SAN FRANCISCO WATERFRONT COASTAL FLOOD STUDY, CA

APPENDIX B – ENGINEERING [DRAFT]

JANUARY 2024

USACE TULSA DISTRICT | THE PORT OF SAN FRANCISCO

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Acronyms and Abbreviations

Acronym	Definition
ADA	Americans with Disabilities Act
AF	Artificial Fill
ARA	Abbreviated Risk Analysis
AWSS	Auxiliary Water Supply System
BART	Bay Area Rapid Transit
CCSF	City and County of San Francisco
CFRM	Coastal Flood Risk Management
CGS	California Geological Survey
cfs	cubic feet per second
CPT	Cone Penetration Test
CSRA	Cost and Schedule Risk Analysis
CWCCIS	Civil Works Construction Cost Index System
DSM	Deep Soil Mixing
EM	Engineering Manual
HEC-RAS	Hydrologic Engineer Center River Analysis System
HH&C	Hydrology, Hydraulics & Coastal
HTRW	Hazardous, Toxic, and Radioactive Waste
LiDAR	Light Detection and Ranging
LPW	Low-Pressure Water
MCE	Maximum Credible Earthquake
MDE	Maximum Design Earthquake
mm/yr	millimeter(s) per year

Muni	San Francisco Municipal Railway
NNBF	Natural and Nature-Based Features
NOAA	National Oceanic and Atmospheric Administration
NWF	Northern Waterfront
OBE	Operating Basis Earthquake
O&M	Operations & Maintenance
pcf	pounds per cubic feet
PDT	Project Delivery Team
PED	Preconstruction, Engineering and Design
PFMA	Potential Failure Mode Analysis
PG&E	Pacific Gas and Electric Company
POSF	Port of San Francisco
PWSS	Portable Water Supply System
SFFD	San Francisco Fire Department
SFMTA	San Francisco Mass Transit Authority
SFPUC	San Francisco Public Utilities Commission
SLC	Sea Level Change
SPT	Standard Penetration Test
SQRA	Semi-Quantitative Risk Assessment
SWF	Southern Waterfront
TNBP	Total Net Benefits Plan
TPCS	Total Project Cost Summary
TSP	Tentatively Selected Plan
USACE	U.S. Army Corps of Engineers

USGS	U.S. Geological Survey
VLM	Vertical Land Movement
WE	Wastewater Enterprise
WETA	Water Emergency Transportation Authority
YBM	Young Bay Mud

Section B-1. Existing Information and Data

B-1.1 Surveying and Mapping

Where available, Light Detection and Ranging (LiDAR) remote sensing method of data collection will be used to define the existing topography for the designs described in this section. LiDAR data for the study area was collected in 2010 and 2011 and was used to create 1-foot and 2-foot contours usable by AutoCAD and ArcGIS. The existing LiDAR topographic data referenced the NAD83 horizontal datum and the NAVD88 vertical datum. The LiDAR data was supplemented with aerial survey point data collected in 2014 for the southern half of the study area, and in 2019 for the northern half of the study area. Bathymetry data for the Embarcadero seawall portion of the study was collected in 2019 and used in the design of offshore measures.

Contours created using the available LiDAR data described above were used during the plan formulation phase to determine the required increase in elevation above existing ground levels required for the coastal defense measures. The estimated increase in elevation was used to calculate quantities used in the cost estimates.

The quantities were also estimated based on as-built drawings, other historical documents, and constant cross sections over defined lengths. The overall lengths of each cross section varied depending on the location and existing ground elevation. The reliability of the data used provided a realistic basis for the quantities used in the cost estimates. In areas of the coastline where conflicts and other constraints were encountered, the quantities were adjusted to account for the impact.

Extensive land-based surveys will be required during the Preconstruction, Engineering and Design (PED) phase to accurately capture all existing conditions of ground surface elevations, structure footprints, utility infrastructure both at the ground surface and below ground, transportation infrastructure, and other details needed to develop the formal plans and specifications for construction. The City and County of San Francisco (CCSF) is currently partnered with U.S. Geological Survey (USGS) to complete an updated Quality Level 0 LiDAR survey, anticipated to be completed in 2024. The LiDAR data can be evaluated against the land-based surveys to determine the extent of using the LiDAR survey information to supplement the more detailed land-based surveys.

