.50	0.06	0.21	1.12	3.16	4.56

Source: *https://www.usclimatedata.com/climate/san-francisco/california/united-states/usca0987*

Note:

ºF = degree(s) Fahrenheit

B.1-3.2 Winds

The wind climate above the Bay and the larger Bay Area is highly variable, and the steep topography, hills, and valleys throughout the Bay Area drive complex local wind patterns. Strong windspeeds in almost any direction will impact a section of the Bay shoreline. However, due to the orientation of the San Francisco shoreline, the most impactful winds are (1) easterly (i.e., offshore) winds that can impact the shoreline from the Ferry Building and southward, (2) north and northeasterly winds that can impact the Northern Waterfront, and (3) southeasterly winds that can impact the Southern Waterfront.

The strongest winds of the year occur during spring (i.e., March, April, and May). Summer (i.e., June, July, and August) winds are generally lighter with a persistent northwest direction, referred to as onshore flow or a sea breeze that is driven by the daytime heating over land. In fall (i.e., September, October, and November), the pressure gradients lose their strength and windspeeds are reduced over the ocean and the Bay. Wind directions are most variable in the winter months, and wind-driven waves can impact shorelines across the Bay.

B.1-3.3 Gage Records

B.1-3.3.1 Tidal Gages

The Presidio Tide Gage is Station 9414290. Located near the Golden Gate Bridge, the Presidio Tide Gage is the oldest continually operating tidal gage in the Western Hemisphere with the first bit of data being recorded on June 30, 1854 (**Figure B.1-9**).

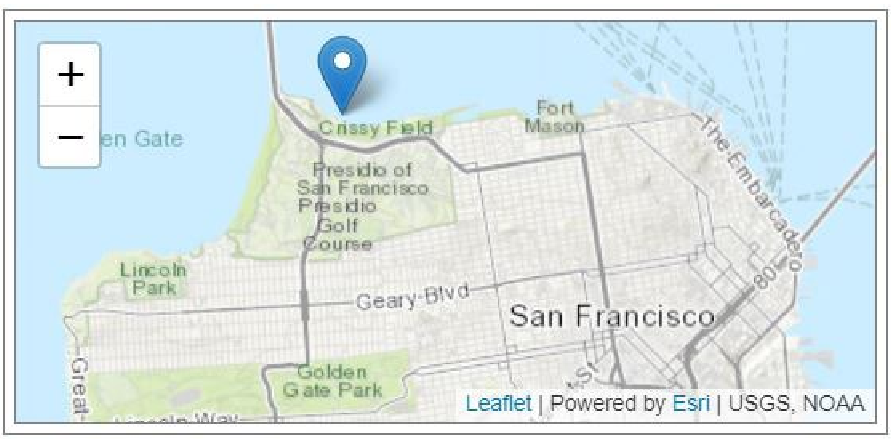


Figure B.1-9: Location of NOAA Gage 9414290

Datum information provided by National Oceanic and Atmospheric Administration (NOAA) on the Tides and Currents website indicates a normal tidal range is about 5.8 feet. At this location, 0-foot NAVD88 is only 0.06 foot below the mean lower low water (MLLW) tidal datum associated with the 1983-2001 tidal epoch (**Figure B.1-10**). The relationship between the tidal datums and NAVD88 varies throughout the Bay, with

MLLW decreasing and mean higher high water (MHHW) increasing to the south, as shown relative to the Alameda tide gage. (https://tidesandcurrents.noaa.gov/stationhome.html?id=9414290)

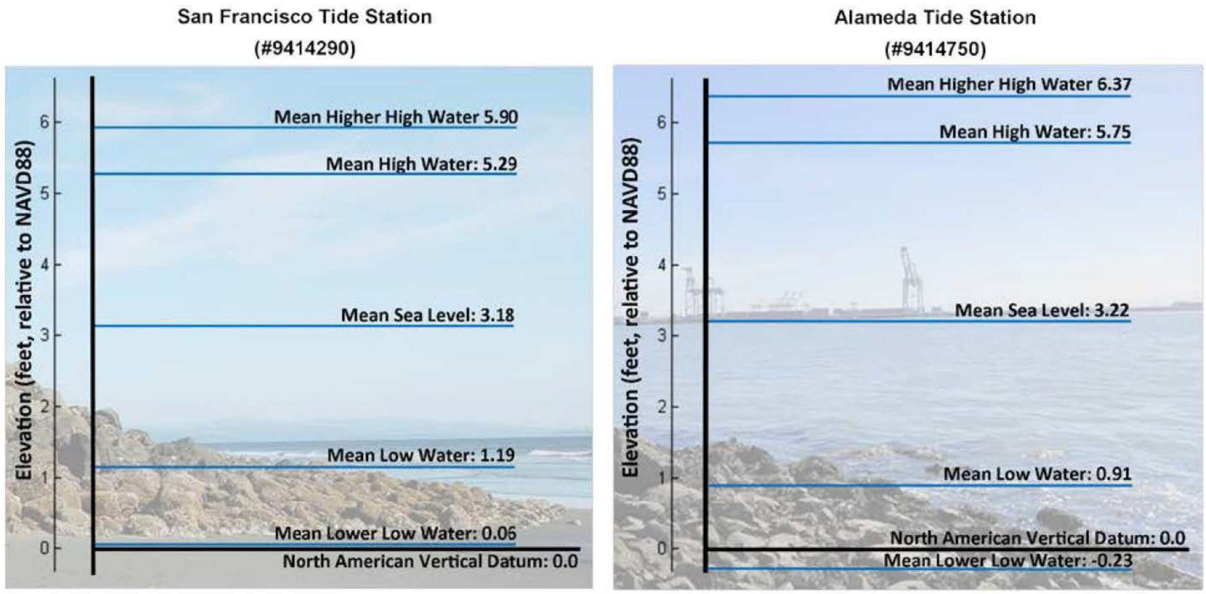


Figure B.1-10: Comparison of San Francisco Bay Tidal Datums and NAVD88

A 118-year period of record (1901-2018) from the San Francisco Tidal gage (9414290) is used for the interior drainage analysis.

B.1-3.3.2 Precipitation Gage

Downtown San Francisco NOAA gage (COOP:047772) is used for the assessment. A 110-year period of record (1908-2018) is used for the precipitation gage. Discussion of future precipitation trends is included in *Appendix J: Climate*.

B.1-3.4 Horizontal and Vertical Datums

Horizontal datum for this study is tied to the State Plane Coordinate System using North American Datum of 1983 (NAD83) (California Zone 3). Distances are in feet by horizontal measurement. The vertical datum for this study is tied to NAVD88, a requirement of ER 1110-2-8160. Elevations are in feet.

B.1-3.5 Numerical Models

B.1-3.5.1 Coastal Models

Coastal hydraulic data for estimating coastal inundation frequency and consequences were taken from model data completed by FEMA to update Coastal Flood Insurance Rate Maps (FIRMs). The previous U.S. Army Corps of Engineers (USACE) San

Francisco Bay study coastal flood hazard mapping was based on the 1984 San Francisco Bay Tidal Stage vs. Frequency study (USACE, 1984) and did not include any wave hazard analysis. In 2004, FEMA initiated a detailed coastal engineering analysis that was used to update the Coastal FIRMs for San Francisco that became effective on March 23, 2021.

The FEMA modeling relied on a regional MIKE21 Flow and Spectral Wave hydrodynamic and wave dynamic numerical model of the Bay to develop a 31-year continuous timeseries of water levels and waves (DHI 2011, 2013). The numerical modeling effort of the FEMA study underwent independent technical review by USACE staff and BakerAECOM (a FEMA subcontractor).

The high-fidelity numerical modeling output that provided the foundation for the updated FEMA FIRMs is well suited for providing the coastal storm inputs required for G2CRM.

B.1-3.5.2 Hydrology and Hydraulics Models

USACE Engineer Regulation 1165-2-21 states “In urban or urbanizing areas, provision of a basic drainage system to collect and convey the local runoff to a stream is a non-Federal responsibility. This regulation should not be interpreted to extend the flood damage reduction program into a system of pipes traditionally recognized as a storm drainage system.”

However, the Water Resource Development Act (WRDA) of 2022, Section 8106 expanded the scope of feasibility studies, such that:

> “In carrying out a feasibility study for a project for flood risk management or hurricane and storm damage risk reduction, the Secretary, at the request of the non-federal interest for the study, shall formulate alternatives to maximize the net benefits from the reduction of the comprehensive flood risk within the geographic scope of the study from the isolated and compound effects of… (4) a rainfall event of any magnitude or frequency”. Additionally, WRDA of 2020, Section 203 amended by WRDA of 2022, Section 8325 indicates that the “Secretary shall expedite the completion of the following feasibility studies, as modified by this section… (1) San Francisco Bay, California – The study for flood risk reduction authorized by section 142 of the Water Resource Development Act of 1976 (90 Stat. 2930), is modified to authorize the Secretary to - (A) investigate the bay and ocean shorelines of …San Francisco… for the purposes of providing flood protection against tidal and fluvial flooding; … (C) with respect to the bay and ocean shorelines, and streams running to the bay and ocean shorelines, of … San Francisco…, investigate the effects of proposed flood protection and other measures or improvements on - (i) the local economy; (ii) habitat restoration, enhancement, or expansion efforts or opportunities; (iii) public infrastructure protection and improvement; (iv) stormwater runoff capacity

and control measures, including those that may mitigate flooding; (v) erosion of beaches and coasts; and (vi) any other measures or improvements relevant to adapting to rising sea levels."

While the storm drainage system is not a primary Coastal Flood Risk Management responsibility, any impacts to the interior hydrology due to sea level rise and the proposed project have to be evaluated and mitigated to the extent justified under USACE policy, if necessary. The San Francisco Public Works (SFPW) has an integrated catchment-combined storm sewer model for the entire study area (InfoWorks). The CCSF Sewer System Master Plan InfoWorks ICM model is the planning and operations model utilized by the San Francisco Public Utilities Commission (SFPUC) and SFPW entities. The model utilizes the InfoWorks ICM software that calculates the hydrology and hydraulics of the combined sewer system and overland flow. The model consists of two linked components, the urban rainfall-runoff hydrology model and they hydraulic network conveyance model and the rainfall-runoff surface hydrology calculations which use the EPA-SWMM5 computation engine that is incorporated into the ICM software.

For this study, USACE developed a Hydrologic Engineering Center-River Analysis System (HEC-RAS) two-dimensional (2D) model using watershed features from the CCSF's Infoworks model. The USACE HEC-RAS model was used for rainfall and for estimating drywell/pump system requirements.

Section B.1-4 Coastal Hydraulics

B.1-4.1 Water Levels

A thorough assessment of coastal water levels used for the coastal hazards analysis is presented in *Sub-Appendix B.1.1: Coastal Extreme Water Levels and High Tide Flooding*. The following is a summary of water level data expected for current conditions and for FWOP conditions with sea level rise. For this discussion, total water level (TWL) elevations, which include a combination of water levels, wave hazards (ocean swell and wind-driven waves) and increased still water elevations resulting from sea level rise, are a useful metric.

B.1-4.1.1 Current Conditions

Most analyses of flooding and flood-related damage and loss focus on extreme events with relatively rare occurrence frequencies, such as the 1-percent-annual-chance flood event (i.e., an event with a 1-percent annual exceedance probability [AEP], or 1% AEP). The coastal storm inputs developed for G2CRM include coastal events that range from the monthly water level (99.9994% AEP) to the annual (1-year) water level events (63.2% AEP) to the 100-year extreme water level (1% AEP). These events are considered very frequent to frequent to rare based on their recurrence interval (**Table B.1-2**). For San Francisco, even the difference between the 10-year and the

100-year return frequency is less than 12 inches (**Table B.1-2**). Events with a larger recurrence interval (lower return frequency), such as the 200-year (0.5% AEP) or 500-year (0.2% AEP) event are considered very rare and were not modeled in the coastal storm database for G2CRM.

Table B.1-2: Frequency of Water Levels or Events (relative to 2000)

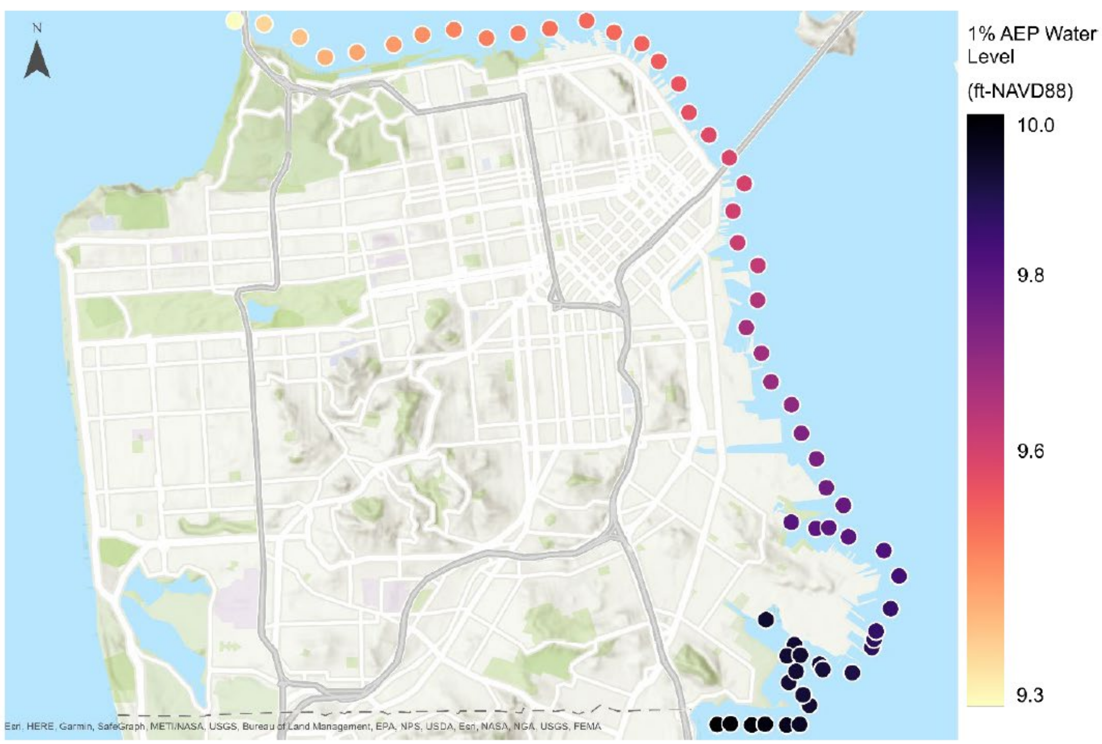
Frequency	EY[b]	AEP	Recurrence	Presidio Water Level (feet NAVD88)[a]	
				1900 - 2020	1970 - 2020
Very Frequent	12	99.999386%	1-month	6.87	6.91
	6	99.75%	2-month	6.98	7.01
	4	98.17%	3-month	7.04	7.09
	3	95.17%	4-month	7.12	7.17
	2	86.47.%	6-month	7.23	7.28
Frequent	1	63.21%	1-year	7.42	7.47
	0.5	39.35%	2-year	7.62	7.67
	0.2	18.13%	5-year	7.88	7.95
	0.1	9.52%	10-year	8.09	8.18
Rare	0.04	3.92%	25-year	8.36	8.48
	0.02	1.98%	50-year	8.57	8.73
	0.01	1.00%	100-year	8.78	8.98

[a] Water Levels (feet NAVD88) are calculated for the Presidio tide gage, baselined to 2000
[b] EY = Average number of exceedances per year

Except for King Tides, which are predictable astronomical tides, extreme water levels represent a temporary, short-term (hours to months) increase in sea level above the predicted astronomical tide level. This difference in water elevation between the predicted and observed tides may include storm surge, El Niño and/or Pacific Decadal Oscillation cycles, local wind setup, freshwater inflows, or a combination of these factors. Observations of extreme water level at tide stations typically do not include short-term wave effects, although wave effects can also influence water levels at the shoreline. Because of the absence of wave effects, the extreme water level elevation is also referred to as the stillwater elevation (SWEL). An extreme water level with a 1% annual chance of occurring may be referred to as the 100-year extreme water level elevation, the 100-year SWEL, the 1%-annual-chance SWEL, or the 1% AEP. For consistency in terminology, this report uses the term AEP when referring to water levels that exceed average annual maximum values. Extreme water levels used in this assessment range from the 50% (2-year) AEP to the 1% (100-year) AEP.

The 1% AEP TWL typically consists of a Bay water level below the 1% AEP water level coupled with a companion wave height that is smaller than the 1% AEP wave height. In other words, the peaks are not coincident as they are driven by separate forcings – the 1% AEP TWL does not equal the 1% AEP water level plus 1% AEP wave height. Offshore, the 1% AEP TWL is better characterized as the 1% AEP wave crest elevation (WCE), so as not to confuse it with the TWL calculated directly at the shoreline that includes the additional component of wave runup. In most cases, the 1% AEP TWL directly at the shoreline (with wave runup) is greater than the offshore 1% AEP WCE. Wave runup expected at idealized shoreline types is discussed further in *Sub-Appendix B.1.3: Wave Overtopping Sensitivity Assessment*.

Figure B.1-11 presents the local variation in the 1% AEP water levels along the San Francisco shoreline, with a similar 0.5-foot difference observed between Aquatic Park and Heron's Head Park.



