SAN FRANCISCO WATERFRONT COASTAL FLOOD STUDY, CA

APPENDIX B.1.4 – HYDROLOGY AND HYDRAULICS INTERIOR DRAINAGE ANALYSIS [DRAFT]

JANUARY 2024

USACE TULSA DISTRICT | THE PORT OF SAN FRANCISCO

Table of Contents

List of Tables

List of Figures

Acronyms and Abbreviations

Acronym	Definition
2D	two-dimensional
AEP	Annual Exceedance Probability
ARI	Average Recurrence Interval
Bay	San Francisco Bay
CCSF	City and County of San Francisco
cfs	Cubic Foot (Feet) Per Second
CSD	Combined Sewer Discharge
CSO	Combined Sewer Overflow
DEM	Digital Elevation Model
EM	Engineering Manual
FEMA	Federal Emergency Management Agency
FWP	Future With Project
FWOP	Future Without Project
HEC-RAS	Hydrologic Engineering Center-River Analysis System
LiDAR	Light Detection and Ranging
MGD	Million Gallons Per Day
MHHW	Mean Higher High Water
NAVD88	North American Vertical Datum of 1988
NOAA	National Oceanic and Atmospheric Administration
PED	Preconstruction Engineering and Design
SFPUC	San Francisco Public Utility Commission
SFPW	San Francisco Public Works

Acronym	Definition
SLC	Sea Level Change
SSMP	Sewer System Master Plan
TNBP	Total Net Benefits Plan
TSP	Tentatively Selected Plan

Section B.1.4-1. Introduction

The San Francisco Waterfront Coastal Flood Study (SFWCFS) is investigating the impacts of climate change to City of San Francisco Bayside area. The details of the study area can be found in the main report.

In accordance with U.S. Army Corps of Engineers (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 city does not see substantial flooding beyond what it would with the current local storm drainage system without a project in place. The San Francisco Public Utility Commission (SFPUC) and San Francisco Public Works (SFPW) departments current levels of service are 20% annual exceedance probability (AEP), 3-hour duration storm in regard to the collection system and 1% AEP, 3-hour duration storm for overland street conveyance. 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 Hydrologic Engineering Center (HEC) River Analysis System (RAS) two-dimensional (2D) modeling software. This software is used to evaluate the future without project (FWOP) conditions and various project alternatives and the impacts of those alternatives on the interior drainage of the San Francisco Bayside area. In addition, the SFPUC and SFPW completed a high-level analysis of the future with project (FWP) and FWOP conditions using their City and County of San Francisco (CCSF) Sewer System Master Plan (SSMP) 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.

The feasibility-level rainfall-tide correlation analysis, completed in accordance with EM 1110-2-1413, is used to estimate the interior and exterior boundary conditions for the HEC-RAS model. *Appendix J: Climate Change* provides additional detail on the future impacts of precipitation and sea level change (SLC) have the potential to impact interior drainage.

Section B.1.4-2. 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.1.4-1.

Figure B.1.4-1: San Francisco Watershed and Urban Watersheds.

B.1.4-2.1 Bayside Drainage Areas

The Bayside drainage area is the only portion that would impact the study area. Of the five urban watersheds, three of them have surface run off that would directly impact the study area. The division of the Bayside drainage area and the SFWCFS area are shown on Figure B.1.4-2.

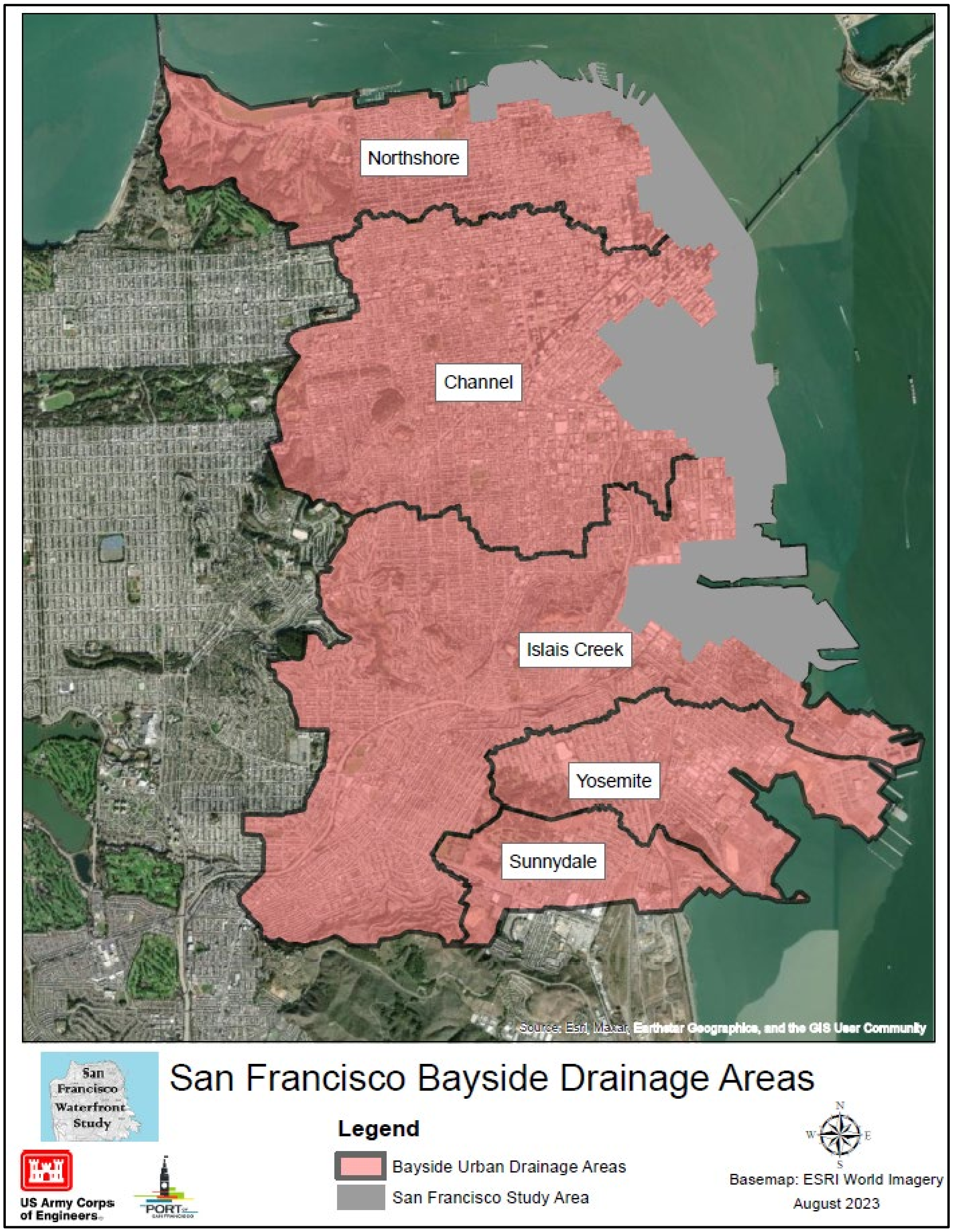

Figure B.1.4-2: Bayside Drainage Area Urban Watersheds with Study Area Extents

B.1.4-2.2 Data Sources

Precipitation data and tidal elevations are used to complete the rainfall-tide correlation analysis as well as the interior drainage analysis. The data is derived from multiple sources which are described in the following paragraphs.

Precipitation data is available from the 1908 through 2018 water years. Hourly data is gathered from the Downtown San Francisco National Oceanic and Atmospheric Administration (NOAA) gage (COOP:047772; 1908-2011) and the San Francisco SFPUC Mission Street gage (ID: RG31; 2011 – 2018) to get a 110-year period of record for the downtown area. The Mission Street gage is a combination of NOAA data through December 2011 and SFPUC gage data from January 2011 through 2018. The SFPUC records data in tips and millimeters which was converted to inches using the SFPUC methods.

In the evaluation of the FWP alternatives and FWOP condition, NOAA Atlas Precipitation Frequency estimates are used in to develop the rainfall applied to the model. The NOAA Atlas 14 values used for this analysis are taken from downtown San Francisco from the NOAA's online resource and shown in Table B.1.4-1 and Table B.1.4-2.

Table B.1.4-1: Best Estimate NOAA Atlas 14 Values, Downtown San Francisco

NOAA Atlas 14 Precipitation Frequency Estimates - Best Estimate (Inches)						
Duration (minutes)	**AEP %**					
	50	**20**	**10**	**4**	**2**	**1**
5	0.16	0.22	0.26	0.31	0.35	0.40
10	0.23	0.31	0.37	0.45	0.51	0.57
15	0.27	0.38	0.45	0.54	0.61	0.69
30	0.38	0.52	0.62	0.74	0.84	0.95
60	0.53	0.73	0.87	1.05	1.19	1.33
120	0.75	1.02	1.20	1.45	1.64	1.84
180	0.93	1.27	1.49	1.80	2.03	2.28
360	1.27	1.74	2.05	2.47	2.81	3.17
720	1.68	2.33	2.78	3.39	3.89	4.41
1440 (24 hours)	2.17	3.06	3.68	4.53	5.22	5.95

Table B.1.4-2: Upper Bounds NOAA Atlas 14 Values, Downtown San Francisco

NOAA Atlas 14 Precipitation Frequency Estimates - Upper Bounds (Inches)						
Duration (minutes)	AEP %					
	50	20	10	4	2	1
5	0.18	0.25	0.30	0.37	0.44	0.50
10	0.26	0.36	0.43	0.54	0.62	0.72
15	0.31	0.43	0.52	0.65	0.76	0.87
30	0.43	0.59	0.71	0.89	1.04	1.20
60	0.60	0.83	1.00	1.26	1.46	1.69
120	0.85	1.16	1.38	1.73	2.01	2.33
180	1.06	1.44	1.71	2.15	2.5	2.89
360	1.44	1.97	2.35	2.96	3.46	4.01
720	1.9	2.65	3.19	4.06	4.78	5.59
1440 (24 hours)	2.46	3.47	4.21	5.35	6.28	7.33

Hourly tidal data is gathered from the San Francisco Presidio tidal gage 9414290 for the 1901 through 2018 water years resulting in a 118-year tidal period of record. In addition to historic tidal data tidal frequency data was also used for FWP and FWOP analysis. Due to the varying tidal elevations along the San Francisco Bay, see *Sub-Appendix B.1.1* for more detail, the mean higher high water (MHHW) values were taken from the San Francisco Bay Tidal Datums and Extreme Study for the exterior bay conditions. A point taken near the central point in each reach is used to estimate the conditions for each of the areas. Table B.1.4-3 shows the tidal elevations used for each of the reaches in the study.

Table B.1.4-3: Tidal Estimates for MHHW based on FEMA Analysis for the Reaches

Federal Emergency Management Agency (FEMA) Analysis	
Location	MHHW (NAVD88)
Presidio	5.9
Reach 1 (North Shore)	6.1

Federal Emergency Management Agency (FEMA) Analysis	
Location	**MHHW (NAVD88)**
Reach 2 (North Shore and Channel)	6.2
Reach 3 (Channel)	6.3
Reach 4 (Islais Creek)	6.5

NAVD88 = North American Vertical Datum of 1988

Section B.1.4-3. Rainfall-Tide Correlation Analysis

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. This section analyzes the available gage data to determine the most appropriate exterior (bay level) condition to apply to a large rainfall event for a feasibility-level study. The 1% AEP tide and monthly tide are only approximately 2 feet different as shown in *Sub-Appendix B.1.1*.

Overland flow is the main driver for inland drainage which occurs during large or high intensity events. When determining interior and exterior boundary conditions, an analysis considering large rainfall events such as the 1% AEP rainfall and assigning an appropriate bay condition based on observed data is considered an acceptable approach at this stage. In Preconstruction Engineering and Design (PED) a more detailed analysis should be completed for the rainfall and tide correlation.

B.1.4-3.1 Methodology

For the FWP and FWOP conditions, understanding the San Francisco 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.

B.1.4-3.2 Correlation

For this study, the correlation was evaluated in two ways, rainfall and tides that occur together at 3, 6 and 24-hour durations and tides that occur for moderate to high rainfall events. In the San Francisco Bay region, the tides can be influenced by several factors, one being storms. This variability can be seen on Figure B.1.4-3 through Figure B.1.4-5 where high peak daily tides can occur on days with no rainfall and on other days where there are higher daily rainfall tidal elevations are at or near MHHW. The longer duration 24-hour storms and peak daily tides shows a tendency for higher tides between mean high water and the 1-year (99% AEP) tide with the moderate to large rainfall events. When looking at the shorter 3-hour duration and the accompanying peak tide from that 3-hour interval there is more variability in the tide for the moderate to large rainfall events. It should be noted that when review the peak tide data the correlation was evaluated based on set time intervals and not by maximum 3, 6, and 24-hour values as well at total tide elevations. Additional review of maximum 24-hour rainfall was based on the 24-hour period on which the rainfall occurred and if the event fell over two days the peak tide for that event was considered the peak tide on the second day. Further evaluation of storm surge to maximum rainfall values could be evaluated during PED.