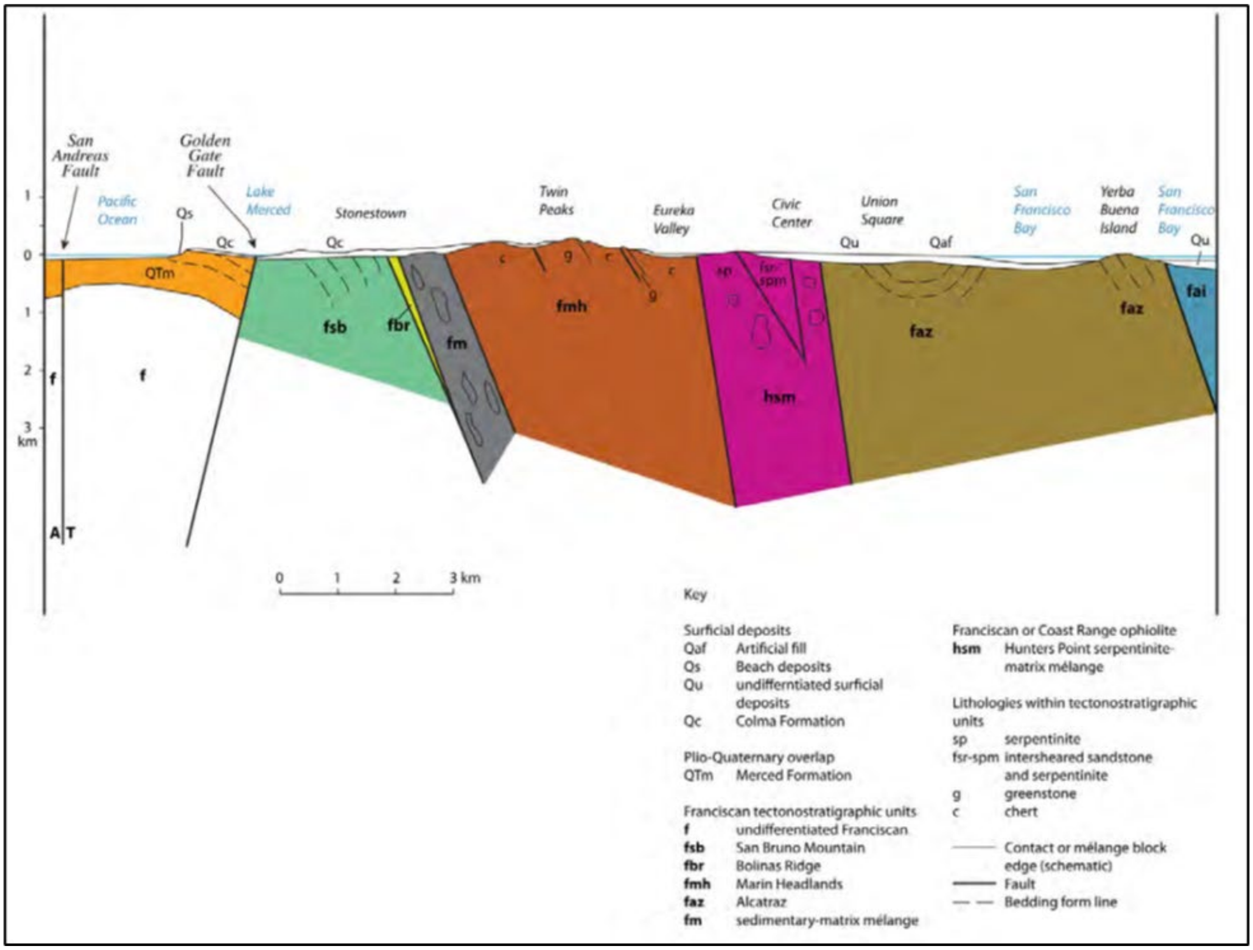
B-1.2 Geological and Geotechnical Assessments

B-1.2.1 Regional Geology

B-1.2.1.1 Basement Complex in San Francisco

The geology of San Francisco itself is comparatively simple and comprises several fault-bounded northwest trending bands of Mesozoic and Paleogenic basement rock overlain by various units of Quaternary surficial deposits, which includes artificial fill (AF). The basement rock is part of the Franciscan Complex which represents the accretionary prism (segments of the subducting slab of the oceanic plate crust) of the convergent continental plate margin where the Juan de Fuca plate was subducted beneath the margin of the North American continent. The Franciscan Complex mainly consists of marine deposited sedimentary and volcanic rocks in close association with bodies of serpentine. Following deposition, the Franciscan rocks were regionally uplifted resulting in extensive faulting and folding.

The bedrock in the project area is specifically assigned to the Alcatraz Terrane (**Figure B-1**). These rocks form the bedrock foundation beneath the Quaternary deposits observed along the project extent.



Source: Bartow and Johnson 2018

Figure B-1: Geologic Cross Section of San Francisco from Yerba Buena Island to the Pacific Ocean through Twin Peaks and Lake Merced