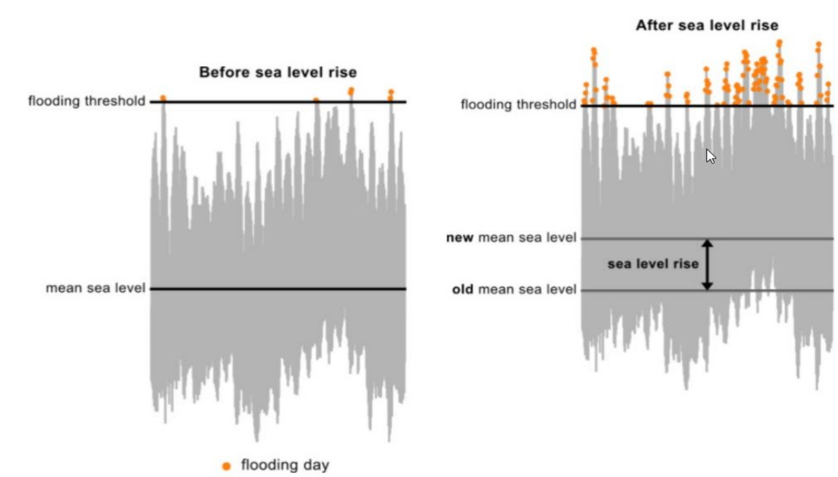
Source: (DHI 2011, 2013, May et al. 2016b)

Figure B.1-11: San Francisco Variation in 1% AEP Water Level (relative to 2008)

B.1-4.1.2 Future Without Project Conditions (With Sea Level Rise)

In the Bay, the difference between MHHW and the 1% AEP coastal water level is on the same order of magnitude as future sea level rise by the year 2100. Future flooding by high frequency events could result in more damage and disruption to shoreline communities and infrastructure than lower frequency events (Sweet et al. 2016, 2018, Ghanbari et al. 2019, Taherkhani et al. 2020). High frequency events include very frequent events (such as the 6-month to 1-month water level) and near daily events or high tide flooding.

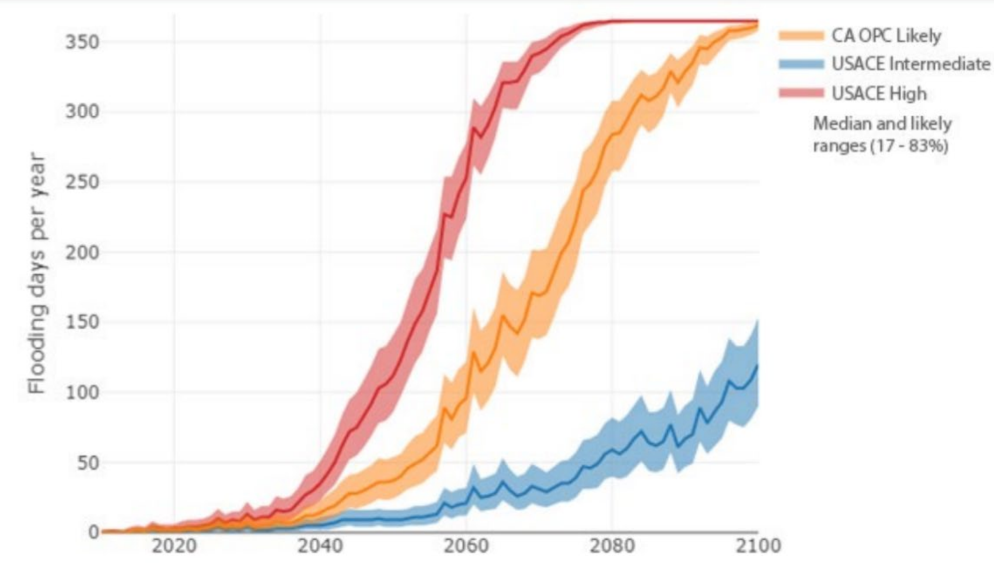
For example, if sea level rises by 6 inches, a 1% AEP water level (100-year water level) will become similar to the current 4% AEP water level (about 25-year water level) in the Bay (Vandever et al. 2017; CCSF 2020). If sea levels rise by 24 inches, Bay Area coastal communities could experience multiple flood events, in addition to 90 to 150 days of high tide flooding, each year (Ghanbari et al. 2019; Sidder 2019). **Figure B.1-12** provides a schematic example of this dynamic. Before sea level rise, a hypothetical flooding threshold could be overtopped a few times each year, primarily in the winter season (**Figure B.1-12**, left). However, with sea level rise, the same flooding thresholds could be overtopped frequently throughout the entire year (**Figure B.1-12**, right). This more frequent, yet less severe flooding will cause chronic and cumulative damages (FEMA 2015, Sievanen et al. 2018, Sidder 2019). Therefore, developing an appropriate strategy to adequately account for high tide flooding along the POSF shoreline is important for the POSF and the CCSF.



Source: (University of Hawaii 2021)

Figure B.1-12: Schematic on the Effect of Sea Level Rise on Flooding Events

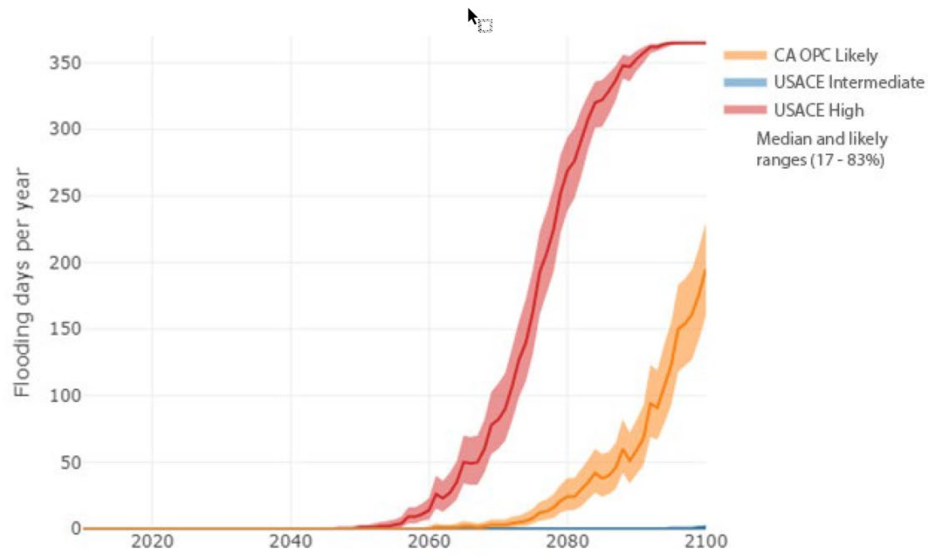
Figure B.1-13 and **Figure B.1-14** are adapted from the University of Hawaii Sea Level Center Flooding Days Projection Tool, configured for San Francisco, and using the USACE Intermediate (http://www.noaa.gov/NOAA Intermediate Low), USACE High (NOAA Intermediate High), and approximately the California Ocean Protection Council (OPC) Likely (NOAA Intermediate) sea level rise projections (Sweet et al. 2017, OPC and CNRA 2018, USACE 2019a, University of Hawaii 2021). This tool provides an estimate of the number of days a given flood threshold could be overtopped, based on the NOAA sea level rise projections and their median and likely ranges.



Source: (University of Hawaii 2021)

Figure B.1-13: High Tide Flooding Days per Year with Flooding Threshold of 8.4 feet NAVD88 in San Francisco

As sea levels rise, high tide flooding will become more frequent along the POSF's waterfront shoreline. Some areas of the shoreline, such as the near the Ferry Building along the Embarcadero waterfront, experience minor high tide flooding today during the highest annual tides (such as a King Tide NAVD88). **Figure B.1-13** presents the number of days each year that high tides would overtop this flood threshold over time. Today, this threshold would be overtopped between 1 and 6 times each year. By 2030, this threshold could be overtopped between 1 and 22 times depending on the rate of sea level rise. By 2050, this threshold could be overtopped between 4 and 150 times. By 2070, this threshold could be overtopped every day of the year under the highest sea level rise projections – in the absence of high winds and coastal storm events that occur each year in the winter months, particularly when high winds push additional water over the shoreline. This area has a flooding threshold of about 18 to 22 inches above MHHW, or approximately 8.4 feet.



Source: (University of Hawaii 2021)

Figure B.1-14: High Tide Flooding Days per Year with Flooding Threshold of 11.4 feet NAVD88 in San Francisco

The mean shoreline elevation along the POSF's waterfront is about 11.8 feet NAVD88 (Port of San Francisco 2021b). Using this elevation as a flooding threshold, high tide shoreline overtopping is not anticipated until closer to 2050 (**Figure B.1-14**). By 2050, high tides could overtop this flooding threshold 0 to 3 times per year. By 2060, high tides could overtop this flooding threshold by 0 to 23 times per year, depending on the rate of sea level rise. Under the highest sea level rise projections, high tides could overtop the shoreline every day by 2090 in the absence of high winds and coastal storm events. Additional information on the sea level rise calculations used in estimating future water elevations is documented within *Appendix J: Climate*.

Although **Figure B.1-13** and **Figure B.1-14** help highlight the potential timing and importance of high tide flooding for San Francisco, they do not highlight the scale of the problem, the locations most at risk, or the potential inland extent of high tide flooding. To better characterize this dynamic, water level and wave inputs that represent high tide flooding were developed for G2CRM.

High tide water levels that represent the 6-month, 4-month, 3-month, 2-month, and 1-month return frequencies were analyzed for incorporation within G2CRM. The monthly recurrence interval was selected as the highest frequency threshold for consistency with Sweet et al (2022). Analysis of events more frequent than monthly poses a challenge for G2CRM as events can become overlapping (e.g., if a weekly event is used, it is likely that more extreme events may happen concurrently within the model, resulting in model simulation failure). The use of monthly events did not cause model failure. Additional evaluation of repetitive high tide flooding and its implications to the G2CRM economic damage assumptions is documented separately within *Appendix E: Economics and Social Considerations*.

B.1-4.2 Coastal Storm Selection for Economic Damage Modeling

The methodology used to develop storm inputs for economic damage modeling in G2CRM is described in *Sub-Appendix B.1.1: Coastal Extreme Water Levels and High Tide Flooding*. Local storm identification criteria were developed for the Extreme Precipitation Study underway to support the SFPUC, the POSF, and the San Francisco International Airport (May et al. 2019). Past storm events that occurred between 1980 and the present were reviewed and compared with local newspaper stories of damage to Bay Area communities to identify a suite of relevant storm characteristics related to large and damaging storm events. Although large storm events occurred prior to 1980, satellite imagery and other data are only available for storms that have occurred within approximately the past four decades.

Based on historical storm catalog, a storm duration of 3 days was selected for identifying and selecting storm events for the coastal storm inputs to G2CRM. G2CRM does not the support the selection of varying storm durations. In addition, in the absence of applying a stage volume curve within G2CRM, only the peak water level + wave height combination from the 3-day event is selected for assessing damages. The peak water level + weight combination is extended inland using the bathtub method to support the damage calculations. Therefore, for the purposes of the coastal storm inputs, adequately capturing the peak water level and weight combination is more important than capturing the correct storm duration.

B.1-4.3 Coastal Storm Modeling (Inundation Analysis)

B.1-4.3.1 Purpose of Analysis

Sub-Appendix B.1.2 presents a map series depicting the landward extents of temporary coastal flooding from a 1% AEP and monthly flood event (99.999% AEP) SWEL including sea level change (SLC) in 25-year intervals from 2040 through the 2140 planning horizon. The flooding extents corresponding to the FWOP condition, and FWP Alternatives C, D, E, F, and G are shown. For Alternatives D, E, F and G the progression of coastal flooding mapped until the 2140 horizon considers a first action “2040 alignment” and second action “2090 alignment,” for all SLC scenarios. Alternative C only considers a first action “2040 alignment.” The map series also shows the progression of coastal flooding considering a first action only move under Alternative D, E, F, and G. Details on the flood mapping methodology and the supporting topographic and water level data are provided in the following sections. FWP condition for Alternative B is identical to the FWOP condition, expect where individual building footprints are protected from flooding through nonstructural measures.

These maps do not consider the additional elevation of water levels due to wave hazards, including wave setup, wave runup, or overtopping.

B.1-4.3.2 Future Without Project and Future With Project Conditions

The FWOP Maps depict the landward coastal flood extents through the 2140 planning horizon, reflective of 2010 shoreline topography conditions, recent major urban developments, and planned developments already funded but not fully constructed.

The FWP Maps also depict the landward coastal flood extents through the 2140 planning horizon, based on the same topographic data as the FWOP condition but with additional modifications to represent the shoreline modification proposed for each of the FWP structural alternative alignments. These are FWP Alignments C, D, E, F, and G. Alternative B (nonstructural) is not included in the mapping due to insufficient spatial resolution of the proposed nonstructural measure to differentiate this alternative from the FWOP condition.

See *Appendix A: Plan Formulation* for details on the FWP alternatives and their associated alignments for the San Francisco Waterfront Coastal Flood Study.

B.1-4.3.2.1 Total Net Benefits Plan

An additional set of maps were created to represent the shoreline flood protection alignments and elevations selected to represent the Total Net Benefits Plan (TNBP). The TNBP scenario depicts landward coastal flood extents through the 2140 planning horizon, in both 10- and 25-year increments with the first and second actions. The TNBP selected for the Tentatively Selected Plan (TSP) was a single waterfront wide plan with a first and second action that hybridized certain elements of Alternatives B, D, and G across the SFWCFS reaches. The description of the alignments and elevations for the TNBP are provided in *Appendix A: Plan Formulation*.

B.1-4.3.3 Supporting Data and Mapping Methods

B.1-4.3.3.1 Topographic Digital Elevation Mode

The base topographic data for the FWOP mapping is a digital elevation model (DEM) with 1-meter by 1-meter grid cell resolution in ESRI ArcGIS raster format created for the Embarcadero Seawall Program, a component of the POSF Waterfront Resilience Program.

For FWOP mapping, this DEM was modified using ArcGIS tools to represent elevations (representing topographic high ground that would control overtopping) of funded planned shoreline developments. The planned developments include: Agua Vista Park Improvements, Crane Cove Park, Mission Bay Ferry Landing, Mission Bay Park P3, Mission Bay Water Taxi Landing, Mission Rock, P22 Bayfront Park, Pier 70, Pier 70 Shipyard, Pier 94 Backlands Improvements, and the Potrero Power Station.

For FWP mapping, the FWOP DEM was modified for each flood protection alternative to make a total of five independent DEMs, one for each FWP alternative (C, D, E, F, and G). The footprint and elevation for each alternative's line of defense (LOD) was stamped over the FWOP DEM to raise the elevation of the shoreline. A 30-foot buffer on the landward side of the LOD was also applied to ensure each LOD is contiguous across the entire SFWCFS shoreline extent, does not result in any open-water gaps between the proposed LOD crest location and the shoreline due to some alignments being offset from the current shoreline edge, and reaches the appropriate topographic high ground to eliminate any potential flood pathways.

The 1-meter DEM was originally developed from a 2010 Light Detection and Ranging (LiDAR) survey of the San Francisco coastline by the U.S. Geological Survey (USGS) that extended to the Embarcadero promenade (Dewberry 2011), with additional modifications to incorporate elevations of pier structures. Recent aerial photogrammetry survey conducted in 2019 and provided by the POSF was also used to update the waterfront topography where significant differences were found between the 2010 LiDAR survey and the 2019 photogrammetry survey.

B.1-4.3.3.2 Mapping Depth and Extent

The water levels used to create these layers were created using the method of overtopping potential. Overtopping potential is the condition where the water surface elevation exceeds the shoreline. The flood extent layers depict the depth to water over the delineated shoreline features. ArcGIS tools were used for the creation of the flood extents.

Prior to accounting for sea level rise, two reference water levels were mapped for the entire waterfront, one for 1% AEP and one for 99.999% AEP scenario. To capture the spatial variability in tidal datums and extreme tide events across the waterfront, a spatially varying water surface DEM was created for the 1% AEP and 99.000% AEP scenarios. Both scenarios are derived from the San Francisco Bay Tidal Datums and Extreme Tides Study (May et al., 2016) which calculated tidal datums and extreme tide elevations from water levels output from a hydrodynamic model from the FEMA San Francisco Bay Area Coastal Baywide numerical modeling effort (DHI, 2013). The FEMA model output provided water level data in 15-minute time intervals from 1973 through 2003 for 53 offshore locations parallel to the San Francisco shoreline. Shore-perpendicular transects were created from these shoreline points to extend the MHHW tidal datum and 1% AEP SWELs landward. To extend the water surface elevations landward, a series of points were created along each transect and assigned the appropriate MHHW or 1% AEP SWELs, which were interpolated between each transect to create a seamless water surface DEM for the entire San Francisco waterfront. While the 1% AEP stillwater DEM relies on the results from San Francisco Bay Tidal Datums and Extreme Tides Study, the 99.9994% AEP stillwater DEM relies on the results from *Sub-Appendix B.1.1: Coastal Extreme Water Levels and High Tide Flooding*. The

99.999% AEP tidal datum was simplified approximately 2.5 inches above the spatially varying MHHW DEM.

This method does not take into account the associated physics of overland flow, dissipation, levee overtopping, storm duration, or potential shoreline or levee erosion associated with extreme water levels and wave effects. A more sophisticated modeling effort would be required to account for these complex dynamics, however, given uncertainties in future land use geomorphic conditions, additional modeling may not provide additional accuracy.

To create flood extent and depth layers that account for future sea level rise, the 1% AEP and monthly water surface DEMs used a linear superposition method. This approach relies on the assumption that daily and extreme tide levels increase linearly in response to sea level rise (BCDC et al., 2019).

To create a single mapping scenario accounting for sea level rise, the sea level rise value is linearly added to the entire water surface DEM. The ground topographic DEM is then subtracted from the water surface DEM to create a new flood depth and extent raster. Areas of the water surface DEM that are not hydraulically connected to the Bay are identified using a method that finds connectivity by zone. If a raster cell in the DEM is not within the same connectivity zone as the Bay, then that cell is removed from the water surface layer and redefined as a low-lying area that is not hydraulicly connected to the Bay. The resulting DEM that remains after removing low-lying areas is the flood depth and extent raster. This process was repeated for each mapping scenario.