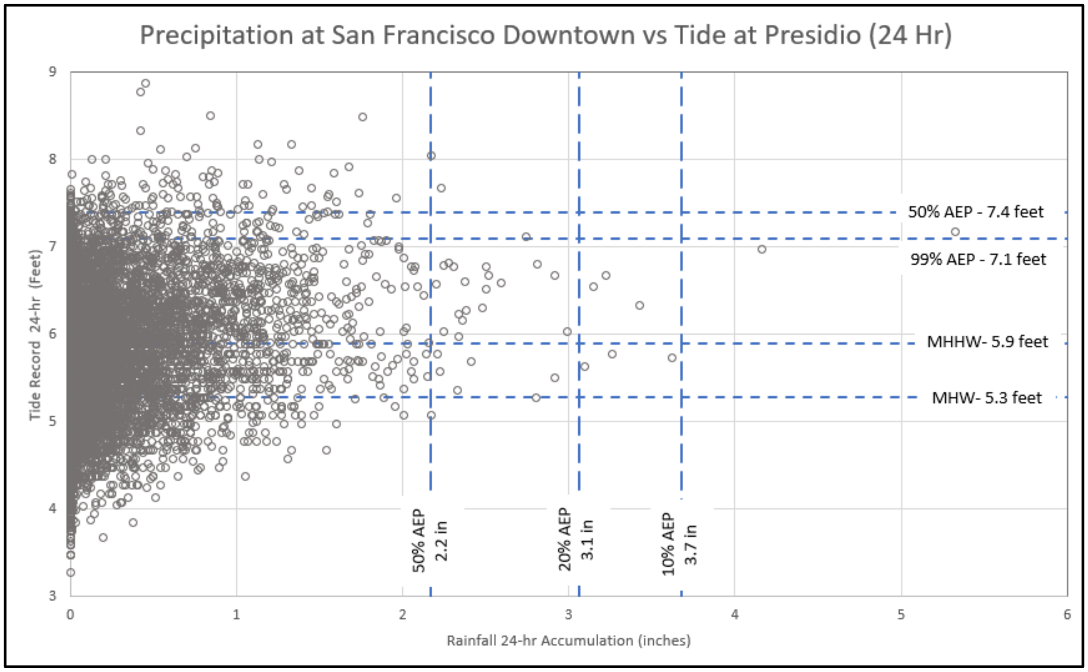

Figure B.1.4-3: Precipitation at San Francisco Downtown vs Tide at Presidio (24 hour)

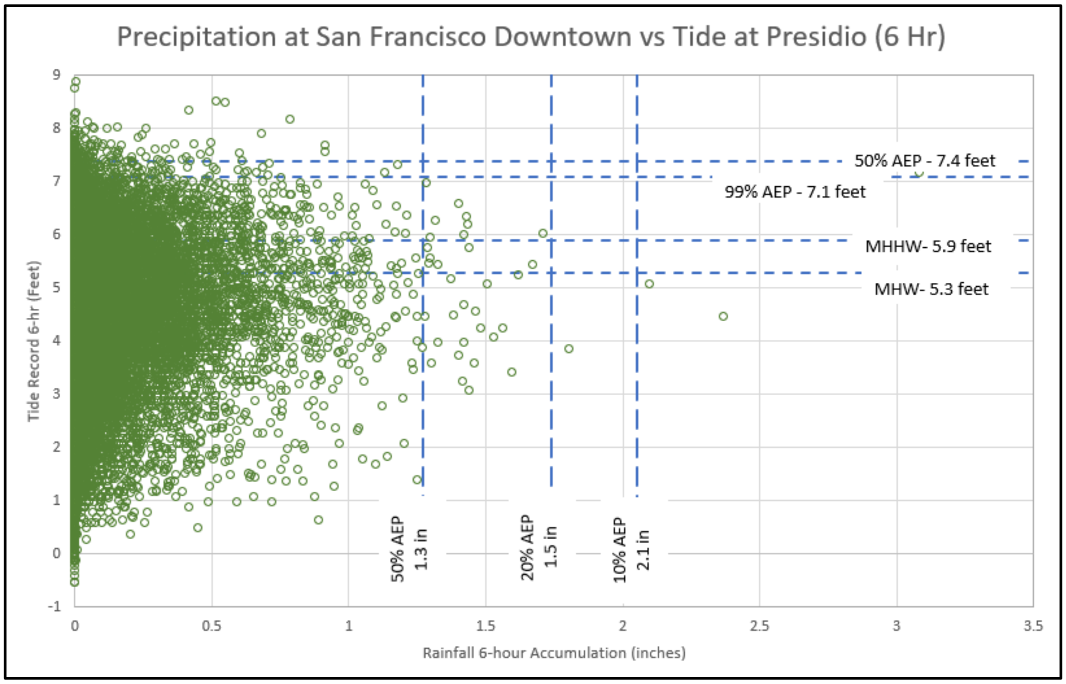

Figure B.1.4-4: Precipitation at San Francisco Downtown vs Tide at Presidio (6 hour)

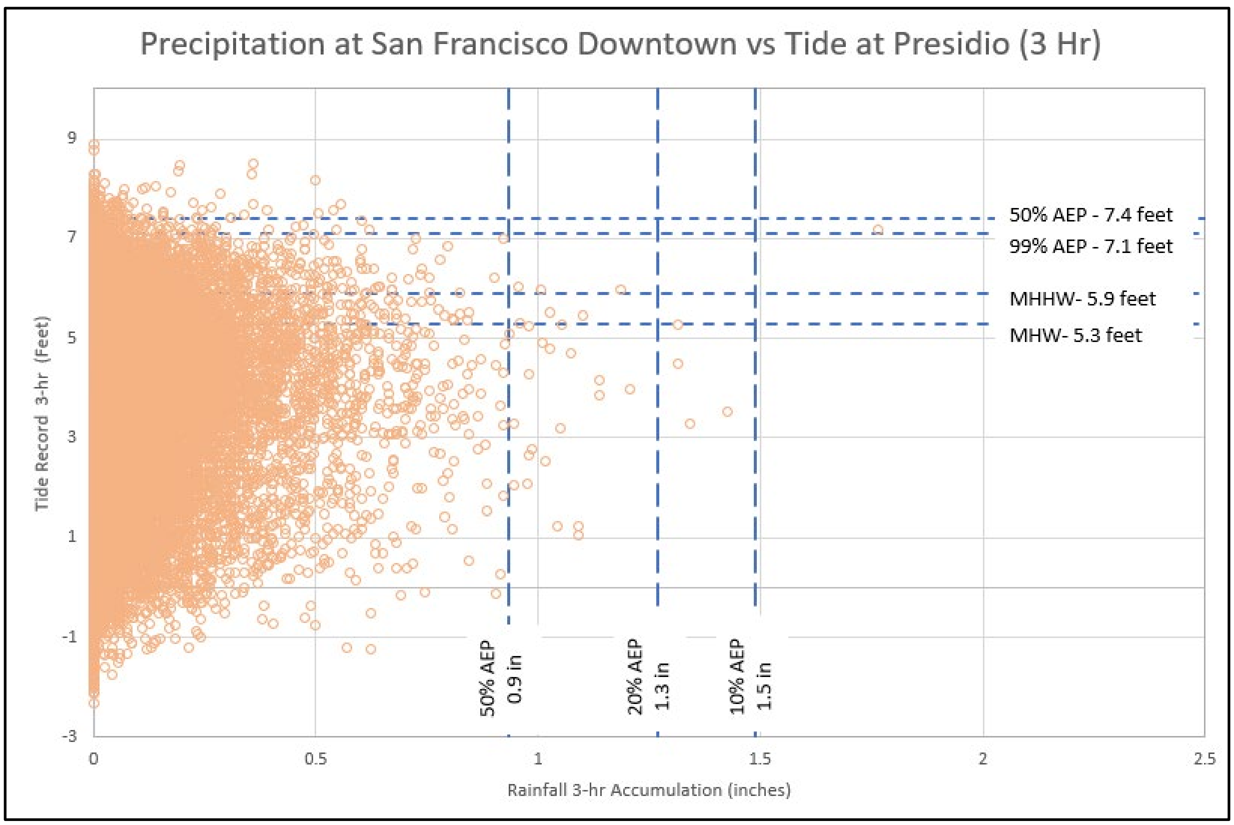

Figure B.1.4-5: Precipitation at San Francisco Downtown vs Tide at Presidio (3 hour)

In addition to a precipitation to tide comparison the minimum, maximum, medians, averages, percent above MHHW and Pearson Correlation Coefficients have also been reviewed. Table B.1.4-4 and Table B.1.4-5 provide information on Pearson Correlation Coefficients, minimum and maximum values for the peak annual tides and accompanying daily 24-hour. The Pearson Correlation Coefficient evaluation is reviewing the correlation between the peak annual tide and rainfall to determine a consistency in rainfall to tide, such as large rainfall always accompanies a certain tide elevation within the range of data available. Correlation values range from 0 to 1, with 0 being no significance and 1 being the highest significance. Figure B.1.4-6 provides a visual presentation of the different datasets regarding peak annual tide at the San Francisco gage.

Table B.1.4-4: Correlation of Peak Annual Tide and Accompanying 24-hour

Dataset	Total Events	Pearson Correlation Coefficient Peak Annual Tide vs 24 Hour Rainfall
1908-2017	110	0.17
1998-2017	20	0.23

Table B.1.4-5: Peak Annual Tide with Daily Rainfall Day of, Day Before, and Day After Peak Tide

Date of Peak Tide by Water Year (Oct-Sept)	Peak Tide (NAVD88)	24-hour Rainfall on Day of Peak Tide (inches)	24-hour Rainfall Day Prior to Peak Tide (inches)	24-hour Rainfall Day After Peak Tide (inches)	Total 72-hour Rainfall (inches)
1/11/2017	7.74	1.29	0.14	0.10	1.52
11/24/2015	7.59	0.00	0.00	0.19	0.19
12/3/2014	7.85	1.49	0.00	1.59	3.08
2/28/2014	7.46	0.04	0.75	0.00	0.79
12/12/2012	7.66	0.01	0.00	0.09	0.09
11/24/2011	7.40	0.00	0.00	0.21	0.21
5/18/2011	7.57	0.19	0.22	0.00	0.41
2/27/2010	7.65	0.29	0.00	0.21	0.50
6/24/2009	7.39	0.00	0.00	0.00	0.00
1/4/2008	7.56	0.20	0.00	1.96	2.16

Date of Peak Tide by Water Year (Oct-Sept)	Peak Tide (NAVD88)	24-hour Rainfall on Day of Peak Tide (inches)	24-hour Rainfall Day Prior to Peak Tide (inches)	24-hour Rainfall Day After Peak Tide (inches)	Total 72-hour Rainfall (inches)
10/9/2006	7.09	0.00	0.00	0.00	0.00
12/31/2005	8.04	0.78	0.02	0.00	0.79
1/8/2005	8.18	0.70	0.00	1.13	1.83
12/24/2003	8.12	0.21	0.00	0.54	0.75
11/7/2002	7.39	0.00	0.00	0.83	0.83
12/2/2001	7.39	1.04	0.02	1.53	2.59
1/10/2001	8.13	0.09	0.55	0.76	1.39
7/1/2000	7.29	0.00	0.00	0.00	0.00
11/30/1998	7.25	0.88	0.00	1.21	2.08
2/6/1998	8.49	0.34	0.21	1.76	2.30
Maximum	8.49	1.49	0.75	1.96	3.08
Minimum	7.09	0.00	0.00	0.00	0.00

NAVD88 = North American Vertical Datum of 1988

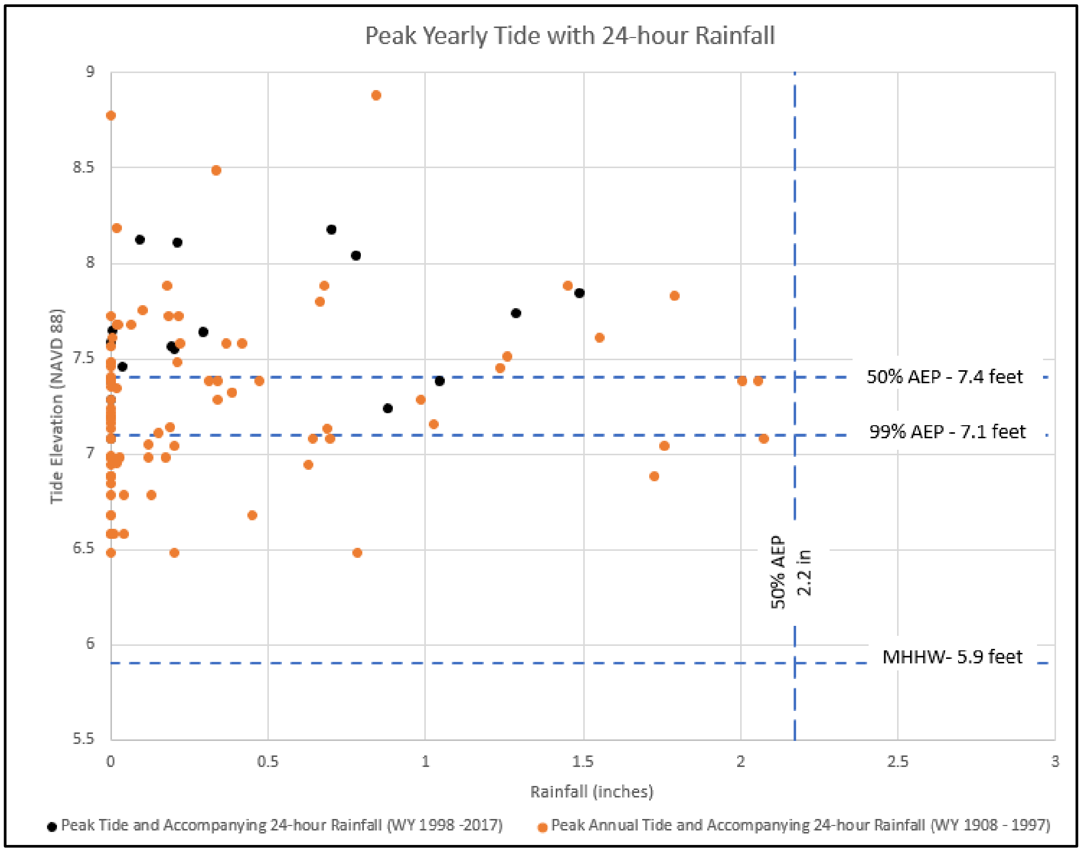

Figure B.1.4-6: Peak Yearly Tide with 24-hour Rainfall

The results show that large 24-hour rainfall events do not tend to occur at the same time as the peak annual tide. This can be seen both with the low Pearson Correlation Coefficient and on Figure B.1.4-6, where most of the 24-hour rainfall occurs at less than the 50% AEP based on NOAA Atlas 14.

Both high tide and high rainfall events are important to understand for this analysis. The larger events would cause overland flow while the lower rainfall events could possibly be handled by the combined storm sewer system in the city. The hourly precipitation data is converted to maximum 24-hour rainfall estimates for the period of record and used to evaluate the higher intensity storms that could potentially overwhelm the combined storm sewer system for an event that may occurred at or near high tides. The annual maximum 24-hour rainfall totals were taken for each of the water years (October to September) and compared to the peak tidal elevations that occurred on the same day. It should be noted that the San Francisco Bay area generally looks at storm years of July through June when evaluating tides. Since the emphasis for this portion of the evaluation is reviewing large rainfall events and the correlating peak tide, the water year was used. Table B.1.4-6 shows the Pearson Correlation Coefficient, median, minimum and maximum peak daily tide, and percent of peak daily tides above MHHW for the annual maximum 24-hour rainfall for various datasets.

Table B.1.4-6: Maximum 24-Hour Precipitation Compared to Peak Tide on that Day

24 Hour Maximum Precipitation						
Dataset	**Total Events**	**% Peak Tide >MHHW**	**Median Peak Tide Elevation**	**Min Peak Tide Elevation**	**Max Peak Tide Elevation**	**Pearson Correlation Coefficient**
1908-2018	110	62%	6.15	5.083	8.01	0.21
Events > 20% AEP	19	68%	6.58	5.283	8.01	0.11
1998-2018	20	85%	6.27	5.359	8.01	0.44

When evaluating the data for the 20-year period from 1998-2017, there is a moderate correlation between the maximum 24-hour rainfall events and accompanying peak daily tidal elevations. When reviewing the peak daily tides for the last 30 years, the median elevation is 6.3 feet, versus the full period of record which is 6.2 feet. Both are slightly higher than MHHW but not at the 99% AEP tide levels. Of the 20 events, 85% of them occurred on a day where the peak daily tide elevation is above MHHW. There are 19 events greater than a 20% AEP in the period of record for 24-hour maximum precipitation. When evaluating those storms there is a low correlation and accompanying peak tidal elevations. The median peak tide elevation for that dataset is higher than both the 20 year and 110 year datasets and 68% of the events occurred on a day that had a peak tidal event above MHHW. Due to the small difference between MHHW and the 50% AEP used for the SFPUC design considerations, sensitivity model runs evaluating both conditions have been completed to determine the impacts for both exterior conditions with the low sea level curve.