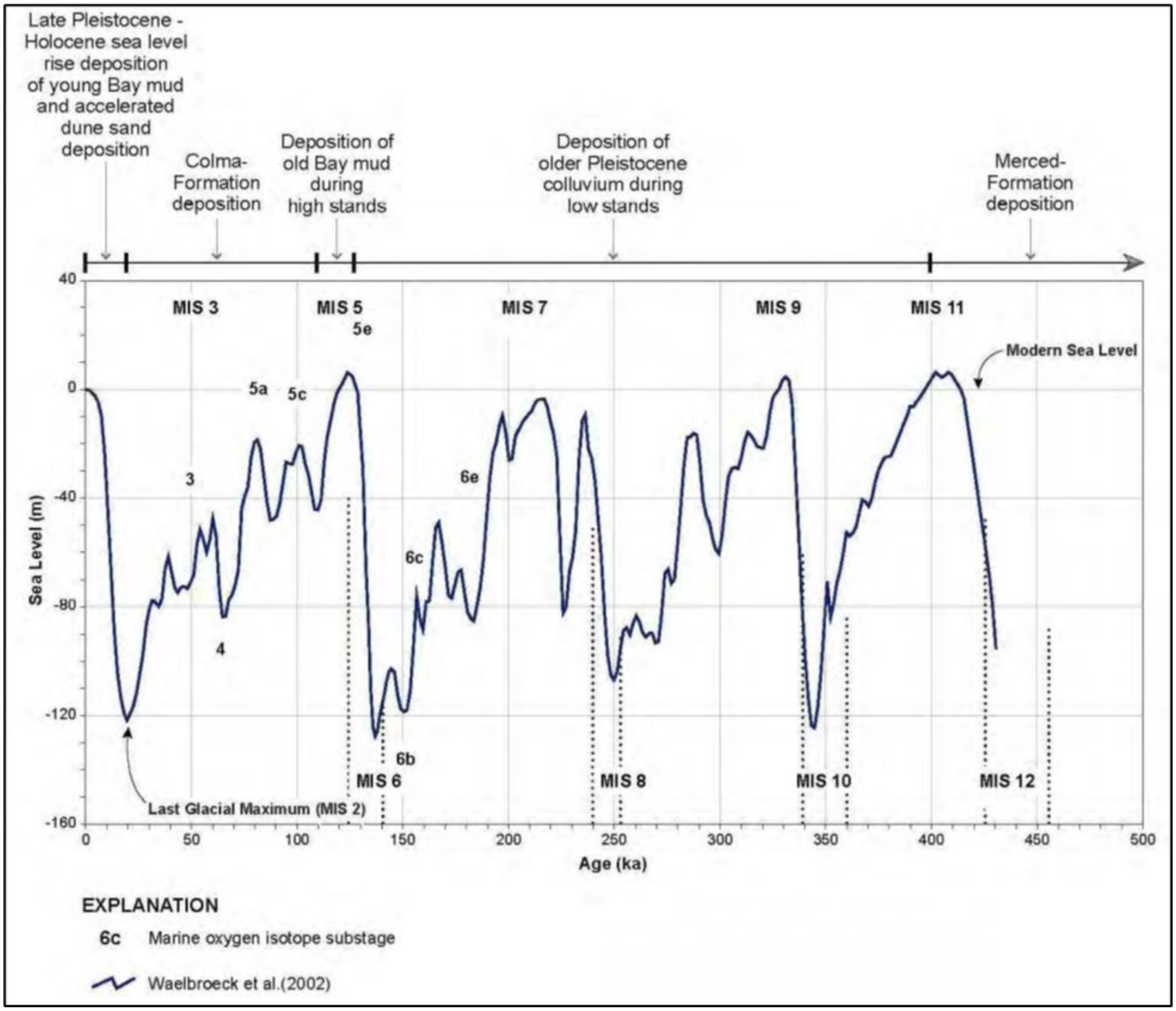
The Alcatraz Terrane crops out in northeast San Francisco, including Alcatraz and Yerba Buena Island, where it forms scattered hills that rise above the bay floor and flat-lying areas of the city. The hills probably represent erosion-resistant parts of a larger, but mostly hidden, northwest–southeast trending slab of the terrane, although it is possible that septa of mélange are present, though eroded and covered, between them. The Alcatraz terrane in San Francisco is bounded on the southwest by Hunters Point serpentinite-matrix mélange, and on the northeast by a parallel slab of Angel Island terrane.

The Alcatraz terrane is composed entirely of unfoliated sandstone, lacking the ocean crust and pelagic sediments present in some other terranes. The sandstone is biotite-bearing moderately lithic graywacke that includes minor potassium feldspar. Bedding ranges from thin to thick. The orientation of thick beds is still distinct because of thin shale partings. Locally the beds are tightly folded or disrupted into broken formation. Fine-grained metamorphic prehnite and pumpellyite are commonly seen in thin sections of the unfoliated graywackes of this terrane. In fact, pumpellyite was first recognized as a metamorphic mineral in Franciscan graywacke from Alcatraz Island.

B-1.2.1.2 Quaternary Deposits in San Francisco

Mesozoic basement complex rocks that occasionally crop out in the steep hills of San Francisco are unconformably overlain by a variety of Quaternary terrestrial, submarine, and estuarine deposits that collectively reflect multiple origins ranging from major sea-level and climatic fluctuations as well as tectonic uplift. Three major depositional phases are recorded in the Quaternary stratigraphy in the San Francisco Bay (Bay) Area: (1) Pleistocene shallow marine and near shore deposition, (2) accumulation of Pleistocene alluvium during sea level low stands, and (3) estuarine and eolian deposition during Pleistocene and Holocene high sea levels (**Figure B-2**).

Nearly all the major Quaternary stratigraphic packages in the Bay Area owe their origin to a rising or lowering of the sea over these glacial–interglacial cycles. For example, sea levels in the late Quaternary fluctuated in elevation by over 330 feet globally between glacial (low sea level) and intervening interglacial (high sea level) periods (**Figure B-2**) resulting in different deposits within the city limits of San Francisco. At least three prior episodes of deposition occurred during sea-level high stands (interglacial) approximately 410,000, 330,000, and 120,000 years ago, when the sea reached inland at sufficient elevation to flood the ancestral Sacramento–San Joaquin Delta and Santa Clara valley, and thus forming ancient estuaries (i.e., slack water and marsh deposits) much like the present-day San Francisco Bay (it is noteworthy to observe that the estimated sea-levels during these 3 high stands are approximately 30 feet higher than present day sea levels, which would predate any discernable human influence on sea levels). During glacial periods, sea levels were up to 400 feet lower than present-day, producing a shoreline as far west as the Farallon Islands. During these sea level low stands, fluvial, alluvial and eolian deposition predominated in the valleys and on hillslopes of the San Francisco Peninsula. In some cases, these non-marine Pleistocene deposits overlie earlier marine sediments deposited during earlier sea-level high stands and in other cases the non-marine deposits are buried by marine deposits related to more recent sea-level high stands. During the early Holocene between 9,500 and 8,000 years ago, the Pacific Ocean flooded the Golden Gate and sea level rose rapidly, about 2 centimeters per year (cm/yr) (0.8 inch per year). The rate of sea-level rise then declined by an order of magnitude between 8,000 and 6,000 years ago. In the last 6,000 years, sea level has risen more slowly at a rate of 0.1 to 0.2 centimeter per year (0.04 to 0.08 inch per year). Expansive tidal marshes in San Francisco Bay became established in the last 2,000 years as sea level stabilized at a slow enough rate to allow formation of persistent and widespread tidal marshes.