B.1-4.3.3.3 Mapping Caveats and Assumptions

The maps are not detailed to the parcel-scale and should not be used for navigation, permitting, regulatory, or other legal uses. Additional caveats are summarized below.

- Flooding from rainfall-runoff events is not considered in the maps. The maps also do not account for stormwater system upgrades or other changes to the Bay or the region that may occur in response to sea level rise.
- The depth and extent of landward flooding shown on the maps do not include local wind and wave effects. These processes could have a significant effect on the depth and extent of flooding especially near the shoreline and can result in flooding earlier than shown in the maps.
- The flood maps do not consider the duration of flooding or the potential mechanism for draining the floodwaters from the inundated land once the extreme high tide levels recede.
- The bathymetry of the Bay is assumed to remain constant. The accumulation of organic matter in wetlands, potential sediment deposition and/or resuspension, and subsidence that could alter the Bay hydrodynamics and/or bathymetry are not captured within the SLR scenarios.

B.1-4.3.4 Scenarios

The following sections summarizes the primary scenarios evaluated for the FWOP and FWP conditions, which are the 1% AEP (100-year) and 99.999% (monthly) exceedance events and serve as the reference water levels to which sea level rise amounts are linearly added for the flood mapping.

B.1-4.3.4.1 Water Levels and Sea Level Change

The 1% AEP scenario represents flooding due to large, but infrequent storm surge events, with increasing landward extent of flooding when coupled with sea level rise. The 99.999% AEP (monthly) scenario represents recurrent flooding from monthly high tide levels. **Table B.1-3** presents the 1% and monthly AEP values per SFWCFS reach. *Sub-Appendix B.1.1: Coastal Extreme Water Levels and High Tide Flooding* provides detailed methodology on the development of the 1% AEP and Monthly reference water levels.

Table B.1-3: 1% and monthly AEP values per SFWCFS Reach

USACE Reach	1-month (99.9994%)	100-year (1%)
	Feet-NAVD88	
Reach 1	6.82	9.40
Reach 2	6.93	9.58
Reach 3	7.09	9.71
Reach 4	7.24	9.66

The flood maps incorporate change in the 1% AEP and Monthly reference water levels across 5 SLC curves: USACE Low, USACE Intermediate, USACE High, OPC 1:200 Likely and High Impact, but Plausible (1:200) curves. Five-time horizons were selected: 2040, 2065, 2090, 2115 and 2140. In total there are 25 mapping scenarios across the 5 SLC scenarios and five-time horizons. The associated inches of SLC, by time horizon, for each SLC curve are shown in **Table B.1-4**.

Table B.1-4: SLC by Time Horizon

	2040	2065	2090	2115	2140
USACE Low	3.24 in	5.17 in	7.20 in	9.13 in	11.16 in
USACE Inter	5.52 in	10.75 in	17.28 in	25.15 in	34.32 in
USACE High	13.08 in	28.51 in	49.56 in	76.10 in	108.24 in

	2040	2065	2090	2115	2140
OPC Likely	9.60 in	21.24 in	35.18 in	49.30 in	63.74 in
OPC 1:200	16.32 in	39.21 in	69.74 in	103.41 in	140.30 in

B.1-4.3.4.2 Total Mapping Scenarios

Table B.1-5 summarizes the mapping scenarios presented in *Sub-Appendix B.1.2* for the FWOP, FWP, and TNBP scenarios.

Table B.1-5: Summary of Mapping Scenarios

Scenario	First Action	First and Second Action (2040 and 2090)	Total Maps
FWOP	• 50 maps – 1 FWOP scenario – 1% AEP and Monthly – 5 SLC curves – 25-year time periods	• 50 maps – 1 FWOP scenario – 1% AEP and Monthly – 5 SLC curves – 25-year time periods	100
FWP	• 250 maps – 5 FWP scenarios – 1% AEP and Monthly – 5 SLC curves – 25-year time periods	• 250 maps – 5 FWP scenarios – 1% AEP and Monthly – 5 SLC curves – 25-year time periods	500
TNBP	• 150 maps – 1 TNBP scenario – 1% AEP and Monthly – 5 SLC curves – 10-year time periods – 25-year time periods	• 150 maps – 1 TNBP scenario – 1% AEP and Monthly – 5 SLC curves – 10-year time periods – 25-year time periods	150

B.1-4.4 Coastal Storm Inundation Results

B.1-4.4.1 Future Without Project

The FWOP Maps that show the impacts to Reach 1 through 4 can be found in *Sub-Appendix B.1.2.1* on pdf pages 2 through 54. The 1% AEP water levels with the five sea level curves are pdf pages 3 through 28 and the monthly water levels with the five sea level curves are on pdf pages 29 through 54.

B.1-4.4.2 Future With Project

The FWP Maps that show the impacts to Reach 1 through 4 can be found in *Sub-Appendix B.1.2.1*. Maps are available for each of the Alternatives C through G for the 1% AEP water levels and monthly water levels with the five sea level curves. Since Alternative B is the nonstructural option, the flood extents are the same as FWOP.

B.1-4.4.2.1 FWP Alternative C – Defend, Scaled for Lower Risk

The FWP Maps for Alternative C can be found in *Sub-Appendix B.1.2.1* on pdf pages 56 through 108.

B.1-4.4.2.2 FWP Alternative D – Defend, Scaled for Low-Moderate Risk

The FWP Maps for Alternative D can be found in *Sub-Appendix B.1.2.1* on pdf pages 109 through 161.

B.1-4.4.2.3 FWP Alternative E – Defend Existing Shoreline, Scaled for Higher Risk

The FWP Maps for Alternative E can be found in *Sub-Appendix B.1.2.1* on pdf pages 162 through 214.

B.1-4.4.2.4 FWP Alternative F – Manage the Water, Scaled for Higher Risk

The FWP Maps for Alternative F can be found in *Sub-Appendix B.1.2.1* on pdf pages 215 through 267.

B.1-4.4.2.5 FWP Alternative G – Partial Retreat, Scaled for Higher Risk

The FWP Maps for Alternative G can be found in *Sub-Appendix B.1.2.1* on pdf pages 268 through 320.

B.1-4.4.2.6 Future With Project Total Net Benefits Plan

Two sets of map books are included in the study for the TNBP. The first set includes mapping for Reaches 1 through 4 at 25-year time periods and can be found in *Sub-Appendix B.1.2.2*. The second set includes mapping for Reaches 1 through 4 at 10-year time periods and can be found in *Sub-Appendix B.1.2.3*.

Section B.1-5 Overtopping Analysis

B.1-5.1 Purpose of Analysis

This assessment includes wave runup sensitivity analysis to support the selection of the 2-foot wave proxy. The sensitivity analyses evaluate potential wave runup and overtopping at select locations under existing conditions and with future sea level rise and examines how wave runup and overtopping can be reduced through design features to achieve a TWL elevation that satisfies the 2-foot wave proxy assumption. For Bay wave conditions, where 1% AEP wave heights are on the order of 2 to 4 or more feet (*Sub-Appendix B.1.1: Coastal Extreme Water Levels and High Tide Flooding*), moderate wave overtopping may be allowable in some conditions. The sensitivity analysis looked at two potential wave overtopping conditions. The more stringent condition limits wave overtopping to prevent hazardous conditions for pedestrians. The other condition limits wave overtopping to prevent damage to structures.

The sensitivity analyses are not a replacement for detailed wave analysis during the preconstruction engineering and design (PED) phase. This assessment does not consider the full range of potential shoreline types, or hydrodynamic and wave conditions, along the San Francisco shoreline. The sensitivity analyses are hypothetical in nature, and not representative of actual conditions or proposed green-gray solutions to dissipate wave energy.

B.1-5.2 Overtopping Flow Estimates

The overtopping equations are derived from the EurOtop Manual (Second Edition) on wave overtopping of sea defenses and related structures. Given the complexity of water levels and waves in the Bay, this wave overtopping sensitivity assessment uses an approach that evaluates wave overtopping on an event-by-event basis using water levels and waves from the 1973 to 2003 FEMA MIKE21 model hindcast period. The wave overtopping sensitivity assessment considers an infinite range of shoreline crest elevations, governed by the wave runup elevations and overtopping thresholds. The overtopping flow estimates were evaluated for three future time horizons and two SLC scenarios shown in **Table B.1-6**.

Table B.1-6: Future Time Horizons and Two Sea Level Curves

Year	OPC Likely (feet)	USACE High (feet)
2040	0.8	1.09
2090	2.9	4.13
2140	5.3	9.02

Source: OPC & CNRA (2018); USACE (2019)

B.1-5.3 Analysis Locations

Three locations that capture a range of existing shoreline and wave conditions were selected to inform the wave overtopping sensitivity analysis, including two locations along the Northern Waterfront, and one location along the Southern Waterfront. The analyses use transects and shoreline profiles developed for the FEMA San Francisco Bay Area Coastal Study. The transect number and locations used in the FEMA analysis is presented on **Figure B.1-15**, and the FEMA 1% AEP TWL for each transect is noted below:

- FEMA Transect 18 – Ferry Building, 1% AEP TWL = 11.2 feet NAVD88
- FEMA Transect 20 – Brannan Street, 1% AEP TWL = 13.6 feet NAVD88
- FEMA Transect 23 – Bayfront Park, 1% AEP TWL = 12.1 feet NAVD88

Figure B.1-15: Transect Locations for Wave Overtopping Analysis (FEMA Analysis Transect Locations)

B.1-5.3.1 Shoreline Profiles

Three shoreline profiles with and without armoring or wave dissipation features. The shoreline profiles include:

- Vertical (no armoring)
- Vertical + rock mound armoring (2-foot rock mound)
- Steep Slope 3H:1V (no armoring)
- Steep Slope 3H:1V (armored)
- Shallow Slope 20H:1V (no vegetation)
- Shallow Slope 20H:1V (vegetated)

Figure B.1-16 to **Figure B.1-18** shows the current shore perpendicular profile for Transect 18 with modifications to represent a vertical wall, steep sloped shoreline (3H:1V), and shallow sloped shoreline (20H:1V) with a crest height of 15.5 feet NAVD88. These are theoretical modifications for illustrative purposes. The wave overtopping sensitivity assessment considers an infinite range of shoreline crest elevations, governed by the wave runup elevations and overtopping thresholds.

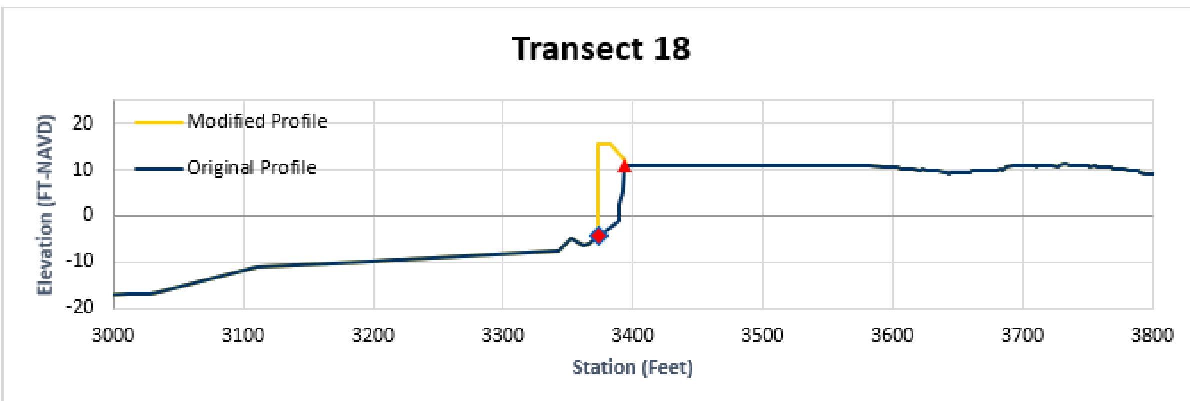


Figure B.1-16: Modified Transect 18 Profile Illustrating a Vertical Structure

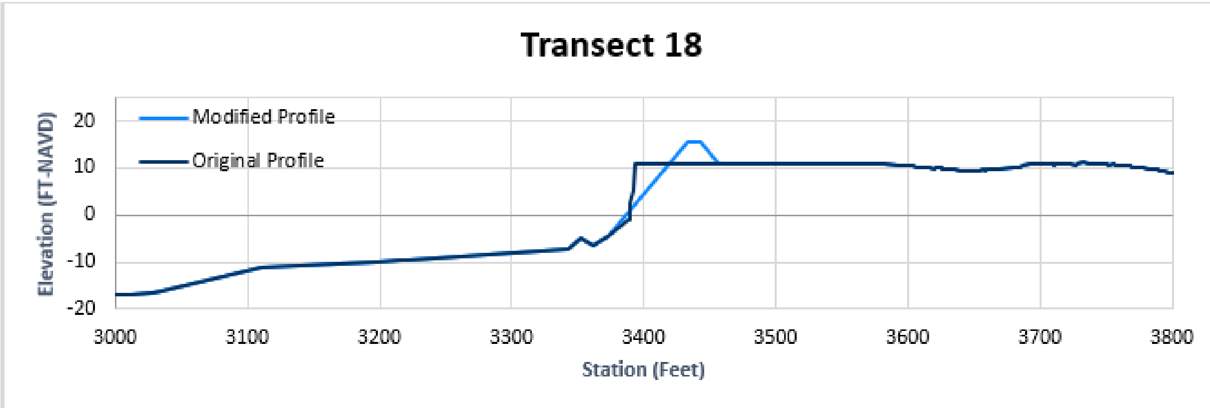


Figure B.1-17: Modified Transect 18 Profile Illustrating a Shoreline with a Steep 3H:1V Slope

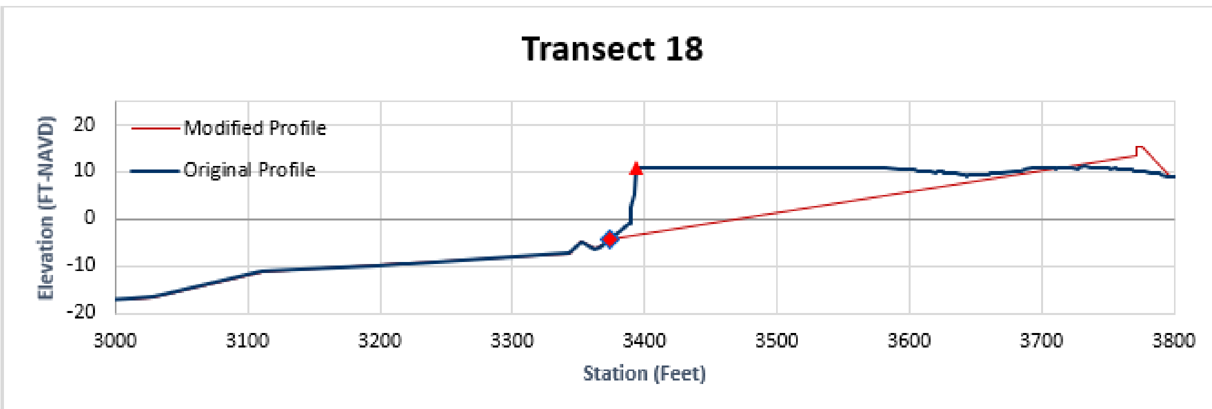


Figure B.1-18: Modified Transect 18 Profile Illustrating a Shoreline with a Shallow 20H:1V Slope

B.1-5.3.2 Overtopping Thresholds

To evaluate potential wave overtopping, thresholds of tolerable overtopping were selected to calculate minimum shoreline crest elevations necessary to limit overtopping. **Figure B.1-19** shows critical values of average overtopping discharges according to USACE Coastal Engineering Manual Volume VI Table VI-5-6 (USACE, 2011)

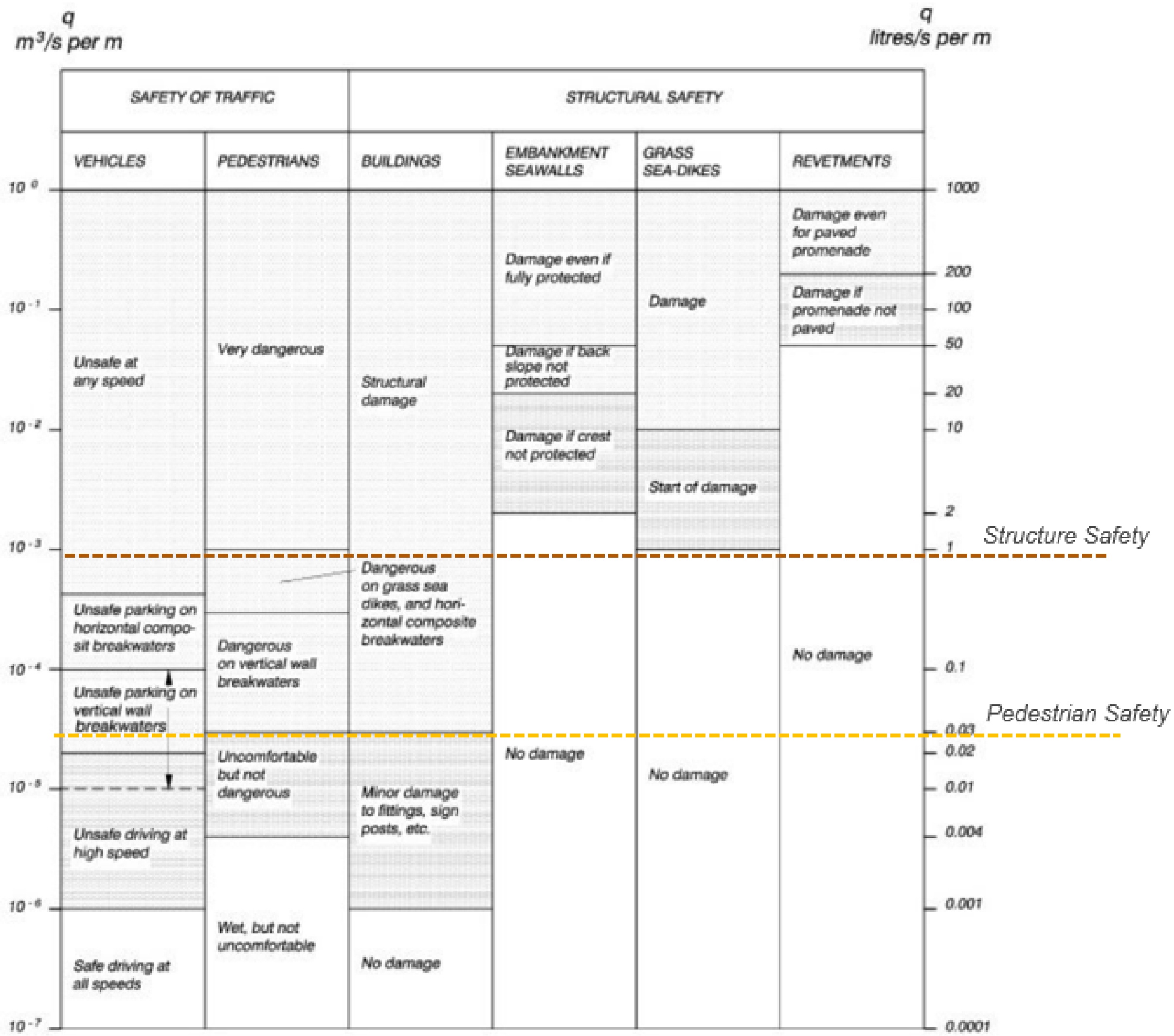


Figure B.1-19: Permissible Wave Overtopping (Source adapted from USACE (2011)

Two overtopping thresholds were selected to capture a range of potential wave overtopping hazards (**Figure B.1-19**):

Pedestrian Safety

- Allowable overtopping (q) = 3e-5 (0.00003 m3/s; 0.03 l/s/m).
- Aligns to the limit upper limit of the "*Uncomfortable, but not dangerous"* overtopping rate, and before conditions become "*Dangerous on vertical wall breakwaters."*
- This threshold is stricter than the EurOtop tolerable overtopping rate to maintain pedestrian safety in a wave climate with small waves (e.g., below 2 meters) where the tolerable mean overtopping rate is relaxed to 10-20 l/s/m. For waves

closer to 1 meter in height, the tolerable overtopping rate is >75 l/s/m which is within the "*Very dangerous*" zone for pedestrians on **Figure B.1-19**.