B.1.4-3.3 Dependence

In the San Francisco Bay region, there are two primary storm categories that effect the area, extratropical cyclones, and atmospheric rivers. Both types of storms can produce heavy rains and high winds. The extratropical cyclones are large and energetic storms which can generate offshore waves up to 3-4 feet and can be accompanied by heavy rains, however this does not always occur. Atmospheric Rivers are narrow, ribbon-like banks of moisture that originate from the tropics and result in large rainfall events. Both atmospheric rivers and extratropical cyclones have the greatest impact in the central California area in the winter months. One other factor that can impact tide is the occurrence of the King Tide, which also occur in the winter months.

When evaluating the 110-year period from 1908-2018, 85% of the annual peak tides occurred between November and February, which is the time when large rainfall can be expected. In the 20-year period from 1998-2018, only two years had annual peak tides occur on days without rainfall. These two years occurred in the summer months. For the

1998-2018 time frame, the 24-hour rainfall that occurred the day of the peak annual tides are relatively small events.

B.1.4-3.4 Coincidence

Coincidence between the interior and exterior condition looks at the relationship between rainfall on the interior and the peak tidal elevation for the exterior. If a large rainfall event falls during a high tide that could impede the operations of the combined storm and sanitary sewer system and cause additional interior flooding. The higher tides could result in higher groundwater near the shore which could impact the storm sewer system as well as reduce the infiltration of the rainfall. If there is a relationship between the interior, rainfall, and exterior, tide, the worst-case or conservative approach could be considered when modeling hypothetical rainfall. This includes intense rainfall occurring coincident with a tide high enough to reduce the capacity of the combined sewer discharge (CSD) locations causing increased overland flow. Figure B.1.4-7 through Figure B.1.4-13, show the rainfall and tide coincidence plots for seven events. The events are either high tide, large rainfall, or a combination of high tide and large rainfall events. The rainfall dominated events evaluated are, December 2022/January 2023, December 2014, November 1994, and November 1991. The peak tide dominated events are, December 2014, January 2005, January 2001, and February 1998.

In the case of the high rainfall events, the rainfall occurred at or near the high tide or higher high tide for the day. In the case of the high tide events, the rainfall, although much lower in magnitude, also occurred at or near high tide or higher high tide, except for January and November 2001, which are closer to the low tide for the day.