Source: Bartow and Johnson 2018

Figure B-2: Global Sea Level Changes over the last 500,000 Years Correlated to Quaternary Deposition in San Francisco

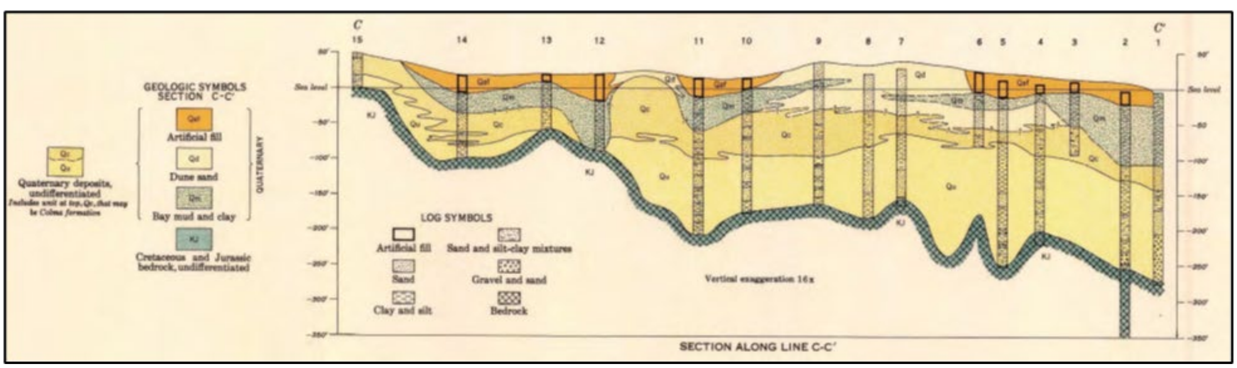
These Quaternary deposits are most influential in regard to construction and performance of the flood control structures associated with this project. **Figure B-3** depicts the following geologic units: Qc-Colma Formation, Qm-Bay Mud, Qu-Undifferentiated Pleistocene deposits (pre-Colma Formation alluvial deposits), and Qd-Dune deposits. The following provides a summary from oldest to youngest of the key geologic units on the San Francisco Peninsula, their association with glacial cycles, and their general engineering properties.

B-1.2.1.3 Lower Layered Sediments

The Lower Layered Sediments include soils of estuarine, alluvial, and colluvial deposition which were deposited approximately 120,000 to 400,000 years ago during older glacial–interglacial cycles (identified as older Pleistocene on Figure B-2). Per Bartow and Johnson 2018, these soils include poorly graded gravel (GP), clayey gravel (GC), poorly graded sand (SP), silty sand (SM), clayey sand (SC), fat clay (CH), and lean to sandy clay (CL). The fine-grained soils are generally stiff to hard, and the coarse-grained soils are typically dense to very dense based on in-situ test results. Testing of samples of fine-grained soils within the deposit are somewhat limited but generally have Atterberg limits values with liquid limits ranging from about 20 to over 50 and plasticity index range from about 5 to greater than 25. In situ water contents in this unit are typically close to the plastic limits of the soils, which corresponds to relatively stiff and moderately preconsolidated soils. Total densities range from approximately 115 to 140 pounds per cubic feet (pcf).

B-1.2.1.4 Old Bay Mud (Yerba Buena Mud)

The Old Bay Mud was deposited approximately 75,000 to 120,000 years ago during an earlier sea-level high stand that was the most recent predecessor to today's San Francisco Bay. This deposit consists of a sequence as much as 100 feet thick of relatively homogenous gray, marine silty clay, and occasional thin laterally discontinuous lenses of fine sand and shell-rich horizons. This deposit is generally present across the project site but is absent in some areas with shallower bedrock. Per Bartow and Johnson 2018, this deposit generally consists of interbedded stiff to very stiff clay (CH and CL) [N-values typically in the range of 10 to 60] and dense to very dense sand (SP) and silty sand (SM) [N-values typically in the range of 30-100+]. Fine-grained clay strata within the deposit generally have Atterberg limits values with liquid limits ranging from 20 to 85 and plasticity index range from 5 to 56. In situ water contents in this unit are typically close to the plastic limits of the soils, indicating the clayey soils are relatively stiff and moderately preconsolidated. Total densities range from 100 to 135 pcf.



Source: Bartow amd Johnson 2018

Figure B-3: Section Generally along Market Street

B-1.2.1.5 Upper Layered Sediment

The Upper Layered Sediment was deposited approximately 8,000 to 75,000 years ago as intermediate sea levels existed associated with entering and exiting the most recent glacial period. This deposit lies just above the Old Bay Mud. The deposits are dominantly of alluvial and eolian origin and are typically capped by a beach sand representing the encroachment of the shoreline in the early Holocene. Per Bartow 2018, the soils in this deposit are generally classified as poorly graded sand (SP), fat clay (CH), or lean to sandy clay (CL). The fine-grained soils are generally medium stiff to very stiff, and the coarse-grained soils are typically medium dense to very dense based on in-situ test results. Fine-grained clay strata within the deposit generally have Atterberg limits values with liquid limits ranging from 19 to 97 and plasticity index range from 3 to 67. The average in situ water content in this unit is typically slightly above the average plastic limit of the soils, indicating the clayey soils are relatively stiff and slightly to moderately pre-consolidated. Total densities range from 110 to 139 pcf.