- With this overtopping threshold, minimum crest elevations can be approximated during "zero overtopping" conditions, for significant wave heights of 1 meter or less. This condition is comparable to the 1% AEP TWL, which accounts for tides, storm surge, wave setup, and wave runup).

Structural Safety

- Allowable overtopping (q) = 1e-3 (0.001 m3/s; 1 l/s/m).
- Relaxed overtopping threshold compared to pedestrian safety.
- Similar to EurOtop, the recommended tolerable overtopping for property (building structure element) behind shoreline structure (e.g., floodwall), where significant wave height is between 1 to 3 meters.

B.1-5.4 Results

For the freeboard required above the 1% AEP SWEL, both the best estimate value and lower and upper bounds are provided. The lower and upper bounds are derived from the EVA analysis of the required shoreline crest elevations calculated for the entire hindcast period, where the lower bound represents the smallest freeboard estimate within an acceptable goodness of fit of the parameters used in the EVA analysis, and the upper bound represents the largest freeboard estimate within an acceptable goodness of fit. See *Sub-Appendix B.1.1: Coastal Extreme Water Levels and High Tide Flooding* for detailed methodology of the EVA analysis.

B.1-5.4.1 Current Conditions

Table B.1-7 presents the best estimate of the freeboard required to meet the structural safety overtopping limits and the upper and lower bound for each shoreline profile.

Table B.1-7: Freeboard (q = 0.001 m3/s/m; structure safety)

Profile	Statistic	Transect 18	Transect 20	Transect 23
Vertical	Best Estimate	1.1	2.1	2.5
	Bounds	0.64 to 1.20	1.80 to 2.41	1.87 to 2.74
Vertical+mound	Best Estimate	1.1	2.1	2.5
	Bonds	0.64 to 1.20	1.80 to 2.41	1.87 to 2.74
3H:1V	Best Estimate	2.2	4.4	4.5
	Bounds	1.49 to 2.58	3.98 to 4.71	4.12 to 4.70

Profile	Statistic	Transect 18	Transect 20	Transect 23
3H:1V+armor	Best Estimate	1.0	2.1	2.5
	Bounds	0.64 to 1.12	1.73 to 2.27	1.86 to 2.59
20H:1V	Best Estimate	0.3	0.4	0.3
	Bounds	-0.25 to 0.38	0.08 to 0.53	0.03 to 0.52
20H:1V+veg	Best Estimate	-0.1	0.0	0.0
	Bounds	-0.29 to 0.29	-0.30 to 0.32	-0.27 to 0.28

For profiles with vertical walls, the best estimate of freeboard ranges from 1.0 to 2.5 feet if the profile is not armored. With armoring (shallow mound armoring), the best estimate of freeboard does not change if non-impulsive wave conditions dominate. Note that a higher mound will reduce the foreshore water depth and trigger impulsive wave conditions, resulting in additional height of shoreline needed to limit overtopping conditions.

Vertical wall estimates are highly sensitive to the presence or absence of impulsive wave breaking at the wall. Impulsive wave conditions are triggered from large waves combined with low enough water depths to trigger wave breaking at the structure. Analysis of the 31-year hindcast period found that extreme conditions were dominated by events with high SWELs and non-impulsive wave conditions, rather than lower SWEL and impulsive wave conditions. Under these conditions vertical walls perform quite well, with reduced freeboard requirement when compared with unarmored steep slopes (3H:1V), which will always trigger wave breaking at some point as waves progress up the slope of the structure. It should be noted, however, that vertical walls still pose a risk of performing drastically worse should they experience waves condition that exceed those considered in this study.

For steeply sloping profiles, the best estimate of freeboard ranges from 2.2 to 4.5 feet if the profile is not armored. With armoring, the mean freeboard can be reduced to approximately 1.0 to 2.5 feet.

Steeper sloped shorelines (e.g., 3H:1V) without armoring require the largest freeboard to minimize wave overtopping conditions, however armoring on the structure slope can greatly reduce the amount of freeboard required.

For shallow profiles without vegetation, the best estimate of freeboard is 0.1 to 0.4 foot, and overtopping can be almost fully mitigated with sufficient vegetation to attenuate the incident wave heights.

Overall, shallow profile slopes with vegetation allow for the highest performance in reducing wave overtopping potential, however the reduction in wave overtopping is primarily attributed to the slope angle, where sufficient landward extent is required to allow the shoreline height to exceed most combinations of storm surge and wave conditions (while minimizing any wave runup).

Table B.1-8 presents the best estimate of the freeboard required to meet the pedestrian safety overtopping limits and the upper and lower bound for each shoreline profile.

Table B.1-8: Freeboard in Feet (q = 0.00003 m3/s/m; pedestrian safety)

Profile	Statistics	18	20	23
Vertical	Best Estimate	2.2	4.2	3.9
	Bounds		3.74 to 4.39	3.65 to 4.26
Vertical+mound	Best Estimate	2.2	4.2	3.9
	Bounds	1.66 to 2.78	3.74 to 4.39	3.65 to 4.26
3H:1V	Best Estimate	3.8	6.7	6.9
	Bounds	3.48 to 4.16	6.41 to 7.32	6.44 to 7.17
3H:1V+armor	Best Estimate	1.5	3.5	3.6
	Bounds	1.02 to 1.84	3.06 to 3.60	3.39 to 4.03
20H:1V	Best Estimate	0.6	0.8	0.8
	Bounds	0.05 to 0.67	0.28 to 0.89	0.30 to 0.88
20H:1V+vegetation	Best Estimate	-0.1	0.0	0.0
	Bounds	-0.28 to 0.30	-0.30 to 0.32	-0.26 to 0.29

For vertical walls, the best estimate of freeboard ranges from 2.2 to 3.9 feet if the profile is not armored. With armoring (shallow mound armoring), the freeboard required does not change if non-impulsive wave conditions dominate. Note that a higher mound will reduce the foreshore water depth and trigger impulsive wave conditions, resulting in additional height of shoreline needed to limit overtopping conditions.

For steeply sloping profiles, the best estimate of freeboard ranges from 3.8 to 6.9 feet if the profile is not armored. With armoring, the freeboard required can be reduced to approximately 1.5 to 3.7 feet.

For shallow profiles without vegetation, the best estimate of freeboard ranges from 0.5 to 0.8 foot, and overtopping can be almost fully mitigated with sufficient vegetation to attenuate the incident wave heights.

B.1-5.4.2 Freeboard with Sea Level Rise

Table B.1-9 to **Table B.1-14** shows the freeboard required for each transect locations, considering sea level rise. The addition of sea level rise to the input water levels resulted in a generally linear response in the required freeboard for all shoreline profile types and all transect locations. This could be attributed to the incident wave heights in the hindcast being primarily non-depth limited; therefore, without increasing the magnitude of the incident wave heights coupled with increasing the water depths, nonlinear increases in wave runup as sea levels rise is not likely to occur. Increasing wave heights due to changes in wind speed that could occur as the climate changes was beyond the scope of this study but should be considered in a future update as increasing wave heights could trigger depth-limited breaking and a nonlinear response in wave runup with sea level rise. A nonlinear response on steep and vertical slopes has been observed in similar studies.

Table B.1-9: Transect 18 – Freeboard in Feet (q = 0.001 m3/s/m; structure safety)

Profile	Statistic	Historical	OPC Likely 2040	OPC Likely 2090	OPC Likely 2140	USACE High 2040	USACE High 2090	USACE High 2140
Vertical	Best Estimate	1.1	1.1	1.1	1.36	1.1	1.2	1.5
	Bounds	0.64 to 1.20	0.69 to 1.22	0.83 to 1.23	0.85 to 1.33	0.70 to 1.23	0.87 to 1.28	0.81 to 1.52
Vertical+ mound	Best Estimate	1.1	1.1	1.1	1.3	1.1	1.2	1.5
	Bounds	0.64 to 1.20	0.69 to 1.22	0.83 to 1.23	0.85 to 1.33	0.70 to 1.23	0.87 to 1.28	0.81 to 1.52
3H:1V	Best Estimate	2.2	2.3	2.4	2.6	2.3	2.5	2.8
	Bounds	1.49 to 2.58	1.54 to 2.65	1.67 to 2.83	1.76 to 3.02	1.58 to 2.68	1.72 to 2.96	1.95 to 3.14
3H:1V+ armor	Best Estimate	1.0	1.0	1.0	1.1	1.0	1.1	1.2
	Bounds	0.64 to 1.12	0.65 to 1.10	0.69 to 1.19	0.70 to 1.21	0.65 to 1.11	0.71 to 1.21	0.74 to 1.25
20H:1V	Best Estimate	0.3	0.3	0.3	0.3	0.3	0.3	0.3
	Bounds	-0.25 to 0.38	-0.24 to 0.37	-0.25 to 0.41	-0.22 to 0.40	-0.25 to 0.40	-0.21 to 0.41	-0.18 to 0.40
20H:1V+ vegetation	Best Estimate	-0.1	-0.1	-0.1	-0.1	-0.1	-0.1	-0.1
	Bounds	-0.29 to 0.92	-0.28 to 0.29	-0.28 to 0.29	-0.28 to 0.29	-0.28 to 0.29	-0.28 to 0.29	-0.28 to 0.29

Table B.1-10: Transect 18 – Freeboard in Feet (q = 0.00003 m3/s/m; pedestrian safety)

Profile	Statistic	Historical	OPC Likely 2040	OPC Likely 2090	OPC Likely 2140	USACE High 2040	USACE High 2090	USACE High 2140
Vertical	Best Estimate	2.2	2.3	2.2	2.4	2.3	2.3	2.7
	Bounds	1.66 to 2.78	1.68 to 2.79	1.21 to 2.19	1.25 to 2.36	1.69 to 2.82	1.23 to 2.27	1.37 to 2.59
Vertical+ mound	Best Estimate	2.2	2.3	2.2	2.4	2.3	2.3	2.7
	Bounds	1.66 to 2.78	1.68 to 2.79	1.21 to 2.19	1.25 to 2.36	1.69 to 2.82	1.23 to 2.27	1.37 to 2.59
3H:1V	Best Estimate	3.8	3.8	3.9	4.0	3.8	3.9	4.2
	Bounds	3.48 to 4.16	3.53 to 4.23	3.62 to 4.41	3.72 to 4.60	3.55 to 4.26	3.69 to 4.53	3.97 to 4.95
3H:1V+ armor	Best Estimate	1.5	1.6	1.6	1.7	1.6	1.6	1.8
	Bounds	1.02 to 1.84	1.03 to 1.81	1.06 to 1.93	1.05 to 1.93	1.04 to 1.83	1.06 to 1.97	1.11 to 2.03
20H:1V	Best Estimate	0.6	0.5	0.5	0.5	0.5	0.5	0.5
	Bounds	0.05 to 0.67	0.09 to 0.66	0.08 to 0.70	0.01 to 0.65	0.12 to 0.67	0.03 to 0.64	0.00 to 0.065
20H:1V+ vegetation	Best Estimate	-0.1	-0.1	-0.1	-0.1	-0.1	-0.1	-0.1
	Bounds	-0.28 to 0.30	-0.28 to 0.30	-0.28 to 0.30	-0.28 to 0.29	-0.28 to 0.30	-0.28 to 0.29	-0.28 to 0.29

Table B.1-11: Transect 20 – Freeboard in Feet (q = 0.001 m3/s/m; structure safety)

Profile	Statistic	Historical	OPC Likely 2040	OPC Likely 2090	OPC Likely 2140	USACE High 2040	USACE High 2090	USACE High 2140
Vertical	Best Estimate	2.1	2.1	2.0	2.0	1.8	2.0	2.1
	Bounds	1.80 to 2.41	1.79 to 2.41	1.69 to 2.35	1.74 to 2.37	1.54 to 2.36	1.72 to 2.35	1.81 to 2.41
Vertical+ mound	Best Estimate	2.1	2.1	2.0	2.0	1.8	2.0	2.1
	Bounds	1.80 to 2.41	1.79 to 2.41	1.69 to 2.35	1.74 to 2.37	1.54 to 2.36	1.72 to 2.35	1.81 to 2.41
3H:1V	Best Estimate	4.4	4.4	4.4	4.5	4.4	4.5	4.6
	Bounds	3.98 to 4.71	3.97 to 4.72	3.95 to 4.65	3.98 to 4.64	3.96 to 4.70	3.97 to 4.63	4.01 to 4.65
3H:1V+ armor	Best Estimate	2.1	2.1	2.1	2.0	2.0	2.0	2.0
	Bounds	1.73 to 2.27	1.71 to 2.25	1.68 to 2.22	1.66 to 2.21	1.70 to 2.25	1.67 to 2.21	1.68 to 2.20
20H:1V	Best Estimates	0.4	0.4	0.4	0.4	0.4	0.4	0.4
	Bounds	0.08 to 0.53	0.08 to 0.53	0.07 to 0.53	0.07 to 0.53	0.07 to 0.53	0.07 to 0.53	0.07 to 0.53
20H:1V+ vegetation	Best Estimates	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Bounds	-0.30 to 0.32	-0.30 to 0.32	-0.30 to 0.32	-0.30 to 0.32	-0.30 to 0.32	-0.30 to 0.32	-0.30 to 0.32

Table B.1-12: Transect 20 – Freeboard in Feet (q = 0.00003 m3/s/m; pedestrian safety).