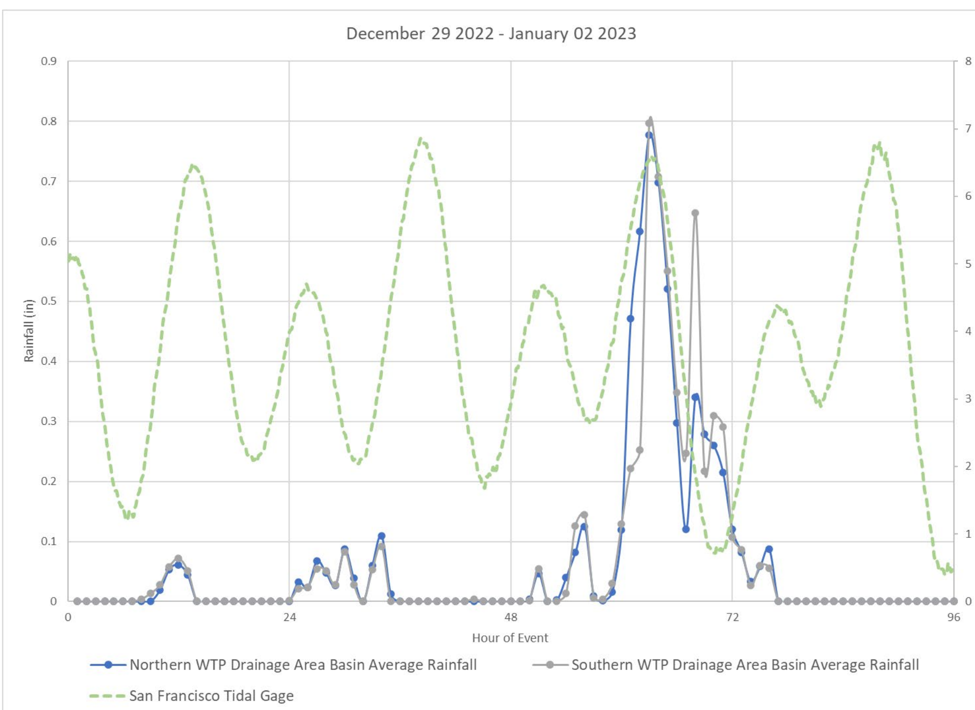

Figure B.1.4-7: December 29, 2022 – January 02, 2023

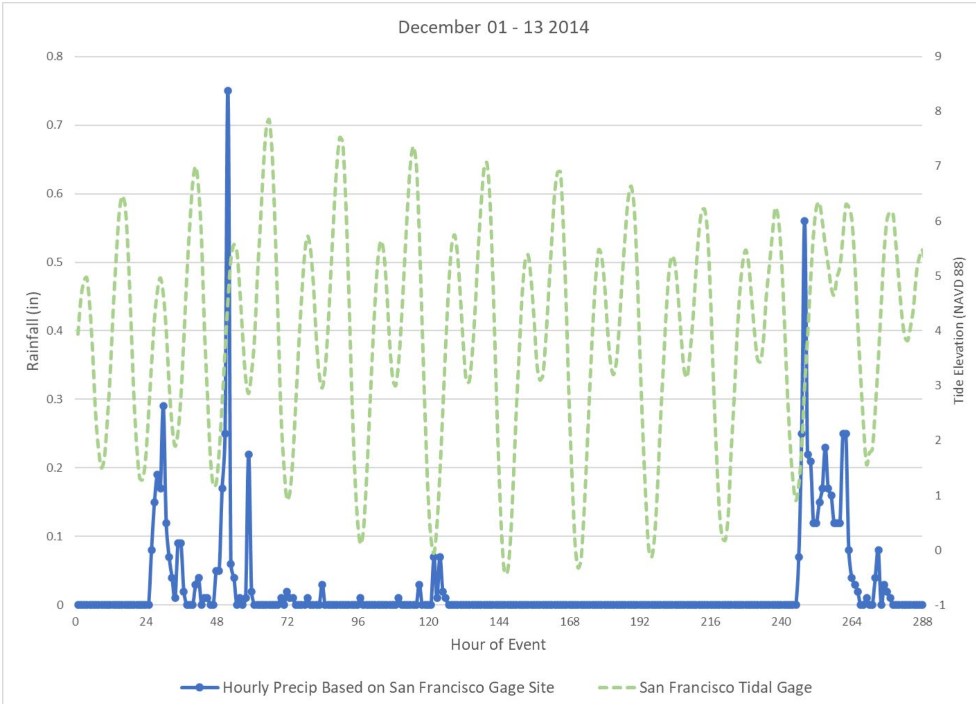

Figure B.1.4-8: December 01 – 13, 2014

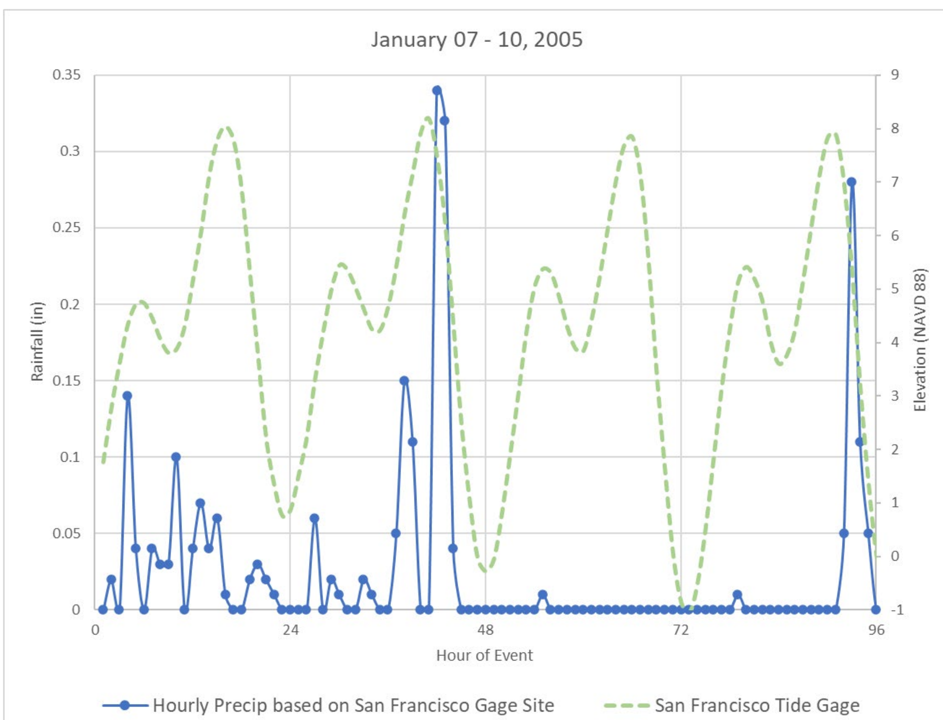

Figure B.1.4-9: January 07 – 10, 2005

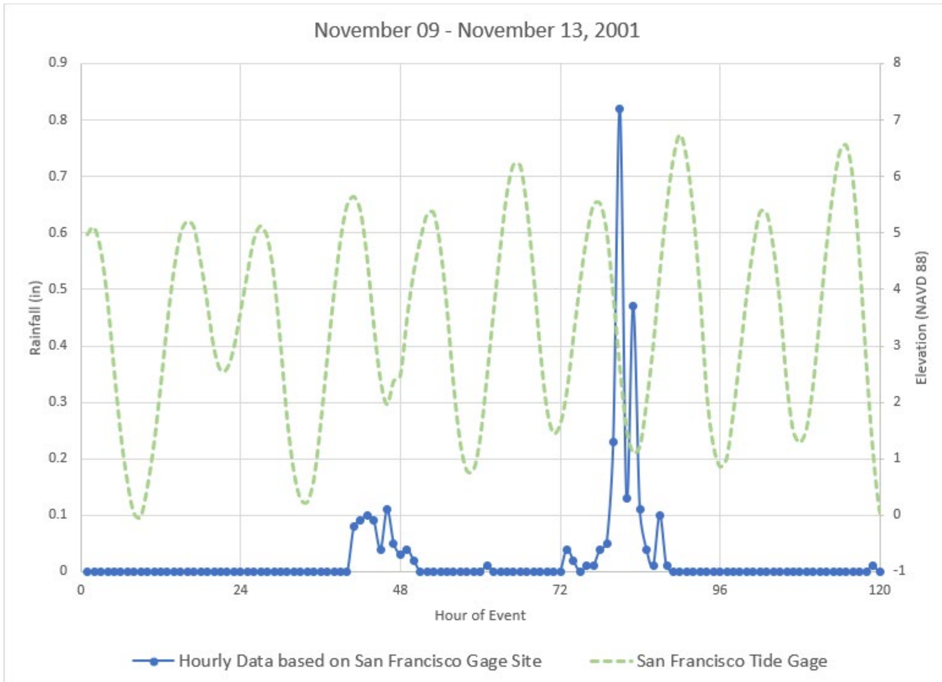

Figure B.1.4-10: November 09 – 13, 2001

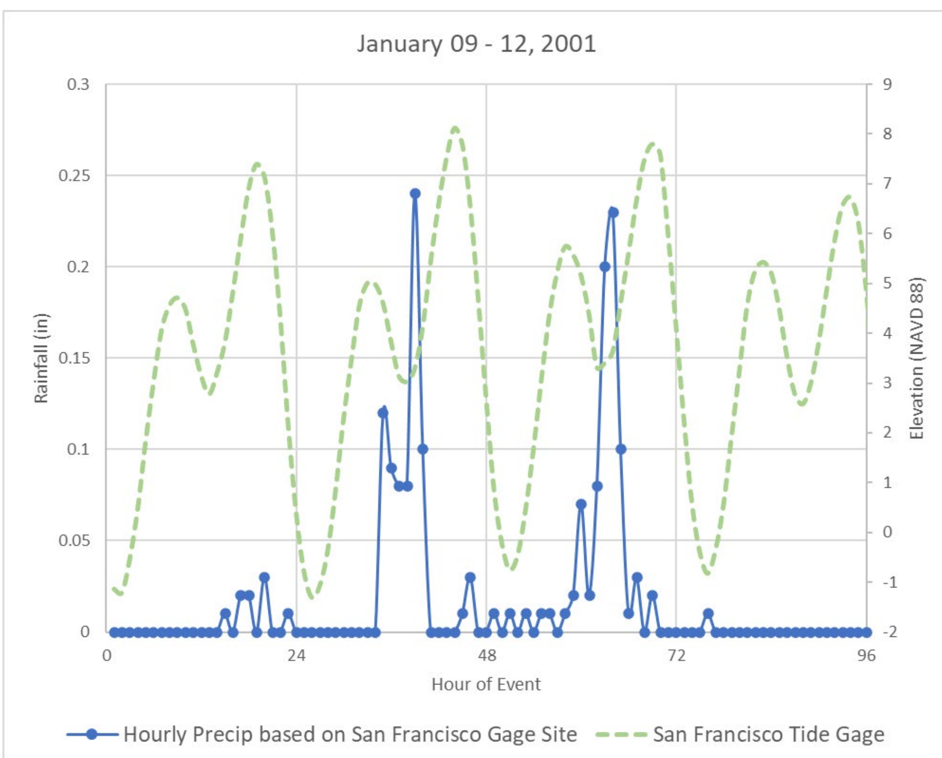

Figure B.1.4-11: January 09 -12, 2001

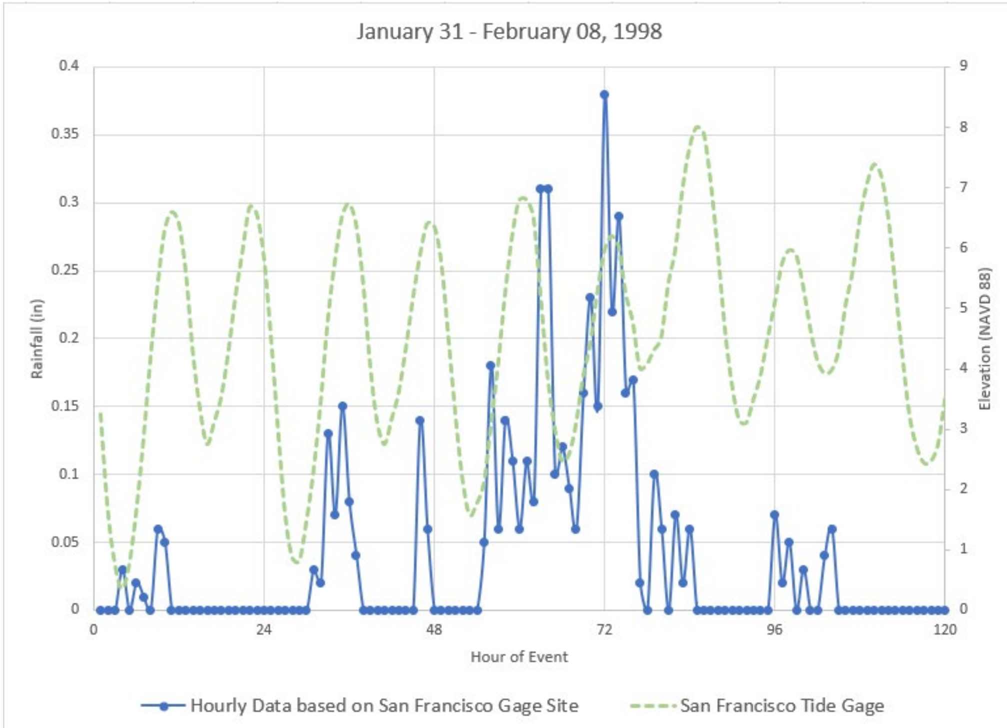

Figure B.1.4-12: February 01 – 09, 1998

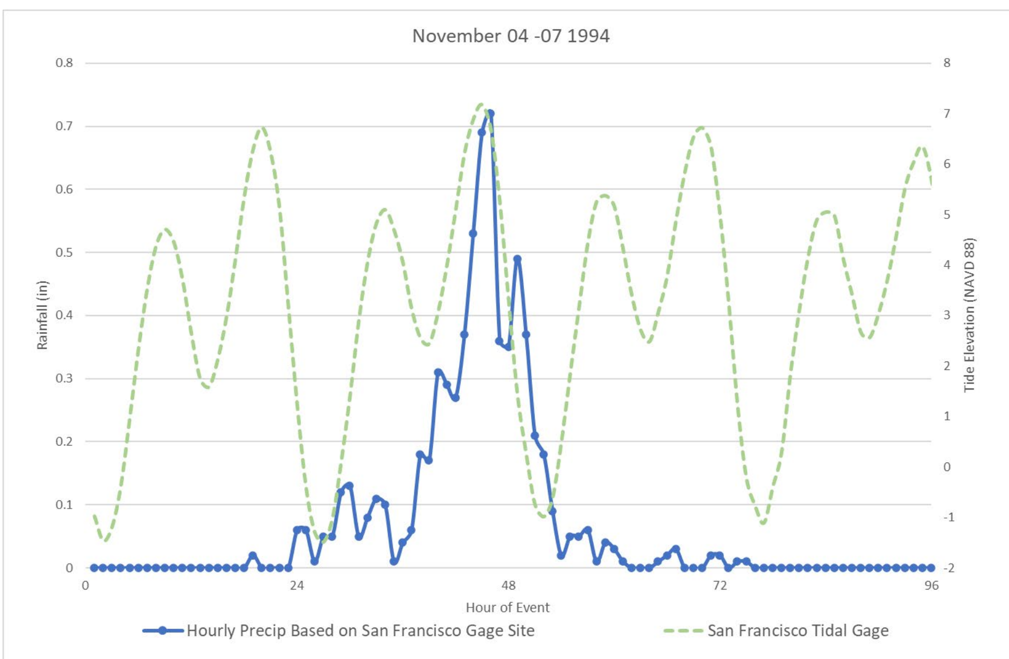

Figure B.1.4-13: November 04 – 07, 1994

B.1.4-3.5 Rainfall Tidal Summary

The peak annual tides, 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, while at other times 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 1998-2018 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.

For this study, the FWP and FWOP conditions will assume a constant MHHW tide elevation from the San Francisco Bay Tidal Datums and Extreme Study for the four primary model areas with SLC, while applying various theoretical storms. Historic events used for testing the HEC-RAS model apply the observed elevations from the Presidio gage. The review of the data shows some coincident time of high tide and rainfall, so, it is assumed that the SFPUC would he operating for wet weather conditions and the two Bayside water treatment plants would be continuously operating.

Section B.1.4-4. San Francisco Storm Management System

The CCSF controls storm water runoff largely (90% of the entire system) through a combined storm and sanitary sewer system that utilizes three water treatment plants throughout the city to treat the runoff. The system consists 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 is to meet the level of service requirements of having freeboard with the collection system of a 20% AEP 3-hour storm and 1% AEP 3-hour storm overland flow for street conveyance.

The two main drainage basins, Westside and Bayside, flow to the water 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 Creek, and Sunnydale. Yosemite Creek 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. More detail on the Bay side network is shown on Figure B.1.4-14.

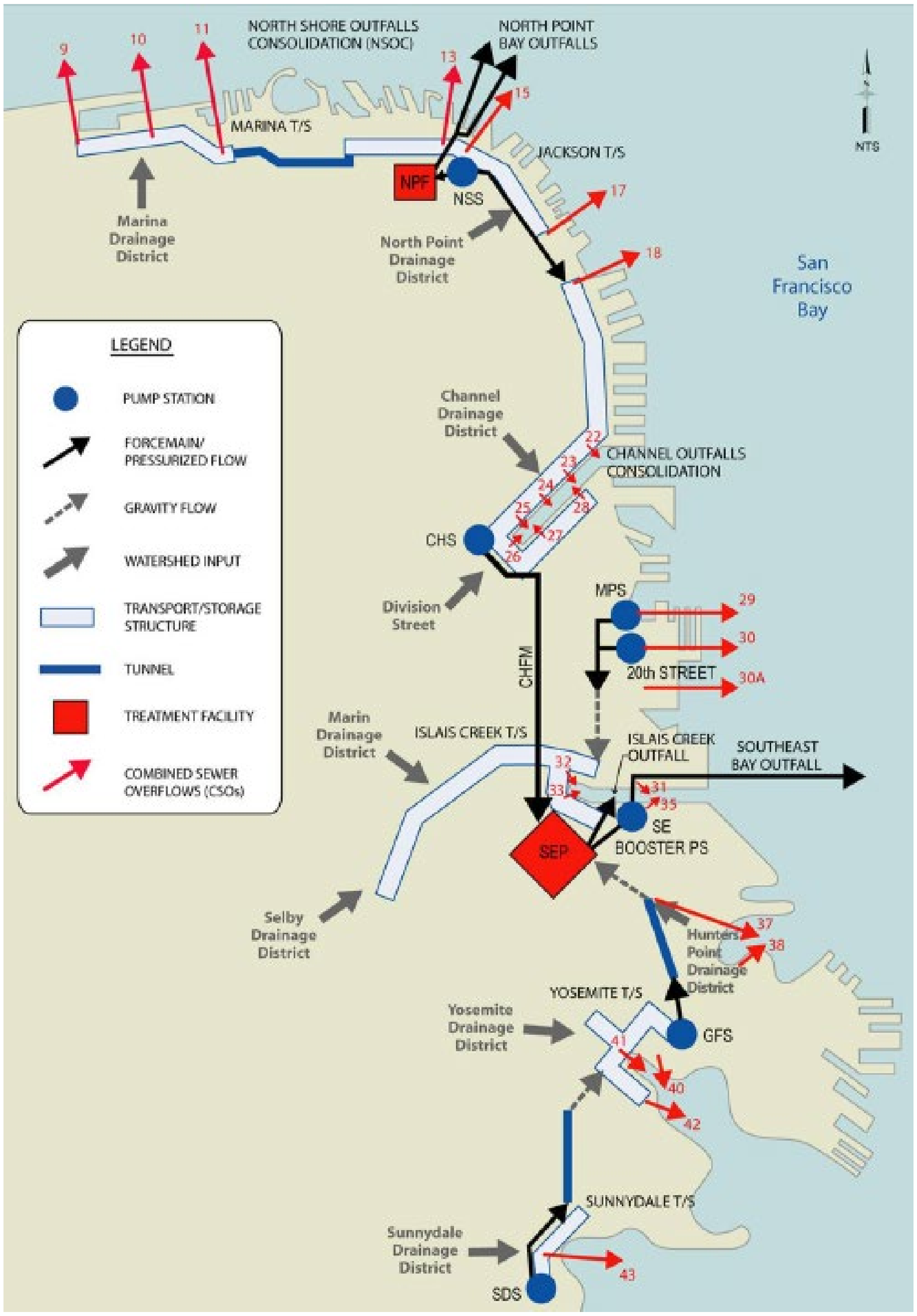

Figure B.1.4-14: Bayside Combined Storm Sewer Schematic

According to the SFPUC maps, approximately 1% of parcels citywide are subject to flooding greater than 6 inches during a 1% AEP 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.1.4-4.1 City and County of San Francisco Sewer System Master Plan InfoWorks ICM Model

The CCSF SSMP ICM 19 model is the planning and operations model utilized by the 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. The hydrologic model calculates the rainfall to runoff transformation through hydrologic parameters assigned to the eight sub-catchments shown on Figure B.1.4-1. The hydraulic module includes conveyance facilities such as sewers, manholes, pumps, weirs, gates, orifices, and outfalls. The rainfall-runoff surface hydrology calculations use the EPA-SWMM5 computation engine that is incorporated into the ICM software. The pipe network, CSD locations, and pump station footprints are shown on Figure B.1.4-15.

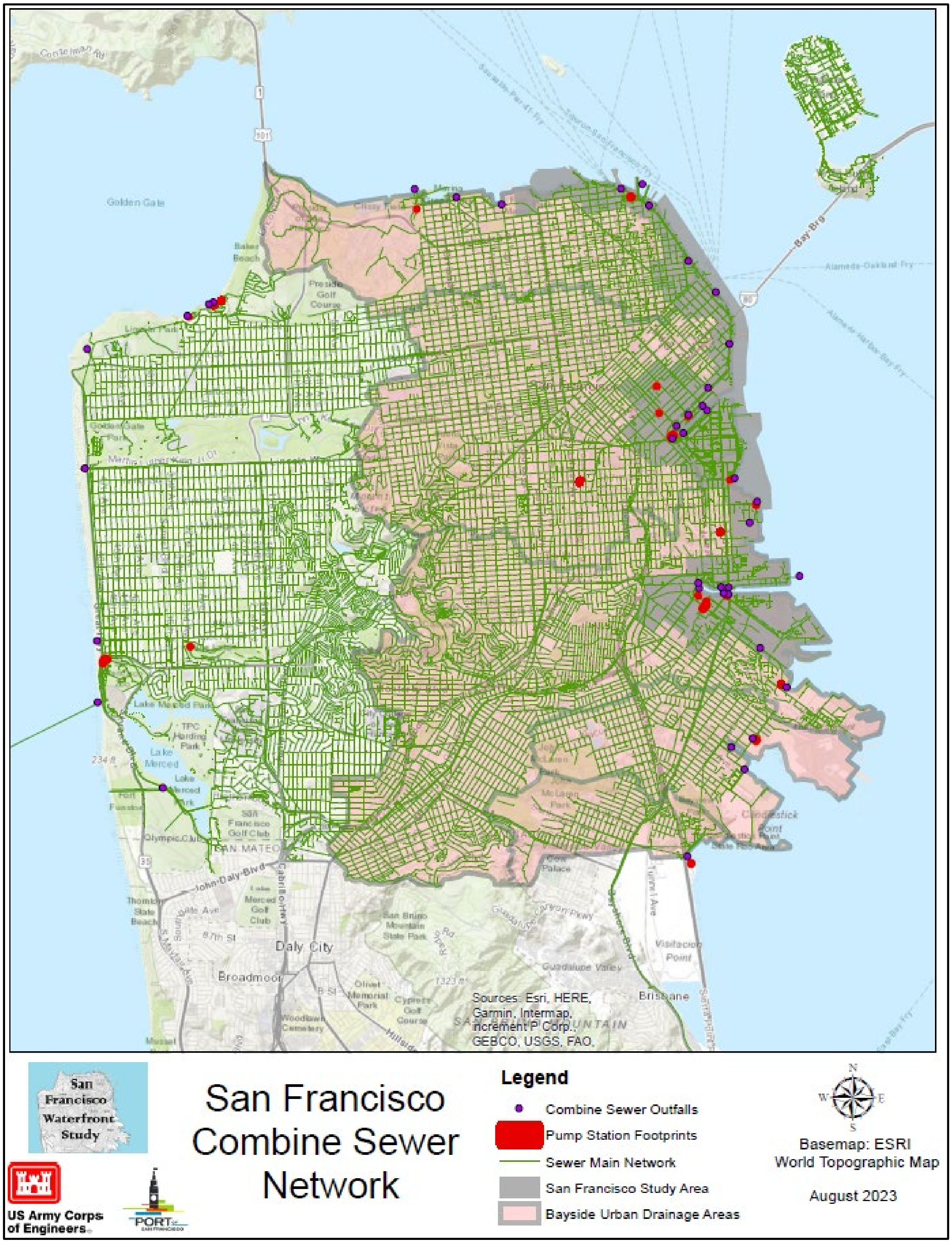

Figure B.1.4-15: Combined Sewer Pipe Network, Pump Station Footprints and Outfalls

B.1.4-4.2 Combine Sewer Outfall Degradation Estimates

To provide a better understanding of the available CSD capacities for the Bayside combined storm sewer system the SFPUC and SFPW provided degradation curves for each of the urban watersheds in the feasibility study area. The reduction in capacity is based on an exponential degradation. A SLC of 0 represents the current planning bay elevations used in the CCSF19 ICM model which ranges from 7.4 in the Northshore to 8 feet in Islais Creek. The curves and reduced capacities are shown on Figure B.1.4-16 through Figure B.1.4-18.

Northshore		
SLR (ft)	5yr-3hr CSD Volume (MG)	100yr-3hr CSD Volume (MG)
0	20	63
0.1	17	52
0.25	14	40
0.5	9	25
1	4	10
1.5	2	4
3.5	0	0
7	0	0

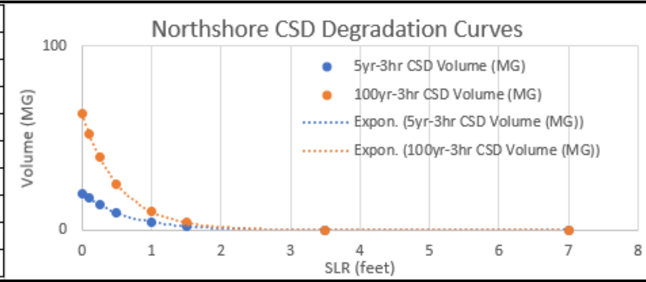

Figure B.1.4-16: Northshore CSD Degradation Curves

Channel		
SLR (ft)	5yr-3hr CSD Volume (MG)	100yr-3hr CSD Volume (MG)
0	96	183
0.1	79	148
0.25	59	107
0.5	36	63
1	13	21
1.5	5	7
3.5	0	0
7	0	0

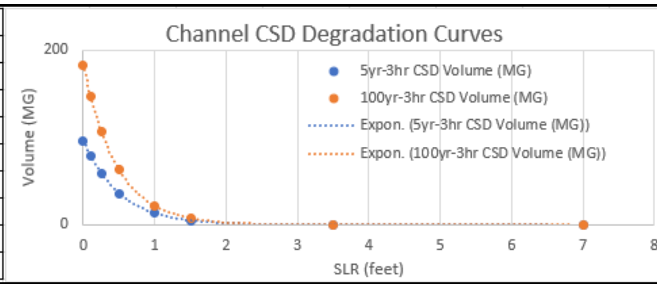

Figure B.1.4-17: Channel CSD Degradation Curves

Islais		
SLR (ft)	5yr-3hr CSD Volume (MG)	100yr-3hr CSD Volume (MG)
0	100	220
0.1	82	177
0.25	61	127
0.5	37	73
1	14	24
1.5	5	8
3.5	0	0
7	0	0

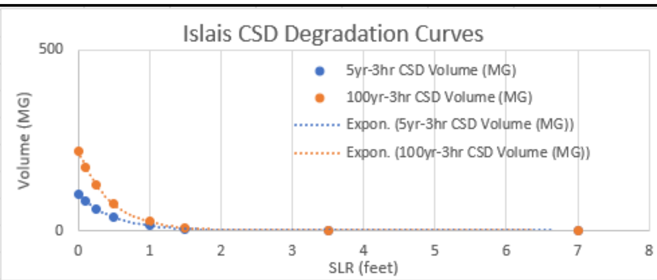

Figure B.1.4-18: Islais Creek CSD Degradation Curves

This data was not used in the assessment but does inform that with the intermediate and high SLC curves there is substantial impact to the available capacity of the combined storm water system.