B-1.2.1.6 Young Bay Mud

The poorly consolidated Holocene (younger than about 8,000 years) marine deposits commonly known as Young Bay Mud (YBM) were deposited during the most recent sea level intrusion coincident with present-day San Francisco Bay. The deposit occurs as a surficial blanket of fine sediments over the floor of the San Francisco Bay and the quiet water coves along the bay shoreline. The deposit is generally less than 165 feet thick. Within the San Francisco city limits, YBM is located along the margins of the Bay, between the modern shoreline and historical limit of the tidal marsh, and generally buried by AF. During construction of the Embarcadero seawall in the late 1800s and early 1900s, a wedge along the wall alignment of this deposit was dredged to approximately elevation -35 feet and backfilled with rock or sand upon which the seawall was constructed.

YBM is typically a clay that is highly compressible. It also can contain interbeds of alluvial fine sand lenses originating from nearshore streams and creeks along the margins of the current and historical bay shoreline. Per Bartow and Johnson 2018, the fine-grained soils are typically classified as lean clay (CL), fat clay (CH), or elastic silt (MH) with varying amounts of sand, shells, and organics. The coarse-grained soils are typically classified as silty sand (SM), poorly graded sand (SP), sandy or clayey silt (ML), and clayey sand (SC). The fine-grained soils are generally very soft to medium stiff, and the coarse-grained soils are typically loose to medium dense based on in-situ test results. Fine-grained clay strata within the deposit generally have Atterberg limits values with liquid limits ranging from 18 to 89 and plasticity index range from 0 to 58. The in-situ water contents in this unit are typically closer to the liquid limit than the plastic limit of the soils, consistent with the clayey soils being relatively soft and normally consolidated. Total densities range from <90 to 130 pcf.

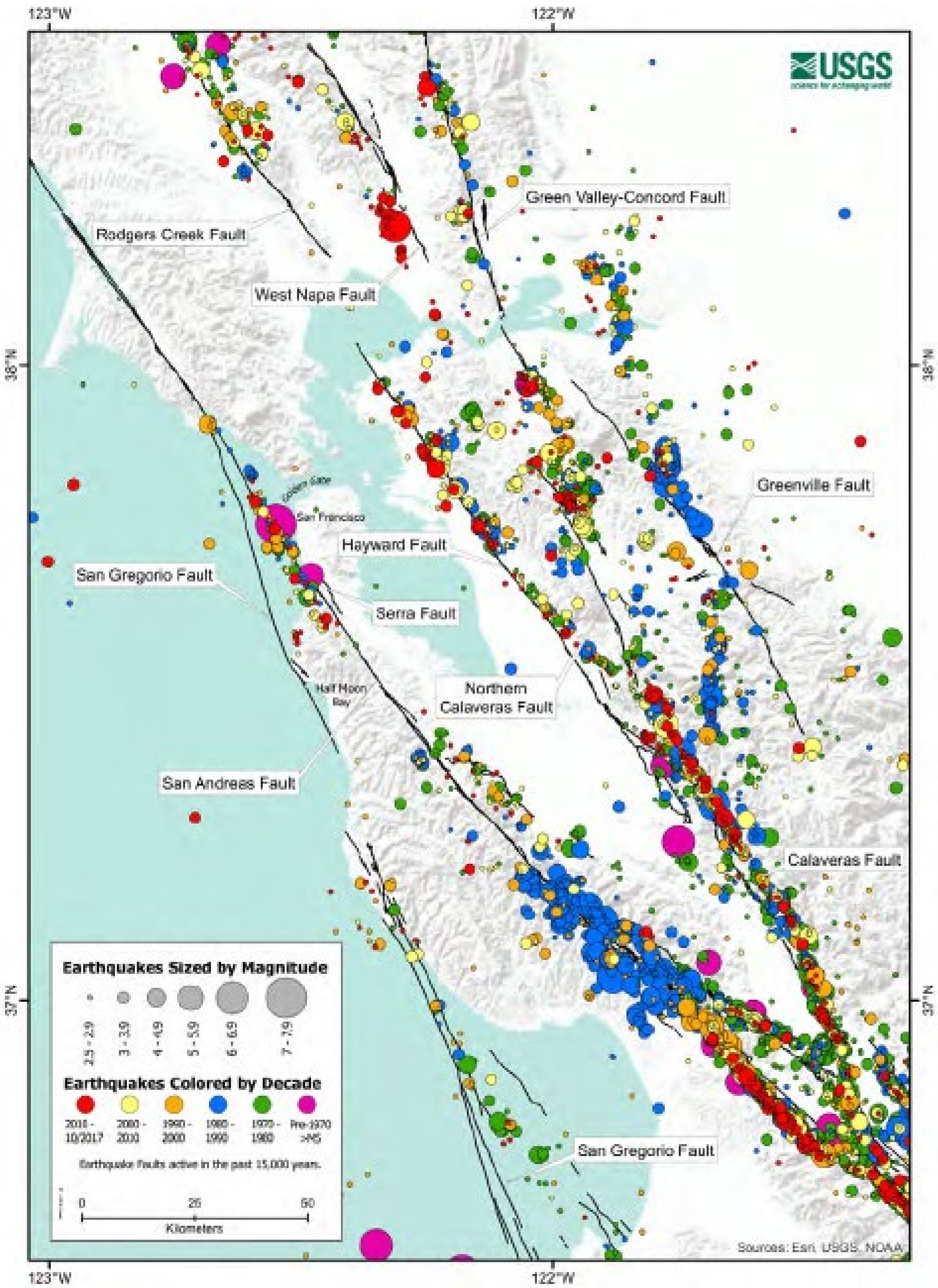
B-1.2.1.7 Artificial Fill (AF)