Profile	Statistic	Historical	OPC Likely 2040	OPC Likely 2090	OPC Likely 2140	USACE High 2040	USACE High 2090	USACE High 2140
Vertical	Best Estimate	4.2	4.4	3.5	3.4	3.5	3.4	3.4
	Bounds	3.74 to 4.39	3.79 to 4.65	3.26 to 3.77	3.23 to 3.87	3.08 to 3.79	3.21 to 3.84	3.29 to 3.95
Vertical+ mound	Best Estimate	4.2	4.4	3.5	3.4	3.5	3.4	3.4
	Bounds	3.74 to 4.39	3.79 to 4.65	3.26 to 3.77	3.23 to 3.78	3.08 to 3.79	3.21 to 3.84	3.29 to 3.95
3H:1V	Best Estimate	6.7	6.7	6.7	6.7	.6.6	6.7	6.8
	Bounds	6.41 to 7.32	6.39 to 7.35	6.39 to 7.36	6.39 to 7.41	6.38 to 7.35	6.38 to 7.36	6.44 to 7.43
3H:1V+ armor	Best Estimate	3.5	3.5	3.4	3.4	3.5	3.4	3.4
	Bounds	3.06 to 3.60	3.04 to 3.58	2.99 to 3.54	2.97 to 3.50	3.03 to 3.58	2.98 to 3.52	3.96 to 3.52
20H:1V	Best Estimate	0.8	0.8	0.8	0.8	0.8	0.8	0.8
	Bounds	0.28 to 0.89	0.29 to 0.89	0.30 to 0.89	0.30 to 0.89	0.29 to 0.89	0.30 to 0.89	0.30 to 0.90
20H:1V+ vegetation	Best Estimate	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Bounds	-0.30 to 0.32	-0.30 to 0.32	0.30 to 0.32	-0.30 to 0.32	-0.30 to 0.32	-0.30 to 0.32	-0.30 to 0.32

Table B.1-13: Transect 23 – Freeboard in Feet (q = 0.001 m3/s/m; structure safety)

Profile	Statistic	Historical	OPC Likely 2040	OPC Likely 2090	OPC Likely 2140	USACE High 2040	USACE High 2090	USACE High 2140
Vertical	Best Estimate	2.5	2.5	2.6	2.6	2.5	2.6	2.7
	Bounds	1.87 to 2.74	1.90 to 2.75	1.95 to 2.78	2.02 to 2.81	1.91 to 2.76	1.99 to 2.80	2.10 to 2.85
Vertical+ mound	Best Estimate	2.5	2.5	2.6	2.6	2.5	2.6	2.7
	Bounds	1.87 to 2.74	1.90 to 2.75	1.95 to 2.78	2.02 to 2.81	1.91 to 2.76	1.99 to 2.80	2.10 to 2.85
3H:1V	Best Estimate	4.5	4.5	4.5	4.5	4.5	4.5	4.6
	Bounds	4.12 to 4.70	4.12 to 44.70	4.14 to 4.72	4.16 to 4.73	4.13 to 4.70	4.15 to 4.73	4.18 to 4.77
3H:1V+ armor	Best Estimate	2.5	2.5	2.5	2.5	2.5	2.5	2.6
	Bounds	1.86 to 2.59	1.85 to 2.59	1.88 to 2.61	1.91 to 2.62	1.86 to 2.59	1.90 to 2.62	1.94 to 2.63
20H:1V	Best Estimate	0.3	0.3	0.3	0.3	0.3	0.3	0.3
	Bounds	0.03 to 0.52	0.03 to 0.52	0.04 to 0.53	0.04 to 0.53	0.03 to 0.52	0.04 to 0.53	0.04 to 0.53
20H:1V+ vegetation	Best Estimate	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Bounds	-0.27 to 0.28	-0.27 to 0.28	-0.27 to 0.28	-0.27 to 0.28	-0.27 to 0.28	-0.27 to 0.28	-0.27 to 0.28

Table B.1-14: Transect 23 – Freeboard in Feet (q = 0.00003 m3/s/m; pedestrian safety)

Profile	Statistic	Historical	OPC Likely 2040	OPC Likely 2090	OPC Likely 2140	USACE High 2040	USACE High 2090	USACE High 2140
Vertical	Best Estimate	3.9	3.9	4.0	4.1	4.0	4.0	4.1
	Bounds	3.65 to 4.26	3.67 to 4.28	3.72 to 4.34	3.76 to 4.40	3.68 to 4.29	3.73 to 4.37	3.83 to 4.44
Vertical+ mound	Best Estimate	3.9	3.9	4.0	4.1	4.0	4.0	4.1
	Bounds	3.65 to 4.26	3.67 to 4.28	3.72 to 4.34	3.76 to 4.40	3.68 to 4.29	3.73 to 4.37	3.83 to 4.44
3H:1V	Best Estimate	6.9	6.9	7.0	7.1	6.9	7.1	7.3
	Bounds	6.44 to 7.17	6.43 to 7.20	6.53. to 7.29	6.59 to 7.37	6.44 to 7.22	6.55 to 7.33	6.65 to 7.45
3H:1V+ armor	Best Estimate	3.6	3.6	3.7	3.7	3.6	3.7	3.8
	Bounds	3.39 to 4.03	3.41 to 4.03	3.42 to 4.04	3.43 to 4.05	3.41 to 4.03	3.42 to 4.04	3.45 to 4.09
20H:1V	Best Estimate	0.8	0.8	0.8	0.8	0.8	0.8	0.8
	Bounds	0.30 to 0.88	0.31 to 0.88	0.31 to 0.89	0.31 to 0.89	0.31 to 0.88	0.31 to 0.89	0.30 to 0.88
20H:1V+ vegetation	Best Estimate	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Bounds	-0.26 to 0.29	-0.26 to 0.29	-0.26 to 0.29	-0.26 to 0.29	-0.26 to 0.29	-0.26 to 0.29	-0.29 to 0.29

B.1-5.5 Limitations of Analysis

Several limitations and caveats are associated with this wave overtopping sensitivity assessment. There are limitations in the assessment due to constraints within the scope of the feasibility study as well as data gaps. Some limitations can be reduced during the PED phase.

Individual water levels and wave height events were only available from the 1973 to 2003 FEMA hindcast period. Since 2004, there may have been combinations of water levels and wave heights that exceed those from the available hindcast. Several extreme events with high wave hazards occurred during the 2022-2023 winter storm season, resulting in structure damage along the San Francisco shoreline. These events are not reflected in the wave overtopping sensitivity assessment.

This assessment does not consider changes in future storm conditions including more severe winter storms with larger storm surge, swell, or wind waves. Higher wind speeds during storm events due to climate change could result in higher wave heights and larger wave runup events.

The extreme wave heights and extreme water level conditions are not always concurrent. Statistical analysis of the combined crest elevation facilitated an understanding of how correlated wave and water level extrema were, but introduced some statistical variation in the EVA where the freeboard estimates have some degree of uncertainty. Both the best estimate and potential lower and upper ranges for freeboard estimates are provided.

The current shoreline profiles from the FEMA San Francisco Bay Area Study were used as-is, bayward from the shoreline toe location. No change in the shoreline profile over time was considered with the sea level rise scenarios, either due to deposition or erosion of sediment or potential dredging activities. Sediment deposition in the foreshore would reduce the ratio of water depth to wave height, potentially triggering wave breaking or impulsive wave conditions leading to higher wave runup, and subsequently higher minimum required shoreline crest elevations.

A simplifying assumption was made to assume marsh vegetation tracks with sea level rise on the shoreline profile (e.g., S. Pacifica tracks accordingly higher with the shift in the Mean Low Water tidal datum). This assumption was reasonable for this wave overtopping sensitivity analysis but should be further refined in subsequent design phases.

This assessment does not consider other parameters relevant for evaluating overtopping hazards, including frequency of overtopping during extreme storm events, or the total volume of overtopping during a storm event.

The freeboard heights presented in **Table B.1-9** to **Table B.1-14** represent the additional height above the 1% AEP SWEL needed to account for wave runup and limit hazardous overtopping. These freeboard heights do not apply to other return frequencies of other SWELs; however, the methods used in this assessment can support developing freeboard height estimates for additional return frequencies.

B.1-5.6 Considerations for Project Engineering and Design Phases

Future refinements to this wave overtopping sensitivity assessment could include:

Refine shoreline profiles to better represent the flood protection measures, including wave dissipation features, in the PED phase.

Evaluate additional shoreline locations and engineered shoreline slopes, including armoring configurations, to capture a wider range of foreshore and shoreline conditions for optimization of wave runup reduction benefits.

Incorporate sensitivity assessment of larger storms occurring due to climate change, which may increase required minimum shoreline crest elevations.

Evaluate additional overtopping thresholds and evaluate minimum crest elevations relative to 1% TWL and Maximum Wave Runup.

Refine vegetation assumptions on natural shoreline slope (e.g., suitable vegetation types and zones relative to local tidal datums as they shift with sea level rise).

Consider a broader range of Natural and Nature-Based Features on a wider variety of shoreline types.

Section B.1-6 Hydrology and Hydraulics Interior Drainage Analysis

B.1-6.1 Purpose of Analysis

In accordance with USACE Engineering Manual (EM) 1110-2-1413, Hydrologic Analysis of Interior Areas, the impacts to interior drainage were evaluated to determine the required interior drainage features for various alternatives to provide interior relief, such that, during storm events, the CCSF does not see substantial flooding beyond what it would with the current local storm drainage system without a project in place. The design criteria for the San Francisco combined stormwater system are to meet the level of service requirements of having freeboard with the collection system of a 20% AEP 24-hour storm and 1% AEP 24-hour storm overland flow for street conveyance. A 20% AEP 3-hour storm is the level of service for project identification and prioritization purposes.

When evaluating the interior drainage, the surface flows become a larger component of the runoff for the less frequent, but larger precipitation storms. Smaller low intensity storms can be handled by the combined storm sewer system either through discharge from the water treatment plants and the control structures when flows exceed the water treatment plant capacities.

The study approach for assessing the interior drainage is evaluated primarily using the HEC-RAS 2D modeling software. This software is used to evaluate the without project conditions and various project alternatives and the impacts of those alternatives on the interior drainage of the Bayside area. In addition, the SFPUC and SFPW completed a high-level analysis of the with and without project conditions using the CCSF InfoWorks ICM Model. The CCSF19 model results are used in this study to inform the response the SFPUC and SFPW would make regarding future climate conditions if no coastal defense was constructed.

For more detail on the Hydrology and Hydraulics interior drainage analysis refer to the *Sub-Appendix B.1.4*.

B.1-6.2 San Francisco Bayside Urban Watersheds

The Bayside drainage area is the only portion that would impact the study area. Of the five bayside watersheds, three of them have surface run off that would directly impact the study area. The three watersheds (North Shore, Channel, and Islais Creek) are included in this analysis. The division of the Bayside drainage area and the San Francisco Feasibility Study area are shown on **Figure B.1-20**.

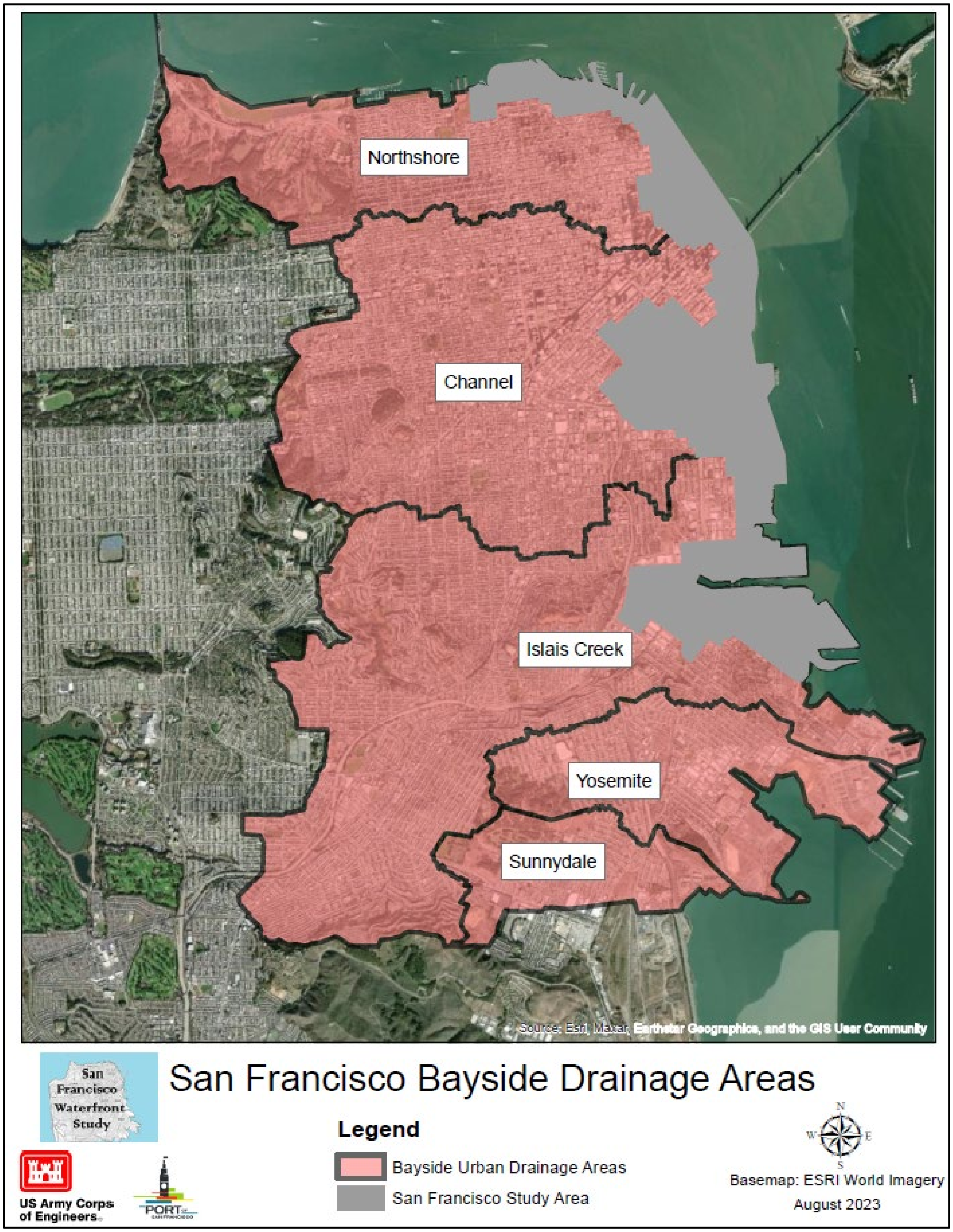


Figure B.1-20: Bayside Drainage Area Urban Watersheds with Study Area Extents

B.1-6.2.1 Rainfall-Tide Correlation Assessment

Two types of storm categories produce most of the coastal hazards and flooding in San Francisco. These are extratropical cyclones and atmospheric rivers. Both types of storms bring on high winds and heavy rains. For the interior drainage analysis interior and exterior boundary conditions need to be evaluated and applied to the appropriate storm size.

For the with and without project conditions, understanding the Bay impacts to storm water runoff is an important factor when analyzing the interior drainage. A rainfall-tide correlation analysis is done as part of this study to help define the boundary conditions for the exterior Bay elevations for the interior drainage analysis. In addition to the interior drainage analysis the assessment provides historical insight into the tidal conditions and large rainfall events that could lead to flooding in the area. For this study the peak annual tide and maximum 24-hour annual rainfall as well as 24-hour rainfall, 6-hour rainfall, 3-hour rainfall and the accompanying peak tides for those intervals are evaluated to assess the interior and exterior relationship. The SFPUC and the SFPW use 3-hour intensities for design considerations.

Data collected from the San Francisco Tidal gage (9414290) and Downtown San Francisco NOAA gage (COOP:047772) was used for the assessment. A 118-year period of record (1901-2018) was used for the tidal gage and 110-year period of record (1908-2018) was used for the precipitation gage.

The review of the peak annual tides shows that in many cases, occur at times where there are small rainfall events within 24 hours. When looking at larger period of record data, there are times when moderate rainfall events fall on days where high tides reached levels near the 99% AEP of 7.1 feet, which is not unexpected. However, when looking at the shorter duration higher intensity events, such as 3 hours or 6 hours, there is variability in the tidal conditions for those shorter timeframes, which shows variability on timing of the rainfall to the high tidal conditions. The seven additional events selected reviewed for coincidence of tides and rainfall indicate that there are times when large rainfall events can occur during high tides that are above MHHW such as January 2023, but most of the time the moderate to large rainfall events occur tides lower than MHHW such as December 2014 and November 2001.

The median peak tide for the annual maximum 24-hour rainfall events from 1998–2017 timeframe evaluated is 6.3 feet, which is approximately 0.4 foot higher than MHHW for the Presidio tide gage. The 6.3 feet is only the peak tidal reading on the day the maximum 24-hour rainfall occurred and does not take into account the coincidence of the two.

The review of the data shows a low to moderate rain-tide correlation when evaluating peak annual rainfall and tides, and some dependence in regard to coincident timing. The interior drainage model assumes that moderate to high rainfall events could occur at or near a mean high water or a mean higher high tide, so the controlled outlet locations along the Bay could be reduced due to tidal conditions.

B.1-6.3 San Francisco Stormwater Management Systems

The CCSF controls storm water runoff largely (90% of the entire system) through a combined storm and sanitary sewer system that utilizes three wastewater treatment plants throughout the city to treat the runoff. When storage and treatment capacity is exceeded, the system discharges by gravity at the combined sewer discharge (CSD) outfall structures. The system is comprised of 1,000 miles of sewers, 19 pump stations, 8 storage structures and 36 outfall locations. The design criteria for the San Francisco combined stormwater system are to meet the level of service requirements of having freeboard with the collection system of a 20% AEP 24- hour storm and 1% AEP 24-hour storm overland flow for street conveyance. A 20% AEP 3-hour storm is the level of service for project identification and purposes.

The two main drainage basins, Westside and Bayside, flow to the wastewater treatment plants and outfall locations. The Bayside drainage basin is the only runoff that impacts the San Francisco Feasibility Study area and is approximately 30 square miles. The Bayside drainage area is comprised of five smaller drainage areas North Shore, Channel, Islais Creek, Yosemite, and Sunnydale. Yosemite and Sunnydale watersheds are not included as part of the assessment as they are not part of the study area and during wet weather operations the overflow for those basins are handled in the area. The Marina District contained within the North Shore basin is located outside of the study area and interconnectivity of areas protected and unprotected from coastal flooding will require further assessment at later phases of the study. An allowance was made to account for the flow pumped toward the wastewater treatment plant from these watersheds. More detail on the Bayside network is shown on **Figure B.1-21**.