Section B.1.4-5. HEC-RAS Model Development

B.1.4-5.1 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% impervious parameters were assigned based on the land classification layer provided by the SFPUC.

B.1.4-5.2 Existing Hydraulic Model

No existing HEC-RAS model was available for use in this study. The SFPUC and SFPW have the CCSF19 ICM model is currently used for all planning and design purposes in the area.

B.1.4-5.3 Digital Elevation Model

Light Detection and Ranging (LiDAR) data is available for the entire San Francisco area. The digital elevation model (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.

Terrain modifications are applied to the model. Minor adjustments to the terrain were completed in areas where there were low spots due to the removal of buildings. This was not done throughout the entire watershed but was applied in portions of the study area near Islais and Mission Creeks. Future developments provided by the Port of San Francisco were also incorporated into the terrain for all FWP and FWOP alternatives.

B.1.4-5.4 HEC-RAS Geometry

The HEC-RAS Geometry was built in the NAD 1983 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. Loss rates and impervious areas were defined for the mesh to determine the runoff for the area. Figure B.1.4-19 shows the layout for the FWOP geometry. This was used as the basis for all of the FWP evaluations.

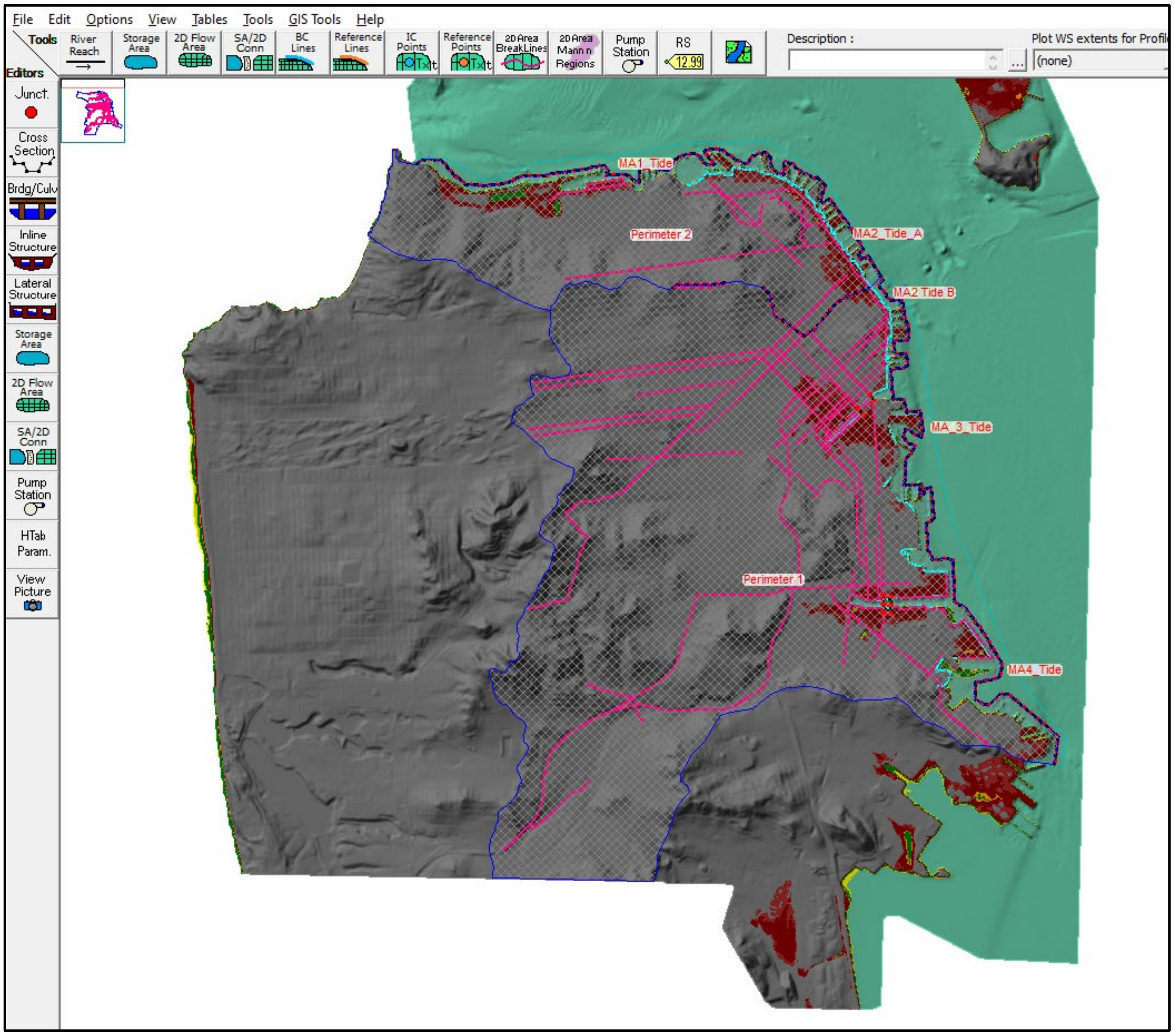

Figure B.1.4-19: Future Without Project Geometry

The geometry is 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 is done so simplified combine storm sewer operations could be incorporated. The 2D areas have cells that range from 40 feet to 160 feet, with the majority of the area at 120 feet. The smaller cell sizes are enforced at the breaklines.

Breaklines are utilized throughout the model for major roadways, high ground, or any potential flow paths in the study area. Breaklines are also used to better define the current sea wall captured in the DEM. The breaklines for the sea wall are only used in the FWOP conditions. The are not included in the FWP conditions because the 2D area connections are used to define the various alternatives and new lines of defense.

2D connections are applied to the study area to assess the various FWP conditions. A weir coefficient of 1 to 2 are used for any 2D connection that represent high ground and weir coefficients of 2 to 3 were used for the bridges and FWP structures.

Four bridges are included in the model. Currently, bridge structures are estimated based on arial imagery and the DEM and need to be updated. Only high and low chord for the bridge decks were included in the initial model runs.

- Mission Creek
  - 3rd Street
  - 4th Street
- Islais Creek
  - 3rd Street
  - Illinois Street

B.1.4-5.4.1 Land Cover

The Land Cover layer used in the CCSF19 ICM model are used for the HEC-RAS model. The layer was visually inspected for the area and deemed appropriate for use in this study. This layer includes all existing conditions, green infrastructure implemented through the Stormwater Management Ordinance from 2010 through 2032. The land cover classifications are shown on Figure B.1.4-20.

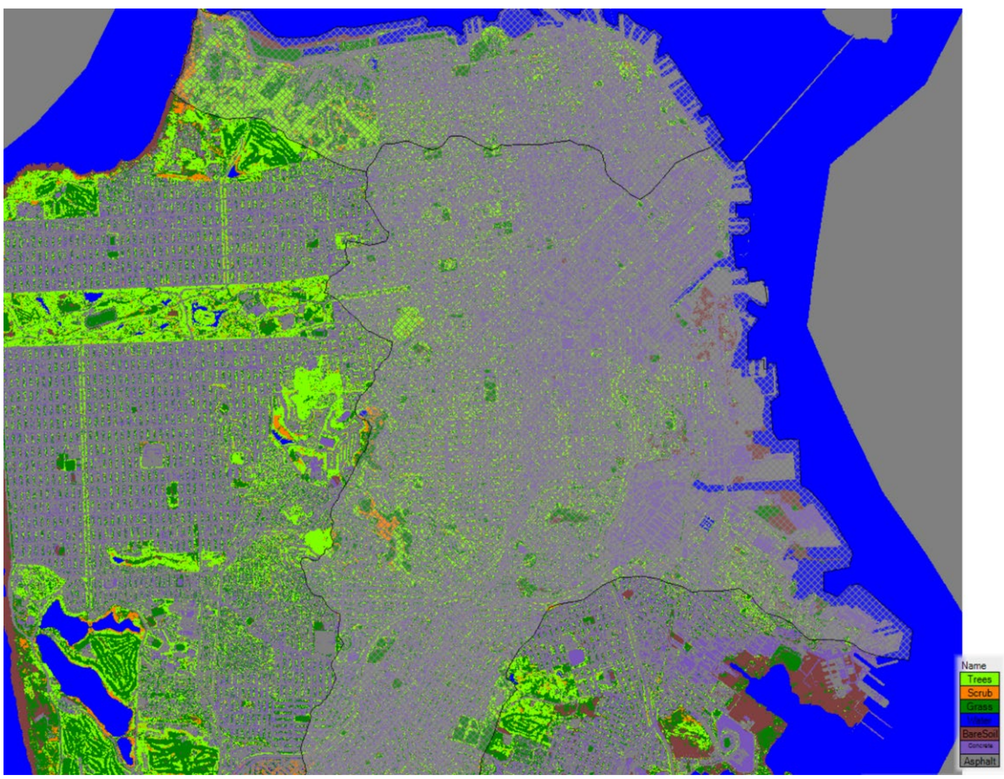

Figure B.1.4-20: Land Cover Layer

B.1.4-5.4.2 Loss Rates

Loss rates are computed in the HEC-RAS 2D model using the deficit and constant method. The infiltration rates are developed based on soil types and in the HEC-RAS 2D User's Manual. The final parameters used in the model are shown in Table B.1.4-7.

Table B.1.4-7: Loss Rates by Soil Type

Map Unit	Soil Type(s)	Maximum Deficit (inches)	Initial Deficit (inches)	Potential Percolation Rate (inches per hour)
121	C	7.2	1.47	0.1
127	D	7.2	1.2	0.025
105	C/D	3.6	1.2	0.05
131	C	1.8	1.47	0.1
130	C	7.2	1.47	0.1
134	C	7.2	1.47	0.1
133	C	7.2	1.47	0.1
129	A	7.2	1.68	0.3
136	A	7.2	1.68	0.3
122	C	7.2	1.47	0.1
106	D	4.8	1.2	0.025
110	C/D	7.2	1.34	0.05
124	D	7.2	1.2	0.025
132	C	7.2	1.47	0.1
125	D	7.2	1.2	0.025
135	C	7.2	1.47	0.1
138	A	7.2	1.68	0.3
111	C	7.2	1.47	0.15

B.1.4-5.4.3 Groundwater

Groundwater in the San Francisco area is expected to rise with the change of sea level for the FWOP condition. See *Sub-Appendix B.1.5* for a detailed discussion on groundwater. For this study, reduced infiltration due to ground water is not considered due to the high impervious area in the city. There are portions of the watershed that have vegetation and riparian areas which are predominately in the upper portion of the watershed which is hilly.

Recharge estimates and dewatering estimates are available in reports completed by the U.S. Geological Society in 1993 and the San Francisco Water Quality Control Board in 1996. Recharge to the Downtown San Francisco groundwater basin was estimated to be 5,900 acre-feet per year in 1993. The 1996 report indicated that 5,600 acre-feet per year is pumped for dewatering though the sewer system. Recharge in the Marina groundwater basin was estimated as 1,341 acre-feet per year in 1993. Recharge in the Islais Valley basin was estimated 1,836 acre-feet per year in 1993.

This study is looking at large high intensity events which are more contusive to runoff with limited infiltration. Groundwater has the potential to reduce the storage available in the combined sewer system. This was not analyzed as part of the assessment. The available storage in the system remained constant throughout the timeline. This should be evaluated in more detail during PED.

B.1.4-5.4.4 Manning's Roughness Values and Impervious Areas

Mannings values and impervious areas are assigned based on the land cover classification. The mannings values are estimated based on the HEC-RAS User's Manual. The values used for the HEC-RAS model are shown in Table B.1.4-8.

Table B.1.4-8: Mannings Values based on Land Cover Classification

Land Cover Classification		
NoData	0.06	0
Asphalt	0.028	70
Trees	0.092	0
Scrub	0.065	0
Grass	0.055	0
Water	0.025	100
BareSoil	0.042	0
Concrete	0.2	90

B.1.4-5.5 Combined Storm Sewer Estimation

The main purpose of the HEC-RAS model is to determine the overland flows from a range of frequency events, to include tidal and rainfall. One reason overland flooding can occur in the area is when the rainfall intensities or total runoff volumes exceed the capacity available through the combined storm sewer system. This can be caused by large rainfall events occurring at high tides that reduce the capacity of the CSDs or with back-to-back events where the initial event has the collection system at or near capacity and unable to handle additional flows from the second event. HEC-RAS does not have the capabilities of modeling the underground combined storm sewer network. The capacity of the combined storm and sewer system was modeled using artificially losses. This was done by estimating simplified wet weather operations by removing volume from the rainfall totals. For this assessment the drainage area was split into two subareas shown on Figure B.1.4-21, one that feeds the Bayside north water treatment plant and the other that feeds the Bayside south water treatment plant.