Since the 1800s the northeastern and eastern margins of the city of San Francisco have undergone significant landscape modification through multiple episodes of fill placement. Areas receiving the most extensive AF include inlets, coves, and saltwater marshes. Filling of the bay margins included a variety of fill placement methods ranging from dumping to hydraulic filling, as well as the intentional sinking of abandoned wooden sailing vessels. AF in the city includes a combination of local native sediments, including hydraulically placed dune sand and bay sediments, and miscellaneous construction debris, including brick, concrete rubble, metal, glass, and timber all of which were typically placed on top of weak YBM. In Yerba Buena Cove, AF includes debris from Gold Rush-era ships that were sunk and used as fill. Following the 1906 earthquake and fire, significant amounts of demolished structural debris was disposed as fill along the bay margin north and east of Market Street. Because of the different source materials and methods used for placement, the AF is highly variable and depends heavily on the source of the fill material. For instance, material sourced from the sand dunes and dredged bay sediments typically consists of loose, poorly graded sand to clayey sand and soft clay and often may be mixed or overlie rubble such as brick, asphalt, concrete, wood, broken rock, and scattered gravel.

Per Bartow and Johnson 2018, the fill materials consisting of soils are generally classified as poorly graded sand (SP), silty sand (SM), or sandy clay (CL). The fine-grained soils are generally soft to medium stiff, and the coarse-grained soils are typically very loose to medium dense based on in-situ test results. These soils are generally normally consolidated and due to the high variability of the composition of the deposit matrix are subject to high variability in settlement potential within short distances.

B-1.2.2 Seismicity

San Francisco lies within the San Andreas fault system, which accommodates a significant fraction of the tectonic motion between the Pacific and North American plates. Although active faults of the San Andreas fault system (**Figure B-4**) do not transect city boundaries, the inevitable slip on these nearby faults will cause future large earthquakes that will shake San Francisco and produce strong ground motions damaging to buildings, lifelines, and other infrastructure, and cause economic losses and fatalities. The shaking caused by these earthquakes will also induce liquefaction and landslides, with the most extensive ground failure likely to be caused by liquefaction around the margins of the city, which includes all the project extent.



Source: USGS Quaternary fault and fold database; https://earthquake.usgs.gov/hazards/

Figure B-4: Map of Historical (M>2.5) Seismicity from the Northern California Earthquake Data Center (NCEDC 2014) Catalogue, with Known Active Faults of the San Francisco Bay Area

In the 1980s, groups of experts began to make use of historical seismicity data, paleoseismological data (constraining estimates of prehistoric earthquake timing and rupture extent), and fault slip-rate data, to produce earthquake rupture forecasts in which the likelihood and magnitude of future earthquakes is estimated (**Figure B-5**). The most recent probabilistic earthquake rupture forecasts were developed and published in 2016 in UCERF3, the Third Uniform California Earthquake Rupture Forecast. This study concluded there is a 72% chance of at least one M≥6.7 earthquake striking the San Francisco Bay Area over the 30-year period 2014–2043. The San Andreas fault (22%) and the Hayward–Rodgers Creek fault (33%) are the most likely to produce a large earthquake damaging to San Francisco over this time. **Table B-1** provides results from UCERF3 indicating the average time between earthquakes together with the likelihood of having one or more such earthquakes in the next 30 years beginning in 2014. Values listed in parentheses indicate the factor by which the rates and likelihoods have increased, or decreased, since the previous model (UCERF2). The Readiness column indicates the factor by which likelihoods are currently elevated, or lower, because of the length of time since the most recent large earthquakes. It is important to note that actual repeat times will exhibit a high degree of variability and will almost never exactly equal the average listed here.

Table B-1: UCERF3 Earthquake Repeat Time, Likelihood, and Readiness Estimates

	San Francisco Region			
Approx Historical EQ	Magnitude (greater than or equal to)	Average repeat time (years)	30-year likelihood of one or more events	Readiness
	5	1.3 (0.7)	100% (1.0)	1.0
	6	8.9 (1.0)	98% (1.0)	1.0
~Loma Prieta	6.7	29 (1.1)	72% (1.1)	1.1
>Loma Prieta and closer	7	48 (0.9)	51% (1.3)	1.1
~1906 SF Event	7.5	124 (0.7)	20% (1.6)	0.9
>1906 SF Event	8	825 (0.7)	4% (1.9)	1.0