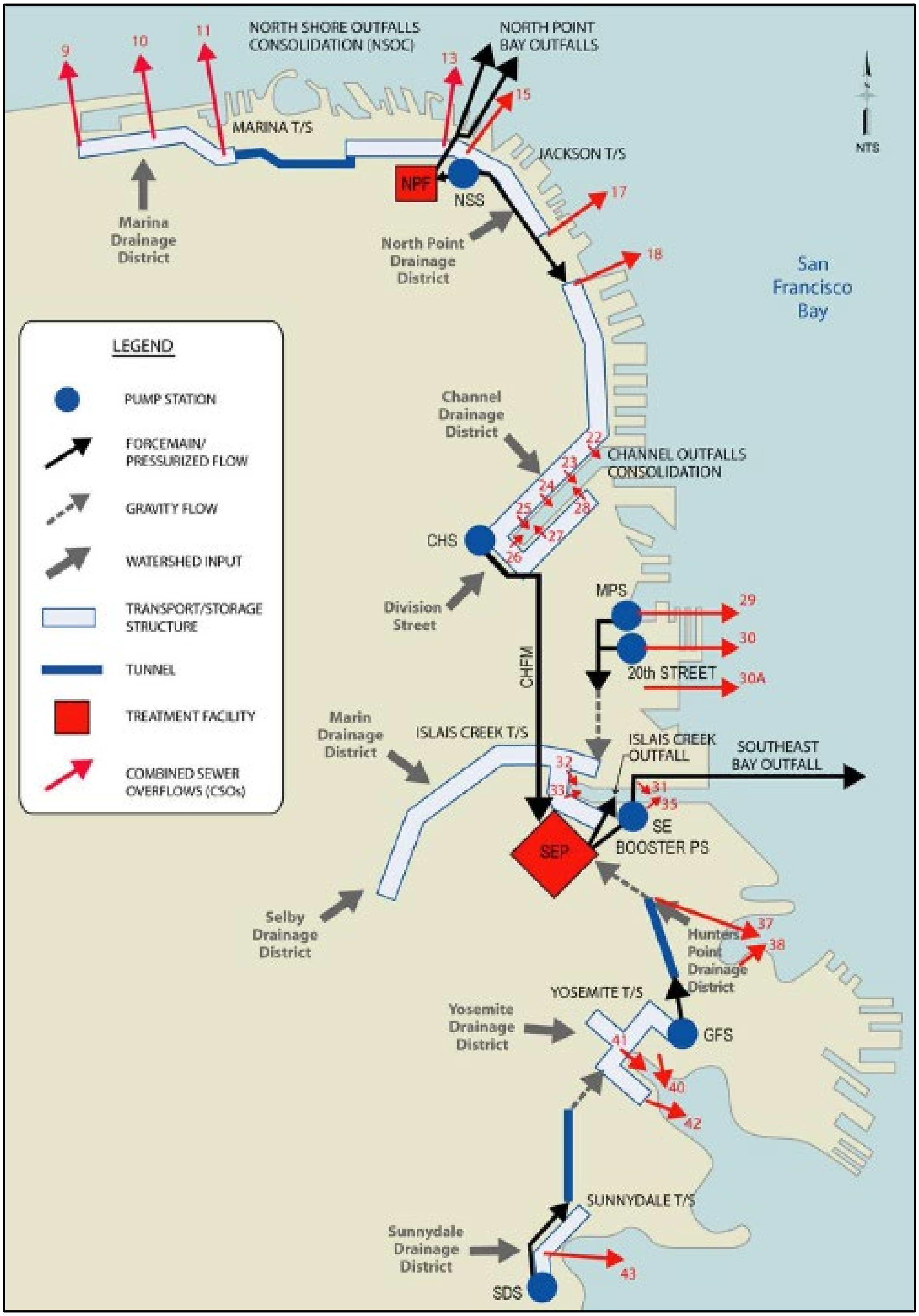


Figure B.1-21: Bayside Combined Storm Sewer Schematic

According to the SFPUC maps, approximately 1% of parcels citywide are subject to flooding greater than 6 inches during a 1% AEP 3-hour storm. Approximately half of these are residential parcels, while the rest are a combination of commercial, industrial, and public parcels. Many of the properties in the 100-Year Storm Flood Zones are built along historical waterways that used to be creeks or wetlands.

B.1-6.4 Hydraulic Model Development

B.1-6.4.1 Methodology

A 2D HEC-RAS model was developed to assess the interior drainage for the with and without project conditions. HEC-RAS version 6.3.1 was used to develop the overland inundations and flow estimates for the various conditions. HEC-RAS cannot model underground pipe networks, so to account for the combined sewer system artificial losses from the rainfall. The reduced rainfall was directly applied to the model for the various scenarios. Losses based on land use and soil type were captured using the deficit constant method and percent impervious parameters were assigned based on the land classification layer provided by the SFPUC.

B.1-6.4.2 Digital Elevation Model

LiDAR data was available for the entire San Francisco area. The DEM generated from the 2010 LiDAR survey at a 1-foot resolution. The DEM was provided by the SFPUC and is consistent with what is currently used their ICM model.

B.1-6.4.3 HEC-RAS Geometry Development

The HEC-RAS Geometry was built in the NAD83 StatePlane California III FIPS 0403 Feet projection. The precipitation with the combined storm sewer estimates removed through artificial losses, was directly applied to the HEC-RAS model 2D mesh areas to calculated overland flow.

The geometry was split into two separate 2D areas, one for the area that is primarily served by the northern water treatment plant and one for the area that is primarily served by the southern water treatment plant. This was done so simplified combine storm sewer operations could be incorporated.

Loss rates and impervious areas were defined for the mesh to determine the runoff for the area. The Land Cover layer used in the CCSF19 ICM model was used for the HEC-RAS model. This layer includes all existing conditions, green infrastructure implemented through the Stormwater Management Ordinance from 2010 through 2032. Loss rates are computed in the HEC-RAS model 2D model using the deficit and constant method. Initial infiltration rates are developed based on soil types and in the HEC-RAS 2D User's Manual.

B.1-6.4.4 HEC-RAS Existing Conditions Evaluation

Limited data was available for use in calibrating the HEC-RAS model. The model was run for three historic events to evaluate the flow paths in the bayside drainage areas. These events are November 1994, December 2014, and December 2022. In addition to the three historic events the 1% AEP 24-hour NOAA rainfall is simulated and compared to the results of the SFPUC's ICM model. The SFPUC provided images of the December 2022 event that were submitted through the 311-information system during the event and results from the ICM model for the NOAA Best Estimate 1% AEP 24-hour storm with the 50% AEP storm surge to aid in calibration of the model and gain a better understanding on how the HEC-RAS model performed using artificial loss rates for the combined storm sewer system estimates.

The HEC-RAS model uses the same DEM and Land Cover layers as the InfoWorks ICM model to allow for consistency in modeling. The SFPUC and SFPW provided multiple validation point locations that are for comparison and points of interesting in the area. As expected, the HEC-RAS model tended to overestimate the flooding in the study area due to the limitations of not being able to model the underground combined storm sewer network.

The loss rates and manning's values from the initial geometry development have been refined during this stage based on the historic events and discussions with the SFPUC.

B.1-6.5 Future Without Project

As sea levels rise, the potential for inland flooding from the collections system and overland flows increases. These impacts to the combined storm sewer system will begin to be seen when the outfall weirs start to become submerged, either temporarily during coastal storm surge or permanently due to sea level rise. This will reduce the capacity to discharge at the CSD locations, causing additional flooding. As sea levels rise to higher levels there is potential to inundate portions of the collection system infrastructure causing further inland flooding. The SFPUC estimates that 24 inches of sea level rise above the 2-year storm surge levels, approximately 7.4 feet, the overall function of the collection system will be impacted and cause flooding in the city, specifically in low-lying areas.

In addition to the impacts on the collection system, as sea levels rise, overland flow will lose the ability to outlet into the Bay. Once the sea levels rise to near current shoreline levels the exterior map condition will start to act as a wall causing the overland flows to backflow into the lower portions of the city.

B.1-6.5.1 SFPUC Future Without Project Analysis

In the absence of a Coastal Flood Risk Management, the SFPUC completed a high-level modeling analysis utilizing their CCSF19 ICM model to better understand how the combined sewer system could potentially perform in a future when large rainfall events fall over the city with rise in sea levels and no coastal defense.

The CCSF19 ICM model used the best estimate NOAA 1% AEP rainfall and the 50% AEP storm surge for the present-day baseline scenario. The future sea level scenarios included increased rainfall depth and intensity based on the recent SFPUC Extreme Precipitation Study. **Figure B.1-22** shows the hypothetical action in response to 7 feet of sea level rise in 2100. The conceptual design for the FWOP condition for 7 feet of sea level rise was evaluated using the following rainfall assumptions:

- 2023 (Baseline)
  - 3.1* inches/hr 5-yr/3-hr storm
  - 4.7* inches/hr 100-yr/3-hr storm
- Year 2050:
  - +20% in 5-yr/3-hr storm
  - +26% in 100-yr/3-hr storm
- Year 2100:
  - +56% in 5-yr/3-hr storm
  - +67% in 100-yr/3-hr storm

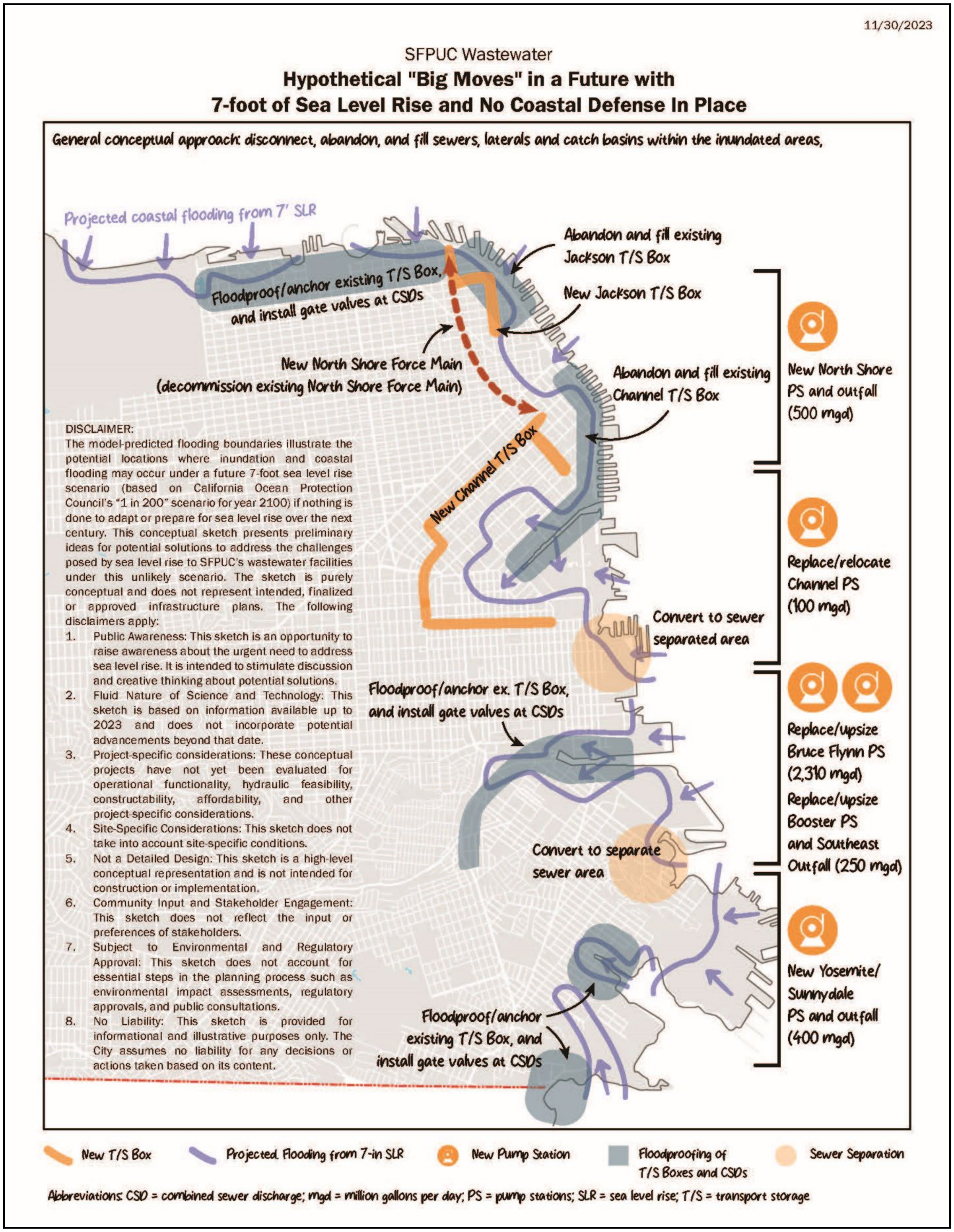


Figure B.1-22: FWOP HEC-RAS Geometry

B.1-6.5.2 Future Without Project HEC-RAS Model

For the FWOP condition it is assumed that only the two bayside water treatment plants would be continuously operating and that the large storage/transport boxes will maintain the current capacity in the future. As sea levels rise the CSD locations will likely see reduced capacity causing flooding in low-lying areas. **Figure B.1-23** shows the FWOP HEC-RAS geometry layout. This geometry is used as the basis for the FWP alternative evaluation.

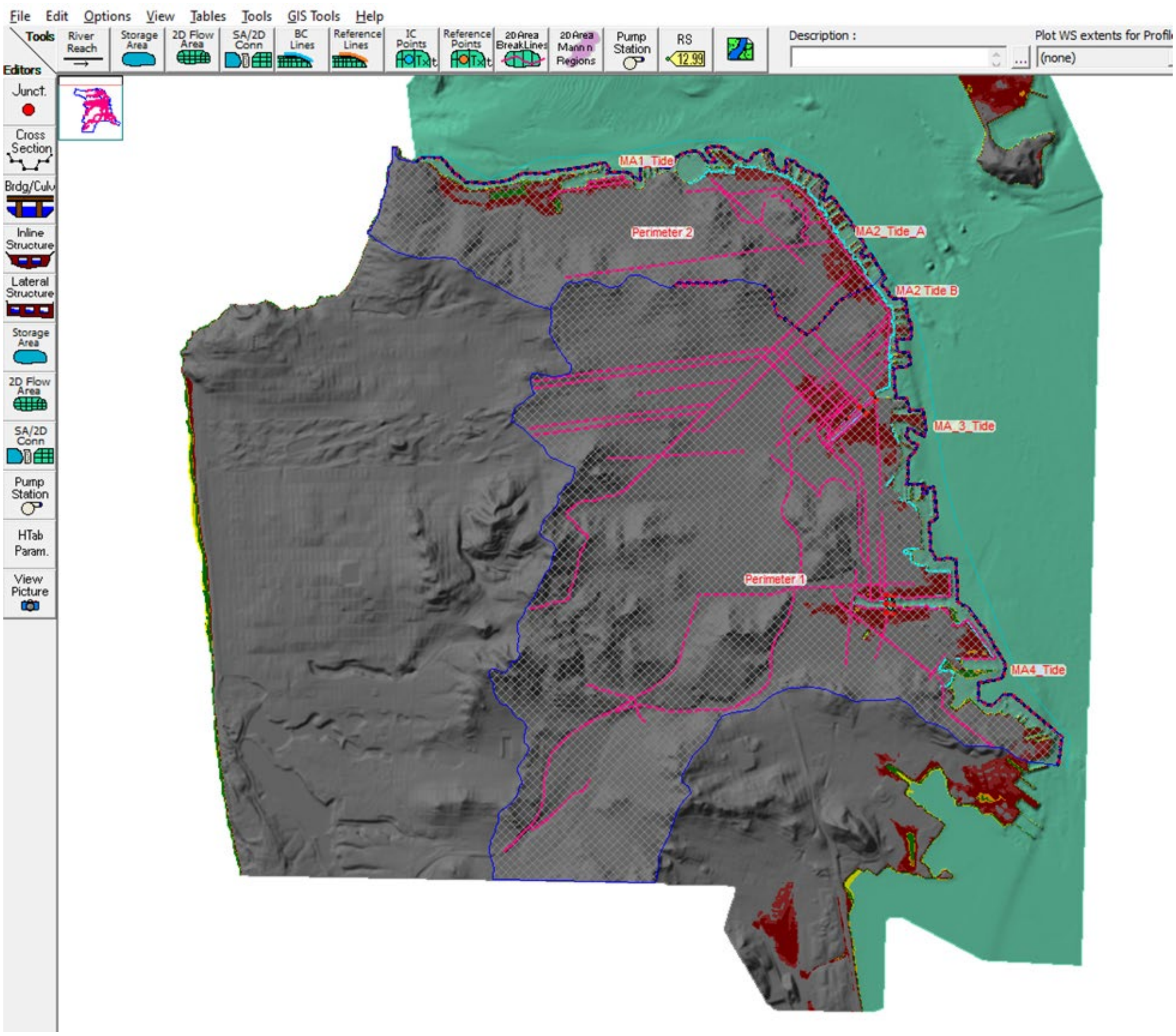


Figure B.1-23: FWOP HEC-RAS Geometry

B.1-6.6 Interior Drainage Assessment of Project Alternatives

Seven project alternatives have been evaluated as part of the feasibility study, however only five of the seven would have impacts to interior drainage with the addition of coastal protection structures. The evacuation of flood waters from the behind the coastal protection was handled with either gravity flow structures, water management structures or pumps.

B.1-6.6.1 Structure Placement and Sizing for the Future With Project Alternatives

The analysis for the federal responsibility for interior drainage impacts is completed using the HEC-RAS FWOP model as a basis. The NOAA 90% Upper Bounds 24-hour 1% AEP rainfall and a base Bay level MHHW design conditions are used to determine the federal responsibility. Sea level rise is applied based on the alternative and whether it is a first or second action, this varied from 1.5 feet to 7 feet for the alternatives.

The pump and culverts with backflow prevention are placed in locations that would provide the most relief due to flooding based on low spots in the terrain.

With the increased rise in sea level, although more area will be inundated, the same rainfall volume will still be impacting the city, except for alternatives with some retreat, such as Alternative G, in which there will be a slight reduction in volume due to the reduced area. Initial volumes and peak flows for each reach were estimated using flow lines in the HEC-RAS model for the FWOP with MHHW alternative. The flow lines provided an initial estimate as to the total volume and peak flows of water that would reach the Bay during a 1% AEP event which would ultimately be blocked by any line of protection for the project. Using those estimates, pumps and culverts were iteratively placed along the LOD and throughout the study area to achieve the greatest amount of relief for the area. An attempt was made to keep structure placement similar for each alternative wherever possible. Validation points in the study area were used to evaluate the size and placement of the structures.