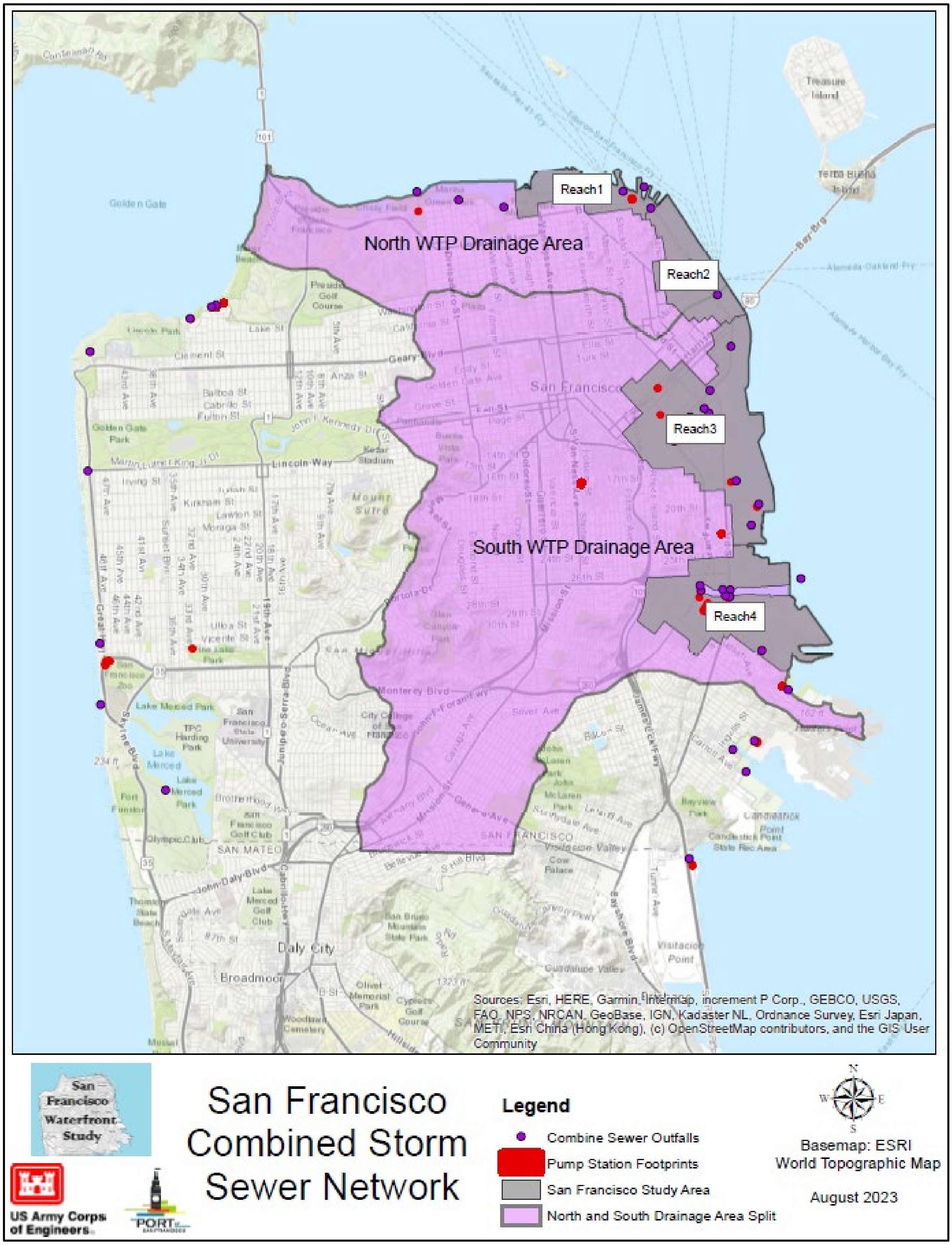

Figure B.1.4-21: Bayside North and South Water Treatment Plant Split

The rainfall is reduced by the amounts shown in Table B.1.4-9 and Table B.1.4-10.

Table B.1.4-9: North Water Treatment Plant Rainfall Reduction

North Water Treatment Plant Drainage Area				
Total Area	2400	Acres	Maximum Rainfall Reduction	
North Water Treatment Plant	150	MGD	0.10	inch(es) per hour
Transport/Storage Capacity	24	MG	0.38	inches

MGD = million gallons per day

Table B.1.4-10: South Water Treatment Plant Rainfall Reduction

South Water Treatment Plant Drainage Area				
Total Area	11500	Acres	Maximum Rainfall Reduction	
South Water Treatment Plant	100	MGD	0.01	inch(es) per hour
Transport/Storage Capacity	83	MG	0.27	inches

MGD = million gallons per day

The artificial losses due to the combined sewer system comprised on the outflows from the water treatment plants and the storage available in the system. The outflows from the water treatment plant were a constant reduction through the entire storm event. The reduction for the storage in the system is considered more of an initial loss at the beginning of the event.

The water treatment plants are considered constantly running and could reduce the flows by the maximum 150 million gallons per day (MGD) for the Northshore portion and 100 MGD for the Channel and Islais Creek portion outputs. The southern portion of the San Francisco watershed includes Yosemite and Sunnydale which are not included in this study. For this reason, the pumping capacity for the southern water treatment plant is reduced from 250 MGD to 100 MGD. All of outflows assumed for the treatment plants were removed from the system since the flows would enter the Bay.

Once the water treatment plant capacity is exceeded the storm network pipes and storage structures began to fill with rainfall. The system service level is a 3-hour 20% AEP rainfall event which is approximately 1.27 inches. Whatever cannot be processed by the water treatment plants is held in storage until the service level capacity is reached. This level of service includes the outflows at the CSD locations which are not captured in this study. When evaluating the rainfall-tide correlation it was noted that moderate to large rainfall 24-hour rainfall events can occur on days with tidal conditions above MHHW. However, with the small duration high intensity events such as the 3-hour durations, the coincident timing with the tide is more variable.

During times when there are higher tides the CSDs along the creeks and bay could be reduced in areas depending on the event. No CSDs were modeled in the geometry due to the limitations of modeling capabilities of underground pipe networks in HEC-RAS. Instead, the available storage in the transport/storage structures and the north and south treatment plant pumping capacities were the estimates used to capture the combined storm water conveyance for the system.

Once the sewer network exceeded the capacity the rest of the rainfall was applied to the HEC-RAS model. Infiltration and impervious areas were included in the HEC-RAS model and applied to the rainfall only once the storm sewer capacity is exceeded.

Section B.1.4-6. HEC-RAS Existing Condition Evaluation

Limited data is 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. 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.

Three historic events were selected to evaluate the interior drainage model. The events are shown in Table B.1.4-11. These events represent high rainfall events occurring in recent history and had observed peak tide water levels above MHHW at the San Francisco (Presidio) Tidal Gage 9414290.

Table B.1.4-11: Modeled Historic Events

Date	Storm Duration (days)	Maximum 24-Hour Rainfall (inches)
Nov. 04-07, 1994	4	6.2
Dec. 11-12, 2014	2	3.5
Dec. 31, 2022 - Jan. 01, 2023	2	5.14*

The loss rates and mannings values were refined based on the initial results of the historic events and discussions with the SFPUC. The maximum inundation the three historic for those events are shown on Figure B.1.4-22.

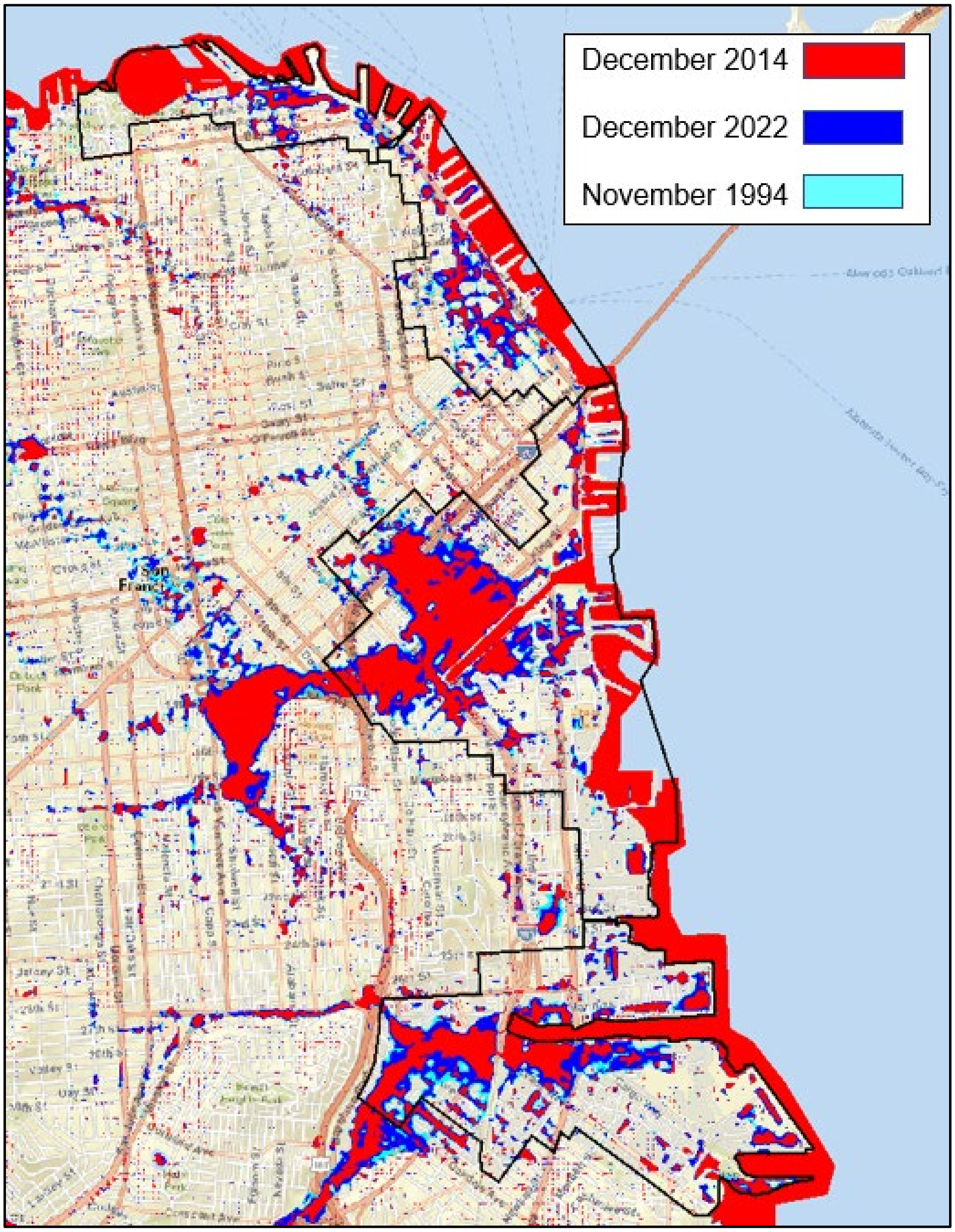

Figure B.1.4-22: Maximum Flood Extents for Dec. 2014, Dec. 2022, and Nov.1994.

The SFPUC provided images of the December 2022 event that were submitted through the 311-information system during the event to aid in calibration of the model. Figure B.1.4-23 shows the maximum inundation extents the December 2022 events along with images that within or near the extents of the study area. The images themselves may or may not capture the maximum water surface, however it does provide context as to areas that at some point during the event was experiencing various levels of flooding.

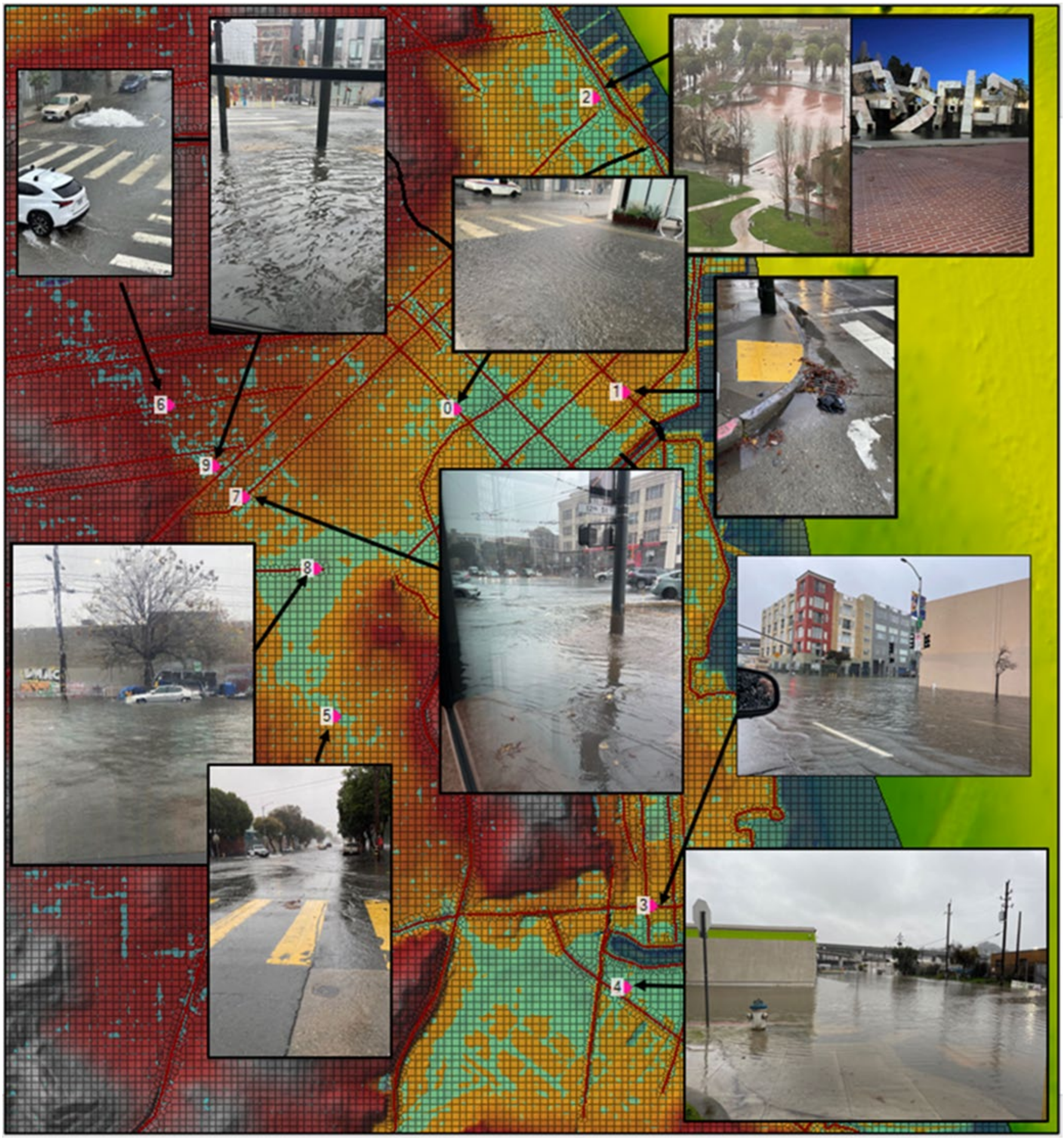

Figure B.1.4-23: Maximum Flood Extents and Accompanying Images at Each Point

The SFPUC provided results from the ICM model for the NOAA Best Estimate 1% AEP 24-hour storm with the 50% AEP storm surge to gain a better understanding on how the HEC-RAS model with artificial losses to account for the storm system compared to the SFPUC ICM model which includes a robust underground combine sewer model. The ICM model and the HEC-RAS model both used the same DEM for the simulation runs, so comparing depths was an acceptable way to evaluate the results. The SFPUC and SFPW provided multiple validation point locations that could be used for comparison. 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, but the overall sizing of the pumps is acceptable for the high-level estimates completed for this portion of the analysis. A more in-depth evaluation of the Tentatively Selected Plan (TSP) will be completed between TSP and final report. During that time the underground network will be evaluated in more detail and refinement of any the pump and culvert sizing will be completed. This work will be performed in close conjunction with the SFPUC and SFPW staff.