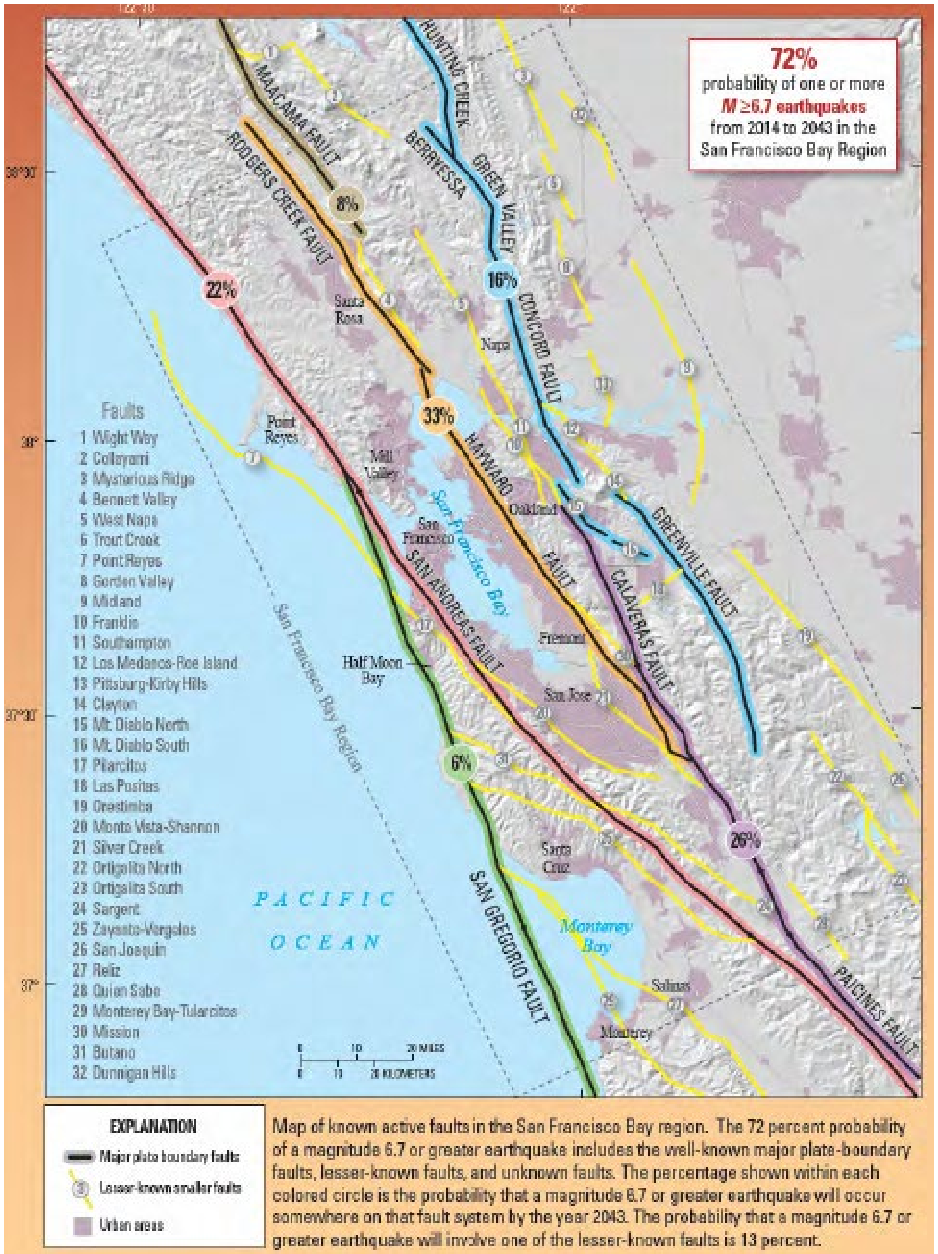


Figure B-5: Map of Active Faults in the San Francisco Bay Area, with Probabilities That a M≥6.7 Earthquake Will Occur Along Each Major Fault Over the 30-year Period 2014–2043

Based on multiple conversations with the Vertical Team and HQ personnel, the overall final seismic design requirements for this project should conform to the requirements of ER 1110-2-1806 *Earthquake Design and Evaluation for Civil Works Projects* (USACE 2016). This guidance document defines two design earthquake levels, each with their own associated performance objective. The Operating Basis Earthquake (OBE) is defined as an event that is reasonably expected to occur within the design life of the project and is typically taken as a ground motion with a 50% probability of exceedance in 100 years (144-year return period). The OBE is intended to protect against economic loss of the project investment and alternate return period ground motions (higher or lower) can be considered based on economic considerations. The Maximum Design Earthquake (MDE) is defined as the maximum ground motion for design purposes with an accompanying performance objective that allows for severe damage or economic loss but without loss of life or catastrophic failure. For normal structures, MDE is typically taken as a ground motion with a 10% probability of exceedance in 100 years (950-year return period); however, this MDE return period can be adjusted based on the overall project hazard potential. For critical structure (where failure would result in loss of life), the MDE is defined as the Maximum Credible Earthquake (MCE) which is the largest earthquake that can reasonably be expected to be generated by a specific source based on seismological and geological evidence. Currently, the flood protection elements have been defined as normal structures since the usual tidal water levels anticipated to exist throughout the project life are not expected to be at a level that would cause loss of life if damage were sustained by the flood control measures during the design seismic event. A formal life safety risk assessment needs to be performed during PED to confirm the classification of the flood control measures as normal structures.