Alternative C and F incorporate gravity flow systems. With these alternatives SLC will have an impact on the duration of inundation for the FWP conditions. For these alternatives the SLC adjustments of 1.5 feet and 3.5 feet were taken into consideration when evaluating the structures but no structures were added to remove any flood depths below the respective FWOP condition due to Bay level impacts. With MHHW plus 1.5 feet of SLC, there is very similar inundation and flood depths to MHHW. Alternative F first action, MHHW plus 3.5 feet, will see more long duration inundation primarily in Reach 3 and 4 which have the tide gates.

Pump locations have been placed in the natural low spots where ponding occurs currently throughout the system. Between TSP and the Final report, additional refinement will take place working with the SFPUC to utilize the current network, likely at the storage and transport boxes, and identify if additional collection areas may be needed beyond the current capacity. Pump and culvert locations and sizing estimated in the feasibility study should be reevaluated during PED.

B.1-6.6.2 Summary of Results

The SFPUC indicated areas in the study area that would be good validation points for the evaluation of the assessment of the FWOP conditions and the pumps, culverts, and tide gates for each alternative. The validation points are listed in **Table B.1-15**.

Table B.1-15: Validation Point Locations and ID

Validation Point Name	Point ID	Reach
Beach & Mason	0	Reach 1 (Northshore)
Davis & California	2	Reach 2 (Northshore)
Embarcadero & Broadway	3	Reach 2 (Northshore)
Evans & Selby	4	Reach 4 (Islais Creek)
Henry Adams & Division	5	Reach 3 (Channel)
Marin & Indiana	6	Reach 4 (Islais Creek)
Merlin & Morris	7	Reach 3 (Channel)
Townsend & 5th Street	8	Reach 3 (Channel)

A comparison to the FWOP condition of the validation points for each alternative is shown below in **Table B.1-16**. Values are shown in the difference from FWOP, with negative values being flooding depths less than FWOP for results from the HEC-RAS modeling.

Table B.1-16: Difference of Each Alternative Compared to FWOP

Validation Point ID	Alternative D First Action and Alternative C (feet)	Alternative D Second Action and Alternative E (feet)	Alternative F First Action (feet)	Alternative F Second Action (feet)	Alternative G First Action (feet)	Alternative G Second Action (feet)
0	0	-0.1	-0.1	-0.1	-0.1	0
2	0	0	0	0	0	0.1
3	0.3	0	0	0	0	0
4	0.1	0.2	-0.1	0	-0.4	0.6
5	0	0	0	0	0	0.5
6	-0.3	0	0	0.5	N/A	N/A

Validation Point ID	Alternative D First Action and Alternative C (feet)	Alternative D Second Action and Alternative E (feet)	Alternative F First Action (feet)	Alternative F Second Action (feet)	Alternative G First Action (feet)	Alternative G Second Action (feet)
7	0.5	0.5	-0.1	0.4	0.5	0.2
8	0.6	0.6	-0.1	0.4	0.6	0.3

B.1-6.7 Total Net Benefits Plan

The TNBP was a hybrid of various alternatives. The final alignment consisted of structural measures that protect to 15.5 feet in Reach 2, 13.5 feet for Reaches 3 and 4, and nonstructural measures on Reach 1 during the first action. During the second action, structural measures are added to all to reaches to protect up to 15.5 feet.

During the first action culverts will be utilized as long as possible such as in the Defend, Scaled for Lower Risk alternative first action (Alternative C) and Defend, Scaled for Low-Moderate Risk alternative (Alternative D). It is assumed that no interior drainage requirements are needed in Reach 1, outside of adding flap gates to the current CSD outfall locations, since only nonstructural measures are in place. During the second action, all pumping requirements needed for the Defend, Scaled for Low-Moderate Risk alternative second action will need to be implemented. A summary of interior drainage features for the first and second action of the total benefits plan are shown in **Table B.1-17** and **Table B.1-18**.

The interaction of structural and nonstructural actions as well as the hydraulic connection of the combined sewer system in the reaches to the Marina District outside of the study area will need to be evaluated further between the TSP and the final report. Refinements will be included in the final report.

Table B.1-17: TNBP First Action Summary of Interior Drainage Features

TNBP First Action	HEC-RAS Interior Drainage Estimates
Reach 1 (North Shore)	Nonstructural – No improvements
Reach 2 (North Shore and Channel)	9 - 4x3 Box Culverts with backflow prevention 3 - 4x3 Box Culverts with backflow prevention New flap gates on existing CSDs
Reach 3 (Channel)	1 - 600 cfs pump (390 MGD) 1 - 200 cfs pump (130 MGD) 13 - 4x3 feet Box Culverts with backflow prevention 2 -4x2 feet Box Culverts with backflow prevention 1 - 3 feet Circular Culvert with backflow prevention

TNBP First Action	HEC-RAS Interior Drainage Estimates
	New flap gates on existing CSOs
Reach 4 (Islais Creek)	1 - 400 cfs pump (260 MGD) 1 - 300 cfs pump (190 MGD) 31 - 4x3 Box Culverts with backflow prevention 1 - 4x2 Box Culvert with backflow prevention 2 - 3x2 Box Culverts with backflow prevention 2 - 2x2 Box Culverts with backflow prevention New flap gates on existing CSOs
Total Pump Flow	1500 cfs (970 MGD)

Notes:
cfs = cubic foot (feet) per second
MGD = million gallons per day

Table B.1-18: TNBP Second Action Summary of Interior Drainage Features

TNBP Second Action	HEC-RAS Interior Drainage Estimates
Reach 1 (North Shore)	1 – 300 cfs pump (200 MGD)
Reach 2 (North Shore and Channel)	1 – 380 cfs pump (250 MGD) 1- 350 cfs pump (230 MGD)
Reach 3 (Channel)	2 – 150 cfs pump (100 MGD) Upsize from 600 cfs to 1350 cfs pump (new 750 cfs/480 MGD)
Reach 4 (Islais Creek)	2 – 150 cfs pump (100 MGD) Upsize from 400 cfs to 1900 cfs pump (new 1500 cfs/970 MGD) Upsize from 300 cfs to 1400 cfs pump (new 1100 cfs/710 MGD)
Total New Pump Capacity	4980 cfs (3240 MGD)

Notes:
cfs = cubic foot (feet) per second
MGD = million gallons per day

B.1-6.8 Sensitivity Analysis

Three sensitivity analysis were run for all the alternatives. The first was compressing the 24-hour rainfall into a 3-hour rainfall duration, greatly increasing the intensity of the storm. The second was looking at using the best estimate 1% AEP NOAA 24-hour rainfall to determine how that compared with the design criteria used by the SFPUC and

SFPW. The last sensitivity analysis run was for the Defend, Scaled for Low-Risk alternative (Alternative C). This looked at the 2-year storm surge elevation used in the San Francisco ICM model with 1.5 feet of SLC compared to the MHHW elevation with 1.5 feet of SLC that was used for the study.

When the rainfall intensity is drastically increased to have the 24-hour rainfall volume occur over 3 hours, the peak flood depths increase by 0.6 to 4 feet using the current pump sizing for some alternatives, with the greatest impacts along Mission Creek and Islais Creek. When evaluating the proposed pump and culvert sizing using the Best Estimate 1% AEP 24-hour duration the results for each alternative brought the water levels back to the best estimate FWOP condition with many areas better than FWOP. The exterior condition evaluation which looked at the 2-year storm surge instead of the MHHW elevations with 1.5 feet of sea level rise yielded similar results for both conditions. Validation point 6, at Marin and Indiana, was the only validation area that did not see the water levels recede back to without project conditions after the peak when the 2-year storm surge Bay elevation with 1.5 feet of SLC was used. This would not likely cause an issue if the storage/transport boxes in the system were included as structures in the model itself allowing the pumps on the south side of the creek to pump out the remaining water. Incorporation of the combined storm sewer system should be explored further during PED.

Section B.1-7 Groundwater Assessment

B.1-7.1 Purpose of Assessment

USACE guidance linkage. Coastal flood hazards within the study area are directly linked to the current and future presence of groundwater below the ground along the San Francisco shoreline. As the relative sea level rises within the Bay, the groundwater table is expected to rise, first intersecting buried infrastructure with eventual emergence impacting surficial drainage, infrastructure, and operations.

This groundwater impacts summary leverages recently completed reports that estimate the depth and extent of the existing groundwater table to qualitatively define the groundwater challenges and potential impacts of proposed flood protection alternatives.

B.1-7.2 San Francisco Bay Hydrologic Region

The San Francisco Bay Hydrologic Region covers approximately 4,500 square miles and includes all of San Francisco and portions of Marin, Sonoma, Napa, Solano, San Mateo, Santa Clara, Contra Costa, and Alameda Counties. The San Francisco Hydrologic Region is the smallest of the ten hydrologic regions in the state but is the second most populous with the cities of San Francisco, San Jose, and Oakland in the basin. The extents are shown on **Figure B.1-24**.

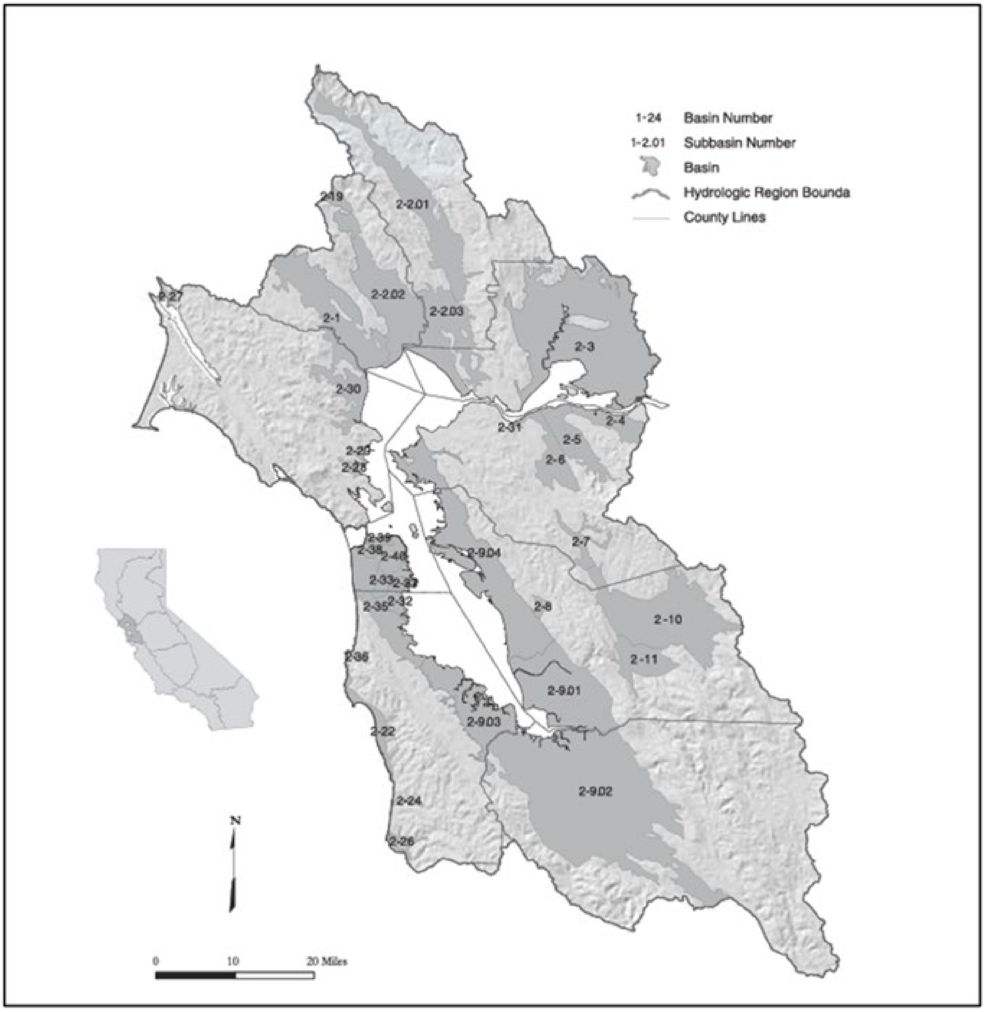


Figure B.1-24: San Francisco Bay Hydrologic Region

There are 28 identifiable groundwater basins in the San Francisco Hydrologic Region totaling 1,400 square miles, or approximately 30% of the hydrologic region. The groundwater basins in this region are predominately defined by valleys formed on alluvial fans.

There are seven major groundwater subbasins identified in San Francisco, three of which directly impact the San Francisco east side waterfront. These are the Marina, Downtown, and Islais Valley groundwater basins.

The Marina and Downtown basins primary water-bearing formations are comprised of unconsolidated sediments and include alluvial fan deposits, beach and dune sands, undifferentiated alluvium, and artificial fill. The artificial fill is mostly dune sand but also contains silt, clay, and various natural man-made debris. The greatest depths to bedrock are less than 300 feet with many portions of the Downtown basin less than 200 feet. Historic drainage of the Downtown basin was likely a creek system made up of two drainage valleys.

Islais Valley is different with primary water bearing formations of bedrock and unconsolidated sediments. The unconsolidated sediments are comprised of dune sand, the Colma Formation, bay mud and clay, and artificial fill. The Colma Formation consists of fine-grained sand, silty sand, and discontinuous beds of clay up to 5 feet in thickness. The artificial fill is similar to that of the Marina and Downtown basins. The greatest depths to bedrock are less 200 feet deep. The groundwater likely flows in the path of old Islais Creek to the Bay.

With many areas of the Marina, Downtown and Islais Creek basins having water bearing formations being relatively shallow at less than 200 feet, this would indicate there may be a low storage capacity for groundwater once infiltrated into the soil (Bulletin 118).

B.1-7.3 San Francisco Bay Municipal Groundwater Supply

There is no municipal groundwater pumping on the Eastside San Francisco groundwater basins that would have impact to the study area. The SFPUC has started using groundwater sources with the use of groundwater blending to supplement the municipal water supply for the area from the Westside groundwater basin. The groundwater supply is taken from approximately 400 feet below the surface. The Westside Basin is a series of aquifers extending from Golden Gate Park in San Francisco southward through San Bruno totaling an area of 45 square miles.

B.1-7.4 Hydrologic Controls

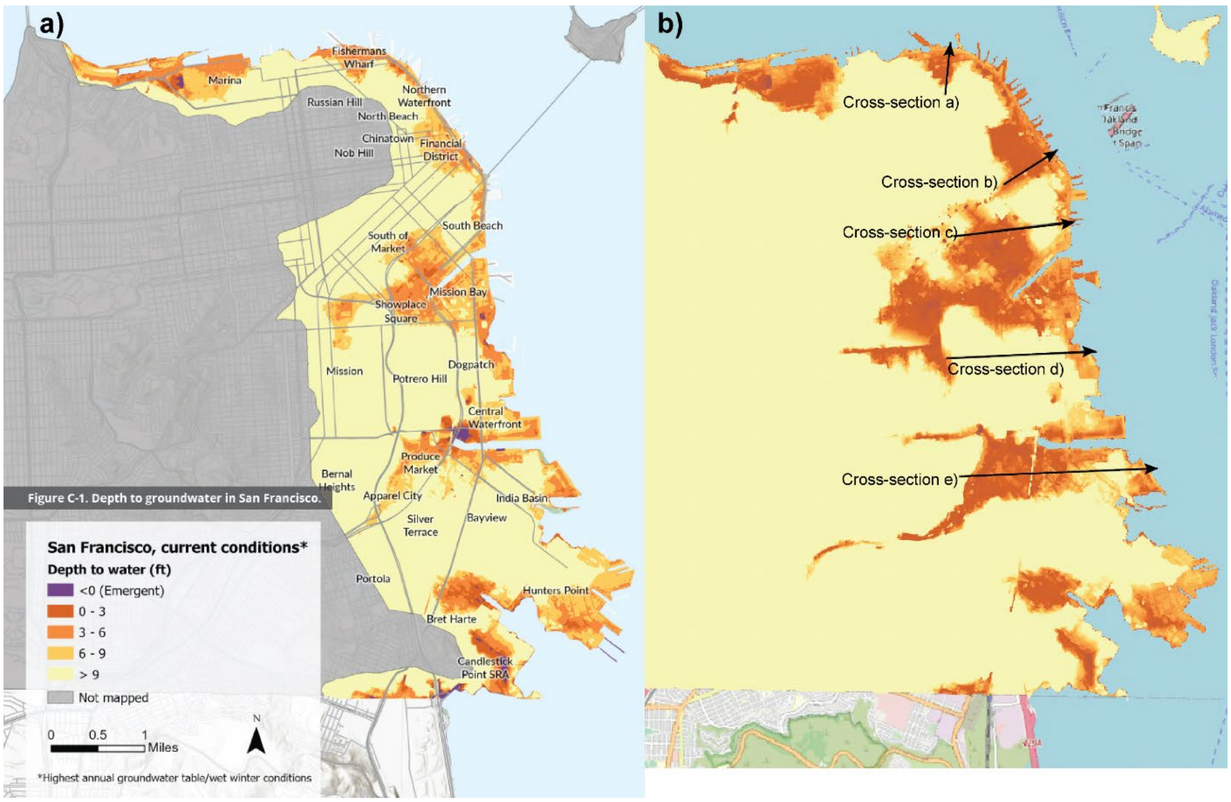
Groundwater in the study area have sources that are both topographically limited systems and flux-controlled systems. With topographically limited systems there is little room for response to sea level rise, while flux-controlled systems generally rise linearly with sea level (Befus et al. 2020; Michael et al. 2013).

For the unconfined systems on the eastside, an increase in water storage occurs primarily through raising the water table and filling additional unsaturated pores above the water table with water. For the eastern half of San Francisco, the groundwater recharge rate has been simulated to be approximately 0.57 foot per year considering both precipitation and water/sewer line losses (Phillips et al. 1993).