B.1.4-6.1 HEC-RAS and CCSF19 Boundary Conditions

The HEC-RAS model is used as the primary model for determining interior drainage estimates for the FWP estimates. The SFPUC and SFPW used their CCSF19 model to determine to provide a high-level analysis in the evaluation of what would be done in the absence of a project regarding the combined storm water system. The SFPUC and SFPW have a level of service that is used for all design criteria in their jurisdiction. The two evaluations use different boundary conditions, however both estimates are completed using 24-hour rainfall.

B.1.4-6.2 Exterior Boundary Conditions

Exterior bay boundary conditions are developed based on period of record data from the Presidio tidal gage. The SFPUC completes all design analysis based on the 50% AEP storm surge from the Extreme Tide analysis completed in 2016. Section B.1.4-3 provides a high-level rainfall-tide correlation analysis, which is the basis for the exterior bay boundary conditions used for the interior drainage analysis for this study.

The bay levels in both models are based on a constant still water elevations. The exterior bay level for the HEC-RAS analysis was applied to the 2D areas in the model, which allowed for overtopping of the sea wall and into the Bay or in the case of SLC from the Bay into the city.

Based on the results from Section B.1.4-3, it was determined that large events such as the 1% AEP rainfall would likely occur near the peak tidal condition near or slightly above MHHW. For the analysis MHHW developed as part of the San Francisco Bay FEMA report was used for the exterior Bay boundary condition in the HEC-RAS model. Table B.1.4-12 summarizes the various elevations for current day and the SLC assumptions used in the alternative analysis.

Table B.1.4-12: MHHW Bay Estimates with Sea Level Change Project Assumptions

Model Area	MHHW (NAVD88)	Low SLC Assumption	Intermediate SLC Assumption	High SLC Assumption
Sea Level Change (feet)		1.50	3.50	7.00
Reach 1 (North Shore)	6.11	7.61	9.61	13.11
Reach 2 (North Shore and Channel)	6.23	7.73	9.73	13.23
Reach 3 (Channel)	6.34	7.84	9.84	13.34
Reach 4 (Islais Creek)	6.45	7.95	9.95	13.45

NAVD88 = North American Vertical Datum of 1988

The ICM model is based on a 50% AEP storm surge exterior boundary condition. The bay levels used in the study area are summarized in Table B.1.4-13.

Table B.1.4-13: 50% AEP Storm Surge Bay Estimates with Sea Level Change Project Assumptions

Model Area	50% AEP Storm Surge (NAVD88)	Low SLC Assumption	Intermediate SLC Assumption	High SLC Assumption
Sea Level Change (feet)		1.50	3.50	7.00
Reach 1 (North Shore)	7.4	8.90	10.90	14.40
Reach 2 (North Shore and Channel)	7.9	9.40	11.40	14.90
Reach 3 (Channel)	7.9	9.40	11.40	14.90
Reach 4 (Islais Creek)	8	9.50	11.50	15.00

NAVD88 = North American Vertical Datum of 1988

In general, the typical weir crest elevations for the CSD locations are at just over 8 feet North American Vertical Datum of 1988 (NAVD88) and the typical embarcadero grade is near 11 feet (NAVD88).

B.1.4-6.3 Rainfall Assumptions

The national and local future precipitation trends discussed in *Appendix J: Climate Study* show and increase in both total volume and intensity over time. For this study, the future rainfall estimates were based on the NOAA Atlas 14 estimates at the upper bounds of the 90% confidence interval shown in Table B.1.4-14 to account for this projected increase. The SFPUC and SFPW evaluation was completed using the NOAA 1% AEP Best Estimate Rainfall as that is the current design criteria for the system shown in Table B.1.4-15.

Table B.1.4-14: NOAA Atlas 14 Precipitation Frequency Estimates (Upper Bound)

Precipitation Frequency Estimates At Upper Bound Of 90% Confidence Interval									
By Duration for ARI (years):	**2**	**5**	**10**	**25**	**50**	**100**	**200**	**500**	**1000**
5-minutes:	0.18	0.25	0.30	0.37	0.44	0.50	0.58	0.70	0.80
10-minutes:	0.26	0.36	0.43	0.54	0.62	0.72	0.83	1.00	1.14
15-minutes:	0.31	0.43	0.52	0.65	0.76	0.87	1.01	1.21	1.38
30-minutes:	0.43	0.59	0.71	0.89	1.04	1.20	1.38	1.66	1.90
60-minutes:	0.60	0.83	1.00	1.26	1.46	1.69	1.95	2.34	2.68
2-hours:	0.85	1.16	1.38	1.73	2.01	2.33	2.68	3.22	3.69
3-hours:	1.06	1.44	1.71	2.15	2.50	2.89	3.33	4.01	4.60
6-hours:	1.44	1.97	2.35	2.96	3.46	4.01	4.64	5.60	6.45
12-hours:	1.90	2.65	3.19	4.06	4.78	5.59	6.51	7.93	9.20
24-hours:	2.46	3.47	4.21	5.35	6.28	7.33	8.51	10.30	11.90

ARI = Average Recurrence Interval

Table B.1.4-15: NOAA Atlas 14 Precipitation Frequencies Estimates (Best Estimate)

Precipitation Frequency Estimates (Best Estimate)									
By duration for ARI (years):	**2**	**5**	**10**	**25**	**50**	**100**	**200**	**500**	**1000**
5-minutes:	0.16	0.22	0.26	0.31	0.35	0.40	0.44	0.51	0.56
10-minutes:	0.23	0.31	0.37	0.45	0.51	0.57	0.64	0.72	0.80
15-minutes:	0.27	0.38	0.45	0.54	0.61	0.69	0.77	0.88	0.96

Precipitation Frequency Estimates (Best Estimate)									
By duration for ARI (years):	**2**	**5**	**10**	**25**	**50**	**100**	**200**	**500**	**1000**
30-minutes:	0.38	0.52	0.62	0.74	0.84	0.95	1.05	1.20	1.32
60-minutes:	0.53	0.73	0.87	1.05	1.19	1.33	1.49	1.70	1.86
2-hours:	0.75	1.02	1.20	1.45	1.64	1.84	2.04	2.34	2.57
3-hours:	0.93	1.27	1.49	1.80	2.03	2.28	2.54	2.91	3.20
6-hours:	1.27	1.74	2.05	2.47	2.81	3.17	3.54	4.07	4.49
12-hours:	1.68	2.33	2.78	3.39	3.89	4.41	4.97	5.76	6.41
24-hours:	2.17	3.06	3.68	4.53	5.22	5.95	6.74	7.86	8.78

ARI = Average Recurrence Interval

The rainfall distribution for the HEC-RAS model evaluation was developed using a 5 minutes, 1 hour, 3 hour, 6 hour, 12 hour, and 24 hour balanced hyetograph.

Section B.1.4-7. 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 with 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 of from 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 bay condition with start to act as a wall causing the overland flows to backwater into the lower portions of the city.

B.1.4-7.1 SFPUC and SFPW FWOP Analysis

In lieu of a government project SFPUC and SFPW completed a high-level modeling analysis utilizing their CCSF19 H&H ICM model to better understand how the combined sewer system could potentially perform in a future with 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 with 7 feet of sea level rise was evaluated for the conceptual design.

Based on the analysis completed by the SFPUC and SFPW, substantial work to relocate transport storage boxes and provide additional force mains in the Channel system which includes the Embarcadero and Mission Creek areas. Relocation of pumps due to sea level rise and additional pumps to maintain the CCSF's level of service design criteria were also included in the plan. With sea level rise, higher groundwater tables will also occur, requiring floodproofing of transport storage boxes and floodwalls in the study area.

Section B.1.4-8. 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.1.4-8.1 Gravity Flow Structures

Gravity flow structures were evaluated for each alternative in the study. For the low SLC scenarios, the coastal protection structures are designed to 1.5 feet of SLC. Under this condition of SLC, much of the interior drainage could be handled by a gravity flow system that is supplemented with pumps as needed. All culverts used in the HEC-RAS model included no negative flow flap gates.

The SFPUC combined storm drain system will have near 0 capacity for all the CSDs at approximately 1.5 feet of sea level rise for the 50% AEP storm surge based on the degradation curves in Section B.1.4-4.2. The 50% AEP storm surge is approximately 1.5 feet higher than the MHHW along the Bay. When looking at a 1% AEP storm occurring at or near MHHW as evaluated in this study, additional culverts with backflow prevention along the line of protection could pass the overland flows that would currently overflow into the Bay, reducing the additional pumping requirements until higher sea level conditions were seen in the future.

Once 3.5 feet of sea level rise has occurred, the future MHHW condition will provide minimal relief during peak runoff conditions and more pumps will need to be installed to remove the overflow.

B.1.4-8.2 Water Management Structures

Tidal gates are included as part of the coastal protection assessment. These tide gates allow for water to flow out to the Bay during low tide or during higher tides at lower levels of sea level rise. One tide gate structure is placed on Islais Creek and Mission Creek.

During the first action, the tidal gates are open during the simulated rainfall event. This allows the rainfall that falls on the Islais Creek and Mission Creek watersheds to drain to the Bay as it does currently. For the second action, the tide gates are converted to culvert type structure. Once converted, for high sea level rise conditions, runoff would be allowed to fill the available storage in the creeks and pumps would be used to move water to the Bay and prevent flooding.

B.1.4-8.3 Pumps

Pumps are included in all the alternatives with coastal protection. These vary in size and placement depending on the exterior bay condition, level of retreat, and use of water management structures.

Pumps are placed in low spots in the study area that would provide the greatest level of relief from flooding based on the coastal protection structure alignment. The water was pumped at a constant rate out of system for the assessment. Elevations for defined for the pumps turning on and off were based on the terrain file and set just above the ground elevation.

HEC-RAS cannot model complex underground pipe networks, such as the one in San Francisco. For this reason, the pumps were not always near a storage transport structure or a main line in the system where they could be tied into the system. During PED the pump locations should be evaluated and placed in areas that would best tie into the combined storm sewer system.

B.1.4-8.4 Structure Placement and Results by Alternative

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 line of defense 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 with project 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 Reaches 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.1.4-8.4.1 Alternative C - Defend, Scale for Lower Risk and Alternative D - Defend, Scaled for Low-Moderate Risk Alternatives

Alternative D First Action and Alternative C use the same exterior boundary condition at the Bay of 1.5 feet of sea level rise on top of MHHW. Alternative C does not include any adaptation after initial completion. Alternative D assumes a higher rate of sea level rise which would require a second action to address the higher sea levels.

With the higher rate of sea level rise the exterior bay conditions will reach a point where gravity flow is no longer an option and new pumps or upsizing of pumps will need to be installed to address the flood risk. Figure B.1.4-24 and Table B.1.4-16 show general locations and sizing estimates for the first action taken for the Alternative D and the only action take for Alternative C. Figure B.1.4-25 and Table B.1.4-17 show general locations and sizing estimates for the new additions for the second action of Alternative D.

Figure B.1.4-24: Alternative D First Action and Alternative C General Locations and Sizing of Interior Drainage Features

Table B.1.4-16: Alternative D First Action and Alternative C Summary of Interior Drainage Features

Alternative D First Action and Alternative C	HEC-RAS Interior Drainage Estimates
Reach 1 (North Shore)	5 - 4x3 Box Culverts with backflow prevention New flap gates on existing CSOs
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 CSOs
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 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)

cfs = cubic foot (feet) per second

CSO = combined sewer overflow

Figure B.1.4-25: Alternative D Second Action General Locations and Sizing of Interior Drainage Features

Table B.1.4-17: Alternative D Second Action Summary of Interior Drainage Features

Alternative D 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)

cfs = cubic foot (feet) per second

B.1.4-8.4.2 Alternative E - Defend Existing Shoreline, Scaled for Higher Risk Alternative

For Alternative E, the interior drainage structures will all be installed in during the first action taken. With 3.5 feet of sea level rise there will be limited gravity flow capabilities and will need to be installed during the first construction phase. All existing CSDs should still have back flow prevention added to allow the combined sewer system to be used at the full capability as long as possible. Figure B.1.4-26 and Table B.1.4-18 show general locations and sizing estimates for Alternative E.

Figure B.1.4-26: Alternative E General Locations and Sizing of Interior Drainage Features

Table B.1.4-18: Alternative E Summary of Interior Drainage Features

Alternative E	HEC-RAS Interior Drainage Estimates
Reach 1 (North Shore)	1 – 300 cfs pump (200 MGD) New flap gates on existing CSOs
Reach 2 (North Shore and Channel)	1 – 380 cfs pump (250 MGD) 1- 350 cfs pump (230 MGD) New flap gates on existing CSOs
Reach 3 (Channel)	1 – 1350 cfs pump (880 MGD) 1- 200 cfs pump (130 MGD) 2 – 150 cfs pump (100 MGD) 1 - 3 feet Circular Culvert with backflow prevention New flap gates on existing CSOs
Reach 4 (Islais Creek)	1 – 1900 cfs pump (1230 MGD) 1- 1400 cfs pump (910 MGD) 2 – 150 cfs pump (100 MGD) 2 - 2x2 Box Culverts with backflow prevention New flap gates on existing CSOs
Total Pump Capacity	6480 cfs (4130 MGD)

cfs = cubic foot (feet) per second

CSO = combined sewer overflow

B.1.4-8.4.3 Alternative F - Manage the Water, Scaled for Higher Risk Alternative

Tide gates allow water to pass to the Bay to match near FWOP conditions for the first action in Reaches 3 and 4. The lines of protection in Reaches 1 and 2 will still impede flow in those areas, so pumps are needed to account for the increased flood risk. All existing CSDs should still have back flow prevention added in the first action to allow the combined sewer system to be used at the full capability as long as possible.

Tide gates were converted to a smaller gated structure with pumps to handle the runoff for the second action. Storage in the creeks allow for a smaller pump size compared to other alternatives, but only slightly. It was assumed that the creek areas would be used as temporary storage while pumps are utilized to remove water. This is an estimate and could be changed during PED. If this is adjusted larger pump sizes may be needed.

Figure B.1.4-24 and Table B.1.4-16 show general locations and sizing estimates for the first action taken for Alternative F. Figure B.1.4-25 and Table B.1.4-17 show general locations and sizing estimates for the new additions for the second action of Alternative F.