B.1-7.5 Existing Groundwater Depth Analysis for San Francisco Waterfront

Two assessment techniques have been applied to understand how and where groundwater levels may change with sea level rise over the study area within larger regional analyses (e.g., Befus et al. 2019). The **empirical mapping technique** interpolates maximum well water level observations to produce maps of the shallowest historic groundwater levels that are raised by the amount of sea level rise (May et al. 2019; Plane et al. 2019). The **numerical modeling technique** solves the mathematical equation of groundwater flow to produce maps of forecasted groundwater levels based on hydrologic and geologic inputs to the model (Befus et al. 2020; May et al. 2019). Additional details comparing these methods can be found in May et al. (2019).

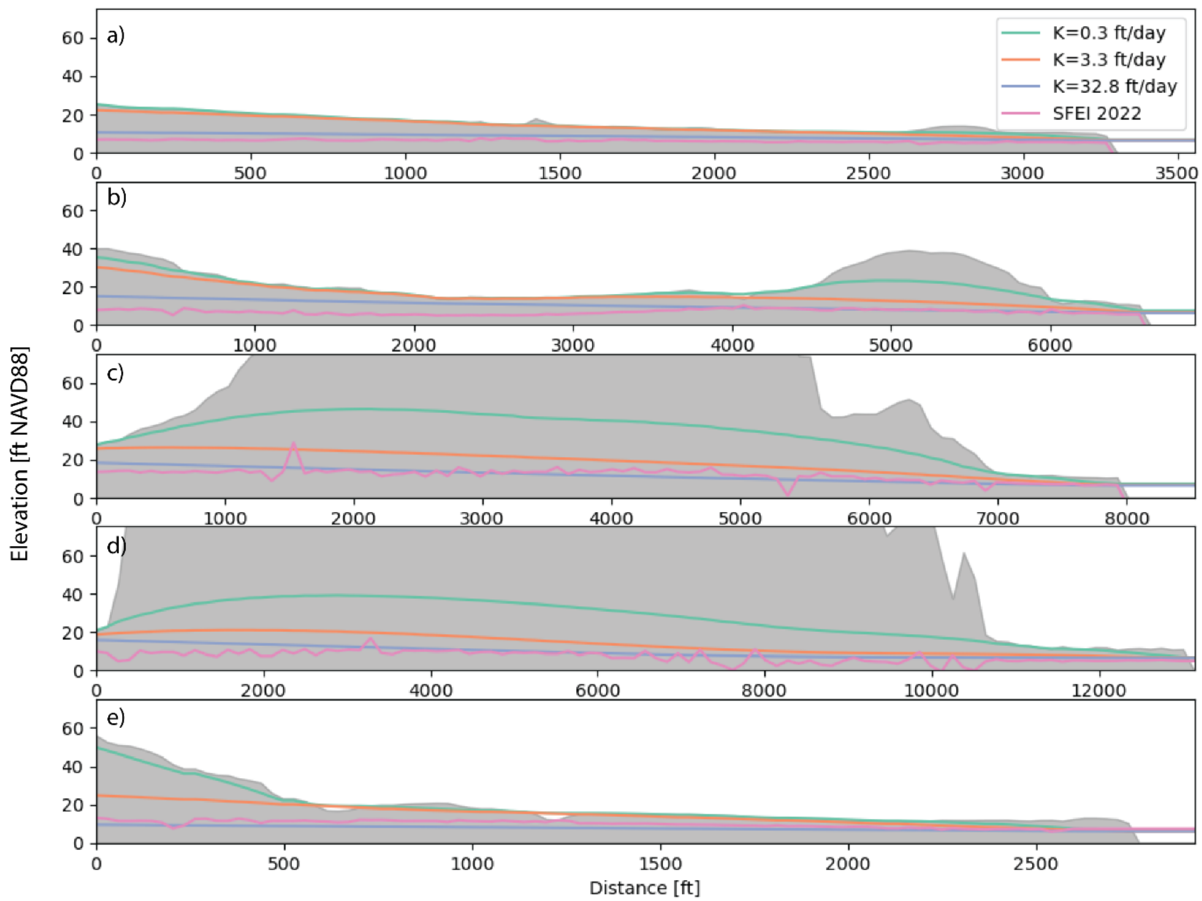
Both techniques indicate the presence of emergent and very shallow groundwater (less than 3 feet deep) for existing conditions within the study area and expansion of these areas with sea level rise, as shown in both plan view (**Figure B.1-25**) and cross-section (**Figure B.1-26**).



Source: (C. L. May et al. 2022; K M Befus et al. 2020)

Figure B.1-25: Existing Depth to Groundwater Maps for the San Francisco Study Area

Map views of (a) the empirical mapping technique results for the present-day depth to the water table using highest observed water table levels (May et al. 2022), and (b) the numerical modeling technique results for the present-day depth to the water table with a homogeneous hydraulic conductivity of 3.3 feet per day (1 meter per day).



Source: (C. L. May et al. 2022; K M Befus et al. 2020a)

Figure B.1-26: Cross-section Views of the Present-Day Water Table Elevation

Cross-section views of the present-day water table elevation for both the empirical mapping technique (May et al. 2022) and the numerical modeling technique with three values of hydraulic conductivity (K) (Befus et al. 2020). The locations of these profiles are indicated on Figure B.1-25.

B.1-7.6 Groundwater Response to Sea Level Rise

It is expected that the existing topography limited groundwater system along the San Francisco Waterfront will rise in response to sea level rise in both the future without project and future with project conditions. The following sections include descriptions of the groundwater issues and challenges that are expected to occur coincident with sea level rise.

B.1-7.6.1 Groundwater Shoaling and Emergence

Groundwater shoaling occurs when a water table gains elevation and becomes shallower from the land surface. Groundwater emergence occurs when the water table intersects the land surface, resulting in either the formation of a new spring, seep, ponding, or evaporative deposit, depending on the nearby climate and topography. With sea level rise, the water table at the shoreline will rise to meet the new sea level. Shallow and emergent groundwater represent hazards for surficial flooding, water quality, transportation, and shallow buried infrastructure (Habel et al. 2017; Knott et al. 2017; 2018; Rotzoll and Fletcher 2013; Su et al. 2020; May 2020). Tides create additional complexity, where the amount of porewater exchange and water level responses caused by the tide is dependent upon the local hydrologic and geologic setting and hydraulic parameters discussed earlier (e.g., Abarca et al. 2013). Over seasons, groundwater tables fluctuate based on the seasonality in recharge set by infiltrating rainfall amounts, with seasonal water level changes of about 3 feet observed in Alameda (May et al. 2022) that suggests similar seasonality could exist in the shallow groundwater along the San Francisco waterfront.

B.1-7.6.2 Saltwater Intrusion

Saltwater intrusion, which is more technically termed saline groundwater intrusion, is caused by the infiltration of groundwater with higher salinities into water-bearing units with previously lower salinities. Causes of intrusion are primarily from hydrologic changes, including pumping, less recharge, and/or sea level rise. Pumping from the unconfined system is expected to cause more intrusion than century-scale sea level rise (Ferguson and Gleeson 2012), although local hydrologic and geologic variability may lower the importance of pumping. Sea level rise has little effect on saltwater intrusion in flux-controlled groundwater systems and causes much more intrusion in topography-limited systems (Befus et al. 2020; Werner et al. 2012; Michael et al. 2013).

Saltwater intrusion is a water quality concern, threatening the ability to use groundwater as a freshwater resource, although secondary concerns exist for potential ecosystem degradation and for accelerating the corrosion of buried infrastructure (May 2020).

B.1-7.6.3 Compounding Effects with Tides and Storms

Groundwater levels and salinity are also affected over short and long timescales by coastal hydrodynamics and seasonal climatology, such that a static representation or one-time measurement would not be sufficient to understand the present-day groundwater conditions. Tides and their seasonal variability induce head and flow fluctuations in coastal groundwater with water level response magnitude decaying exponentially with distance and linearly in time controlled by tidal amplitude and hydraulic diffusivity. Tides also create a small “upper saline plume” recirculation cell of infiltrated seawater in unconfined groundwater systems over the intertidal zone (Robinson et al. 2007), but this mainly occurs along sloping, natural coastlines.

Storms and storm surges can create a larger and longer lasting groundwater level and salinity responses. With a short-lived but large storm surge, groundwater levels can be elevated more and farther inland than would be expected for a similar size tide (Li et al. 2004), representing a groundwater emergence response. More importantly for groundwater salinity, defense overtopping and flooding of saltwater inland can lead to infiltration of that saltwater to the water table, causing groundwater salinization. This groundwater salinization mechanism could also occur with higher tides, and the combined effect of storm surges and large tides could lead to brackish or saline coastal groundwater at present-day.

B.1-7.7 Future Without Project Conditions

Currently, the mean sea level at Presidio gage is approximately 3.2 feet (NAVD88) as shown in *Sub-Appendix B.1.1 Coastal Extreme Water Levels and High Tide Flooding*. The ground water table near the shore is roughly at or above mean sea level and with SLC, the groundwater is expected to rise at the same or rate. The average shoreline elevation is approximately 10.5-11 feet, with low points in each reach at approximately 9 feet for Reaches 1 and 3, 8.5 feet for Reach 2, and 7 feet for Reach 4. Occurrence of inland groundwater emergence is expected to increase in frequency and extent, within both the tidally influenced zone as well as within the zone of inland hydrologic control. Currently the city deals with high groundwater throughout the city and groundwater infiltration into sewer systems and sewage exfiltration into shallow groundwater have both been observed and modeled in shallow coastal groundwater systems where the groundwater tables occurred above and below the network, respectively (McKenzie et al., 2019; Su et al., 2020). With the increased elevation of the groundwater table, there is potential for increased salinity which may increase deterioration and infiltration rates to buried infrastructure (i.e., pipes, structures, tunnels, etc.) which will require Operation, Maintenance, Repair, Replacement, and Rehabilitation expenditures to manage these impacts.

Finally, the increased elevation of the groundwater table and potential for increased salinity may increase deterioration and infiltration rates to buried infrastructure (i.e., pipes, structures, tunnels, etc.) which will require Operation, Maintenance, Repair, Replacement, and Rehabilitation expenditures to manage these impacts. None of these impacts have been monetized nor qualitatively evaluated for the purpose of alternative evaluation. With sufficient levels of sea level rise, low-lying areas of the city are expected to be retreated from due to the frequent occurrence of tidal flooding, and it is within these areas that groundwater emergence is expected to be a leading indicator of this condition.

B.1-7.8 Impacts and Challenges for Proposed Flood Protection Alternatives to Inform Design Solutions

The built environment introduces additional complexity into groundwater behavior in coastal settings.

Structural measures include varying types of shoreline coastal defense systems that are generally comprised of floodwalls and levees. Each structural system is assumed to have a partial depth subsurface cutoff wall to limit seepage and effectively reduce the tidal influence on the landside groundwater elevation. Additionally, due to seismic hazards present within the study area, the structural alternatives all assume ground improvement (e.g., deep soil mixing, jet grouting, compaction grouting, etc.) will be required to ensure the shoreline subsoils are sufficiently stabilized to meet operational and life-safety seismic performance criteria.

The alternatives introduce additional complexity into groundwater behavior in coastal settings. However, the additional infrastructure from the proposed lines of defense is not currently expected to cause issues to the groundwater levels more than what would be seen in the FWOP condition.

The San Francisco shoreline is already highly modified, with substantial filled areas, seawalls, bulkheads, and wharves, and a variety of other shoreline structures. Limited areas within the study area have naturalized shorelines. Most of the shoreline is currently low-lying, with shoreline elevations and inland areas just a few feet above MHHW. As the shoreline is raised and modified, these alterations will impact the groundwater table and the inland water budget. The alternatives proposed for the San Francisco Waterfront Coastal Study were not analyzed in detail relative to groundwater flows and the inland water budget (Chapter 4, Main Report). Instead, three alternative elements are discussed further in *Appendix B.1.5* along with potential groundwater impacts that could occur:

- Raising the seawall / shoreline infrastructure to keep rising Bay water levels from entering inland developed areas
- Using water control structures (tide gates)
- Creating or enhancing natural or nature-based solutions or hybrid solutions to reduce wave energy and minimize coastal flood risks

Section B.1-8 Baywide Induced Flooding Assessment

B.1-8.1 Purpose

This analysis was conducted to assess potential for induced flooding impacts on nearby Bay communities caused by constructing a coastal flood protection structure along the San Francisco shoreline.

B.1-8.2 Methodology

This assessment relied on peer-reviewed reports completed using the USGS Coastal Storm Modeling System for the Bay.

B.1-8.3 Assessment

Hydrodynamic modeling was considered for the preliminary portion of this feasibility study. However, considering the range of alternatives evaluated, the modeling results would not impact the selection of the TSP. Postponing modeling until a later phase, such as the PED phase, is considered reasonable for the following reasons:

1. A modeling study by Wang et al. (2018) indicates that constructing shoreline structures along San Francisco's shoreline will have minimal effect on Baywide water levels and the flooding potential in other Bay Area counties.
2. The POSF's shoreline is highly urbanized, comprised of seawalls, bulkheads, and armored revetments. The alternatives and TSP will not substantively change the locations or amount of shoreline hardening.
3. The new seawalls and shoreline structures can include design modifications and ecological features that reduce erosion, wave reflection, and wave runup and overtopping while also providing habitat benefits for native species.

As sea levels rise in the Bay, low-lying coastal communities will either experience increasingly frequent coastal and tidal flooding, or they will fortify their shorelines using a range of traditional engineered structures and nature-based solutions to mitigate for rising sea levels. In the Bay, which is an enclosed shallow estuary, the tides interact with the estuary's bathymetry, geometry, and shoreline, creating a mix of progressive and standing waves that result in distinct circulation patterns and tidal amplification in the south Bay and north Bay (Conomos 1979). Changing the shoreline characteristics, such as constructing levees or seawalls in areas that are not currently hardened, could impact Baywide tidal dynamics (Holleman and Stacey 2014). Modeling studies of the Bay have shown that implementing shoreline hardening (e.g., levees, seawalls) along one county's shoreline can result in elevated water levels along another county's shoreline, with the magnitude of change dependent on the location of each county (Holleman and Stacey 2014; Wang et al. 2018).

The POSF and USACE recognize that construction of new flood risk reduction structures along 7.5 miles of San Francisco's shoreline could impact Bay hydrodynamics. However, Wang et al (2018) showed that constructing shoreline protection along San Francisco's shoreline has a negligible impact on Baywide water levels and flooding potential along the shorelines of the other eight Bay Area counties (Marin, Sonoma, Napa, Solano, Contra Costa, Alameda, Santa Clara, and San Mateo counties). In fact, constructing shoreline structures along the entirety of the Bay shoreline could reduce water levels along the Napa, Alameda, Santa Clara, and San Mateo shorelines (Wang et al. 2018). This trend was consistent for all rates of sea level rise evaluated, up to 5 feet (1.5 meters) of sea level rise. San Francisco was the only Bay Area County where construction of shoreline flood protection structures would not increase water levels or flood potential in at least one other county. This is likely due to San Francisco's location in the central Bay and its proximity to the Golden Gate inlet to the Pacific Ocean, as well existing hardened nature of San Francisco's urbanized shoreline.

The construction of new seawalls are generally subjected to the greatest public and regulatory scrutiny (Griggs and Patsch 2019; Griggs and Reguero 2021; Gittman, Scyphers, et al. 2016), as seawalls can increase erosion in front of the seawall (Lerma et al. 2022; Nunn et al. 2021) and in nearby areas due to wave reflection (Negm and Nassar 2016). The majority of the POSF's shoreline is already heavily urbanized. The alternatives evaluated in this feasibility study include the placement of new seawalls Bayward of the existing aging seawall (*Appendix A: Plan Formulation*). New seawalls and hardened shorelines are not planned in areas where existing natural shorelines and wetlands are found, such as Warm Water Cove, Pier 94 wetlands, and Heron's Head Park. Therefore, the replacement of the aging hard structures with new hard structures in similar locations that meet existing seismic standards and future sea level rise are unlikely to have a substantive impact on Baywide hydrodynamics.

The potential for hydrodynamic impact can be further minimized using modern seawall designs. Seawall design options include serrated surface textures or surface curvatures intended to minimize wave reflection and erosion (Neelamani and Sandhya 2003; 2005). Seawalls can also include surface textures and specialized concrete mixtures that attract native biota such as bivalves, mollusks, algae, and other species (Xu et al. 2021; Salauddin et al. 2021; Strain et al. 2018; Gittman, Peterson, et al. 2016). The addition of surface textures and ecological habitat may also reduce wave runup and overtopping when compared with more traditional smooth vertical seawall surfaces (O'Sullivan et al. 2020; Vozzo et al. 2021). The POSF's living seawall pilot study is exploring textured wall surfaces and concrete mixtures that can maximize native habitat complexity while also providing flood risk reduction benefits (*Appendix I: Engineering with Nature*). However, these are design details that are beyond the scope of the feasibility phase. These design details will be incorporated during PED.

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Appendix B.1.1

Coastal Extreme Water Levels and High Tide Flooding

Appendix B.1.2

Inundation Maps (Future Without Project and Future With Project)

Appendix B.1.2.1

Future Without Project and Future With Project Maps

Appendix B.1.2.2

Future Without Project and Future With Project 2040 Move Only Maps

Appendix B.1.2.3

Total Net Benefits Plan Maps

Appendix B.1.2.4

Total Net Benefits Plan Decadal Maps

Appendix B.1.3

Wave Overtopping Sensitivity Assessment

Appendix B.1.4

Hydrology and Hydraulics Interior Drainage Analysis

Appendix B.1.5

Shallow Groundwater