Figure B.1.4-27: Alternative F First Action General Locations and Sizing of Interior Drainage Features

Table B.1.4-19: Alternative F First Action Summary of Interior Drainage Features

Alternative F First Action	HEC-RAS Interior Drainage Estimates
Reach 1 (North Shore)	1 – 300 cfs pump (200 MGD) New flap gates on existing CSOs
Reach 2 (North Shore and Channel)	1 – 380 cfs pump (250 MGD) 1- 350 cfs pump (230 MGD) New flap gates on existing CSOs
Reach 3 (Channel)	Tide Gate 2 - 3 feet Circular Culvert with backflow prevention New flap gates on existing CSOs
Reach 4 (Islais Creek)	Tide Gate 1 - 3 feet Circular Culvert with backflow prevention 2 - 2x2 Box Culverts with backflow prevention New flap gates on existing CSOs
Total Pump Capacity	1030 cfs (680 MGD)

cfs = cubic foot (feet) per second

CSO = combined sewer overflow

Figure B.1.4-28: Alternative F Second Action General Locations and Sizing of Interior Drainage Features

Table B.1.4-20: Alternative F Second Action Summary of Interior Drainage Features

Alternative F Second Action	HEC-RAS Interior Drainage Estimates
Reach 1 (North Shore)	No new additions
Reach 2 (North Shore and Channel)	No new additions
Reach 3 (Channel)	Convert Tide Gate Add 600 cfs Pump (390 MGD) Add 1000 cfs Pump (645 MGD)
Reach 4 (Islais Creek)	Convert Tide Gate Add 400 cfs Pump (260 MGD) Add 2600 cfs Pump (1680 MGD)
Total Pump Capacity	4600 cfs (2975 MGD)

cfs = cubic foot (feet) per second

B.1.4-8.4.4

B.1.4-8.4.5 Alternative G - Partial Retreat, Scaled for Higher Risk Alternative

For Alternative G pump are placed in locations that would reduce the number of pumps that would potentially be abandoned based on retreat in the second action. Along Mission Creek, a culvert is used in lieu of a small pump for this reason. The area is able to recede after 24 hours to near FWOP conditions. All existing CSDs should still have back flow prevention added in the first action to allow the combined sewer system to be used at the full capability as long as possible.

Due to the retreat in the second action, the flow along Mission Creek is forced to the north side causing a more intense runoff for the area. This required upsizing of the pumps in the area. In the Islais Creek area the retreat was more modest than in the Mission Creek area. The 400 cubic feet per second (cfs) (260 MGD) pump is abandoned and the 2000 cfs (1290 MGD) pump was upsized to 2300 cfs (1490 MGD) to accommodate the flows.

Figure B.1.4-29: Alternative G First Action General Locations and Sizing of Interior Drainage Features

Table B.1.4-21: Alternative G First Action Summary of Interior Drainage Features

Alternative G First Action	HEC-RAS Interior Drainage Estimates
Reach 1 (North Shore)	1 – 300 cfs pump (200 MGD) New flap gates on existing CSOs
Reach 2 (North Shore and Channel)	1 – 380 cfs pump (250 MGD) 1- 350 cfs pump (230 MGD) New flap gates on existing CSOs
Reach 3 (Channel)	1 – 1350 cfs pump (880 MGD) 1- 200 cfs pump (130 MGD) 1 – 150 cfs pump (100 MGD) 1 - 4 feet Circular Culvert with backflow prevention 1 - 3 feet Circular Culvert with backflow prevention New flap gates on existing CSOs
Reach 4 (Islais Creek)	1 – 2000 cfs pump (1290 MGD) 1- 900 cfs pump (580 MGD) 1 – 400 cfs pump (260 MGD) 1 – 150 cfs pump (100 MGD) 2 - 2x2 Box Culverts with backflow prevention New flap gates on existing CSOs
Total Pump Capacity	6180 cfs (4020 MGD)

CSO = combined sewer overflow

Figure B.1.4-30: Alternative G Second Action General Locations and Sizing of Interior Drainage Features

Table B.1.4-22: Alternative G Second Action Summary of Interior Drainage Features

Alternative G Second Action	HEC-RAS Interior Drainage Estimates
Reach 1 (North Shore)	No New Additions
Reach 2 (North Shore and Channel)	No New Additions
Reach 3 (Channel)	Remove 200 cfs Pump (130 MGD) 1- 600 cfs Pump (385 MGD) Upsize from 1350 cfs to 1850 cfs pump (new 500 cfs/325 MGD) 2 - 3 feet Circular Culvert with backflow prevention
Reach 4 (Islais Creek)	Remove 400 cfs Pump (260 MGD) Upsize from 2000 cfs to 2300 cfs pump (new 300 cfs/195 MGD)
Total Pump Capacity	1400 cfs (905 MGD)

Section B.1.4-9. 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.1.4-23.

Table B.1.4-23: 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 St.	8	Reach 3 (Channel)

A comparison to the FWOP condition of the validation points for each alternative is shown in Table B.1.4-24. 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.1.4-24: 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
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

Section B.1.4-10. Total Net Benefits Plan

The Total Net Benefits Plan (TNBP) is a hybrid of various alternatives. The final alignment consisted of structural measures on between to protect to 15.5 feet on 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 and Defend, Scaled for Low-Moderate Risk alternative. 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 TNBP are shown in Table B.1.4-25 and Table B.1.4-26.

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.1.4-25: 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 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)

CSO = combined sewer overflow

Table B.1.4-26: 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)

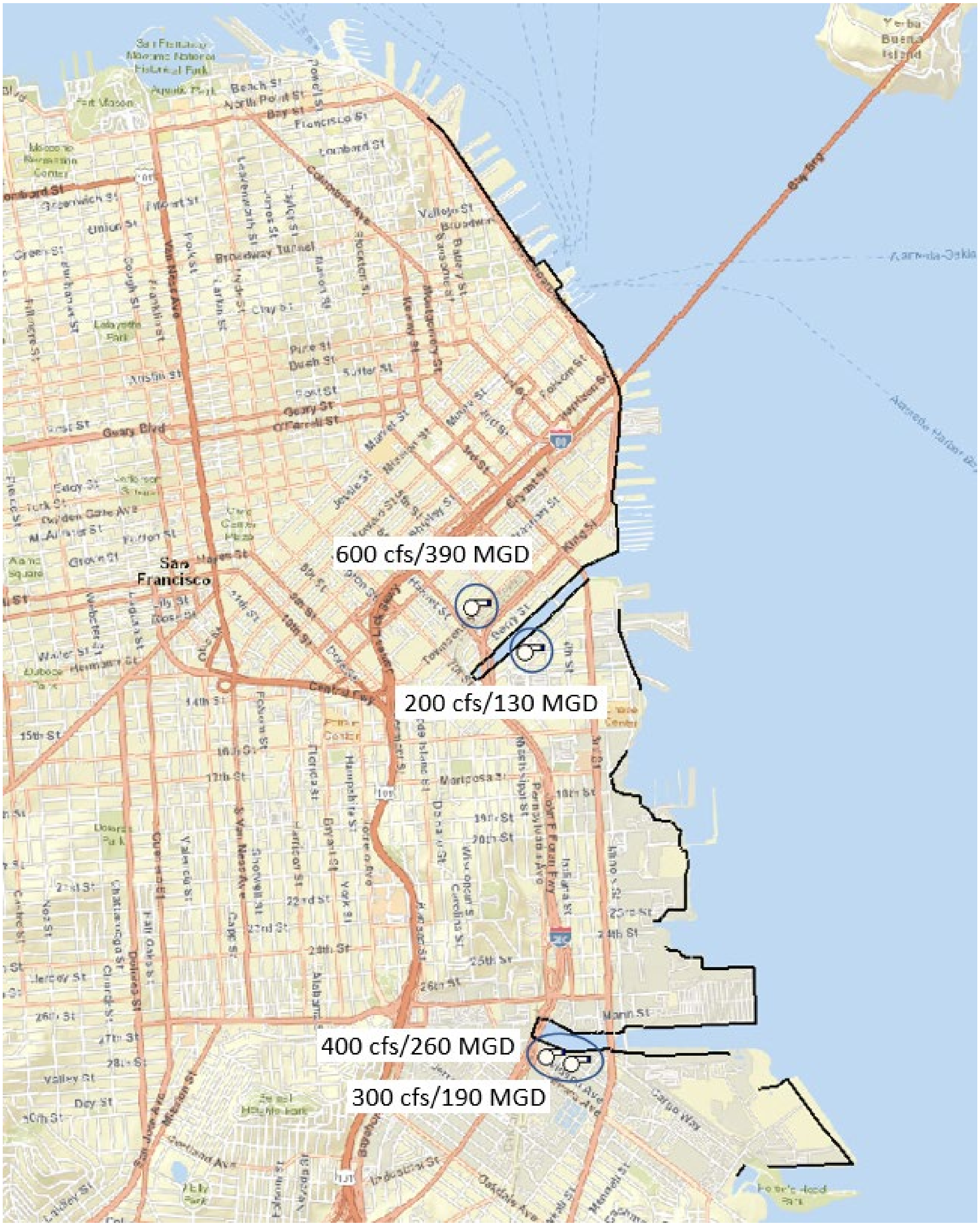

Figure B.1.4-31: TNBP Interior Drainage First Action General Locations and Sizing of Interior Drainage Features

Figure B.1.4-32: TNBP Interior Drainage Second Action General Locations and Sizing of Interior Drainage Features

Section B.1.4-11. Sensitivity Analysis

Three sensitivity analysis are run for all the alternatives. The first is looking at a 3-hour rainfall duration instead of a 24 hour, greatly increasing the intensity. The second is 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 is for 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 which is used for the study. All values in tables show the difference from FWOP, with negative values being flooding depths less than FWOP for results from the HEC-RAS modeling.

When the rainfall intensity is drastically increased to have the 24-hour rainfall volume occur over 3 hours, the peak flood depths increase up to 4 feet with the current pump sizing for some alternatives. Along Mission Creek and Islais Creek the shorter duration has the greatest impact. For the Reaches 1 and 2 (Northshore and northern portion of Channel) there is an increase to the flood depths, but not to the levels along the creeks. Table B.1.4-27 shows the results of the 3-hour duration sensitivity run.

Table B.1.4-27: 3-Hour Duration Sensitivity Simulation, Difference from 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	0.8	0.7	0.8	0.7	0.9	0.9
2	0.6	0.7	1.0	0.8	0.8	0.8
3	1.3	1.4	1.6	1.4	1.4	1.4
4	1.7	1.9	1.8	1.6	1.7	2.9
5	0.9	0.9	1.4	0.8	0.9	4
6	2	2.4	3.2	6	N/A	N/A
7	1.2	2.1	1.2	1	2.1	3.5
8	1.3	2.2	1.3	1.1	2.2	3.6

The SFPUC and SFPW have design criteria to meet the level of service requirements of having freeboard with the collection system of a 20% AEP 3-hour storm and 1% AEP 3-hour storm overland flow for street conveyance. As a sensitivity test the model is run using the 1% AEP NOAA best estimate rainfall for each of the alternatives and compared to the 1% Best Estimate 24-hour HEC-RAS. Although the 1% AEP 24-hour duration rainfall is more volume overall than the 1% AEP 3-hour duration rainfall, the theoretical rainfall captures the 3-hour duration intensity used for the design level of service. The results are summarized in Table B.1.4-28.

Table B.1.4-28: 1% AEP 24-Hour Best Estimate Rainfall Sensitivity, Difference from 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	0	0	0	0
2	0	0	0	0	0	0
3	0.1	0	-0.1	-0.1	-0.1	-0.1
4	0.1	0.1	-0.1	0	-0.6	-0.4
5	0	0	0	0	0	-0.4
6	-0.5	-1.7	0.2	-1.6	N/A	N/A
7	-0.3	-0.5	-0.2	0.1	-0.5	-0.5
8	-0.3	-0.5	-0.3	0.1	-0.5	-1.9

When comparing the simulations with the different bay boundary conditions for Alternative C and Alternative D first action, the validation points all had peak pools within a few hundredths of a foot. These results show that the bay conditions of the MHHW and 2-year storm surge elevations with 1.5 feet of SLC have little to no difference it the flood depths at the peak of the storm. The culverts included in the HEC-RAS modeling have flap gates with no negative flow for the feasibility-level evaluation. 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 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. This should be further explored during PED.

Section B.1.4-12. Modeling Challenges and Assumptions

A summary of the primary modeling challenges and assumptions for the HEC-RAS model development are provided in this section. The challenges and assumptions included in this section are geared to help with considerations for future evaluations of the interior drainage in the system.

Limited calibration data is available for this effort. Additional time should be spent defining loss rates and impervious areas. The losses, mannings values, and impervious areas were generalized estimates based on manuals and visual inspection.

The combined storm sewer system is evaluated in this study as artificial losses in the rainfall. For feasibility-level modeling this is considered an acceptable approach but will need to be evaluated in much more detail during PED.

Modeling the storage/transport structures is done through artificial losses and not physically modeled. Due to this the pumps were placed at low points in the model. The pumps are set to start pump almost as soon as water started to accumulate in the area and build up to a constant rate to the estimated pump size. This will need to be refined and tied into the transport storage boxes to efficiently remove water from the system.

The potential rise in ground water in both the FWP and FWP conditions are not evaluated for this effort. Currently dewatering takes place throughout the city which indicates a groundwater issue today. The modeling completed for interior drainage has not included reductions in pipe capacity or infiltration rates in the future due to the potential rise in the groundwater table. This should be explored in PED.

The model assumed that all water pumped from inside the line of protection is pumped directly out to the Bay. This is to determine pump sizing but none of the water is considered treated and was essentially acting as the CSDs would in high flow events. This will need to be reevaluated if it is determined pumping would need to pass through treatment.

2D Connections are used to simulate the lines of defense. These are roughly placed and consistent widths are used for all the alternatives. This allowed for the general representation of the height and placement of the structure to estimate the pump placement and sizing.

Pump and culvert placement and sizing is based on flood depths at validation points in the HEC-RAS model. Once locations in the model were identified to remove the most water in the area the pumps or culverts were added to reduce the flooding to near FWOP conditions at the validation points for all of the alternatives. The pumps are set to have a constant pump discharge and does not include any sort of efficiency curve development. The goal is to see what needed to be removed from the system to minimize the effects of the peak flood depths when lines of protection are added for climate resiliency.

Section B.1.4-13. References

City and County of San Francisco. 2020. Sea Level Rise Vulnerability and Consequences Assessment.

Federal Emergency Management Agency. 2021. Flood Insurance Study: City and County of San Francisco, California.

May, C., M. Mak, E. Harris, M. Lightner, and J. Vandever. 2016b. San Francisco Bay Tidal Datums and Extreme Tides Study. Prepared by AECOM for the Federal Emergency Management Agency Region IX and the San Francisco Bay Conservation and Development Commission

Phillips, S., Hamlin, S., and Yates, E. 1993. USGS. Geohydrology, Water Quality, and Estimation of Ground-Water Recharge in San Francisco, California.

San Francisco Bay Regional Water Quality Control Board. 1996. San Francisco and Northern San Mateo County Pilot Beneficial Use Designation Project, Draft Report.

San Francisco Public Utilities Commission. 2013. Sewer System Improvement Program: Bayside Drainage Basin Urban Watershed Characterization. Urban Watershed Assessment Team.

San Francisco Public Utilities Commission. 2013. SSIP Draft Model Documentation. Sewer System Improvement Program Management Consultant.

U.S. Army Corps of Engineers. 2018. Hydrologic Analysis of Interior Areas. EM 1110-2-1413.

U.S. Army Corps of Engineers. 2023. Hydrologic Engineering Center River Analysis Software Hydraulic Reference Manual.

U.S. Army Corps of Engineers. 2018 (rev 2022). Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects. ECB 2018-14.

U.S. Army Corps of Engineers. 2019. Risk Assessment for Flood Risk Management Studies. EM 115-2-101.

Climate Data Online. https://www.ncei.noaa.gov/cdo-web/

NOAA Tides and Currents. https://tidesandcurrents.noaa.gov/map/index.html?region=California

NOAA Atlas 14. https://hdsc.nws.noaa.gov/pfds/