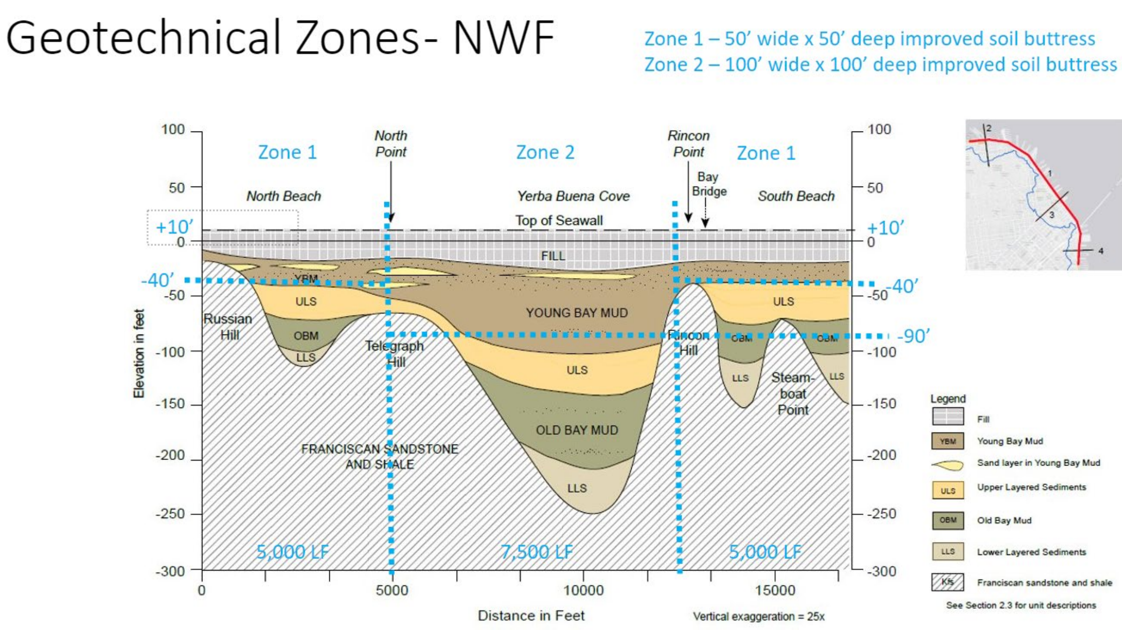
Based on the 2018 USGS seismic hazard results for the project site, the OBE ground motion is estimated to have a peak horizontal ground acceleration (PGA) of 0.31 g and an associated mean moment magnitude of 6.9. The MDE ground motion is estimated to have a PGA of 0.59 g and an associated mean moment magnitude of 7.1. The PGA values are based on an estimated site class of DE with a Vs,30 value of 185 meters per second. These acceleration values indicate significant ground motions can be expected at the project site, even for the OBE event.

In addition to the significant ground motion potential at the site, significant geologic seismic hazards of liquefaction and lateral spreading are expected to occur during the design earthquake events. Damaging liquefaction and lateral spread in San Francisco have occurred during numerous large historical earthquakes, including the 1868, 1906, and 1989 earthquakes. The California Geological Survey (CGS), Seismic Hazards Mapping Program produces regulatory maps for the hazards of surface rupture, liquefaction, and earthquake-induced landsliding. **Figure B-6** shows the CGS map of liquefaction and earthquake induced landslides which indicates the project site is fully within this geologic seismic hazard. However, this figure does not identify lateral spread vulnerability zones known to exist along a majority of the study area shoreline as identified in the 2020 Multi-Hazard Risk Assessment or 2022 Initial Southern Waterfront Earthquake Assessment completed by the Port of San Francisco (POSF).

Figure B-6: California Geological Survey's Seismic Hazard Zones Map Showing Zones of Liquefaction and Earthquake-Induced Landslides

The current seismic designs are focused on the mitigation of the geologic seismic hazards of liquefaction and lateral spread of the foundation and shoreline soils needed to stabilize performance of the flood control elements. These designs are currently based on fairly robust seismic hazard evaluations and preliminary designs recently performed by the local sponsor and incorporating the Project Delivery Team (PDT) engineering judgment. The current seismic design mitigation methods are not differentiated based on OBE or MDE ground motions or performance objective but are judged to be a reasonable, feasibility-level design that will satisfy the requirements of ER 1110-2-1806 (USACE 2016). To provide additional granularity to the design the PDT delineated 2 generalized soil models (designated Zone 1 and Zone 2) along the project length based on the overall thickness of the AF and YBM deposits. These soil deposits represent the soils wherein virtually all the geologic seismic hazards exist for the project. **Figure B-7** depicts a geologic profile along the Northern Waterfront (NWF) area and illustrates how the overall thickness of the deposits varies over this part of the project length. Areas designated as a Zone 1 soil model have a thickness of AF and YBM of approximately 50 feet, while Zone 2 areas have a thickness of approximately 100 feet. **Figure B-8** is a plan view of the areas designated as either Zone 1 or Zone 2 along the entire project length. Based on results from the local sponsor's study and engineering judgment, the current design estimates that a deep soil mixed (DSM) stabilization buttress with a 1:1 height to depth ratio (50 feet by 50 feet for Zone 1 and 100 feet by 100 feet for Zone 2) will be sufficient to stabilize the foundation and shoreline soil to be consistent with the ER 1110-2-1806 requirements for both OBE and MDE.

In several of the project structural alternatives (Alternatives F and G) the line of protection retreated from the shoreline in areas of the southern waterfront (SWF) by up to several thousand feet. In this study the geologic seismic hazard of lateral spread was estimated to only extend landside of the shoreline by a distance equal to or less than 250 feet. This distance was estimated using an estimated angle of 7 degrees from the horizontal for the failure plane of the lateral spread mass and an average bay depth near the shoreline of 30 feet (represents the open face depth). This failure plane was project inland until it reached the ground surface, which was approximately 250 feet [30/tan(7)]. For flood control measures which were located more than 250 feet from the shoreline the geologic seismic hazard was assumed to only consist of potential liquefaction. As such, the required foundation soil mitigation DSM block requirements were reduced to be 50 feet wide by 40 feet deep for levees and 20 feet wide by 40 feet deep for structural walls (**Figure B-9**). The reduced depths were based on geologic information that indicates the AF and YBM thicknesses decrease as the distance from the shoreline increases.



Source: CH2M/Arcadis Team 2020c

Figure B-7: NWF Geologic Profile with Soil Zones 1 and 2 Delineated

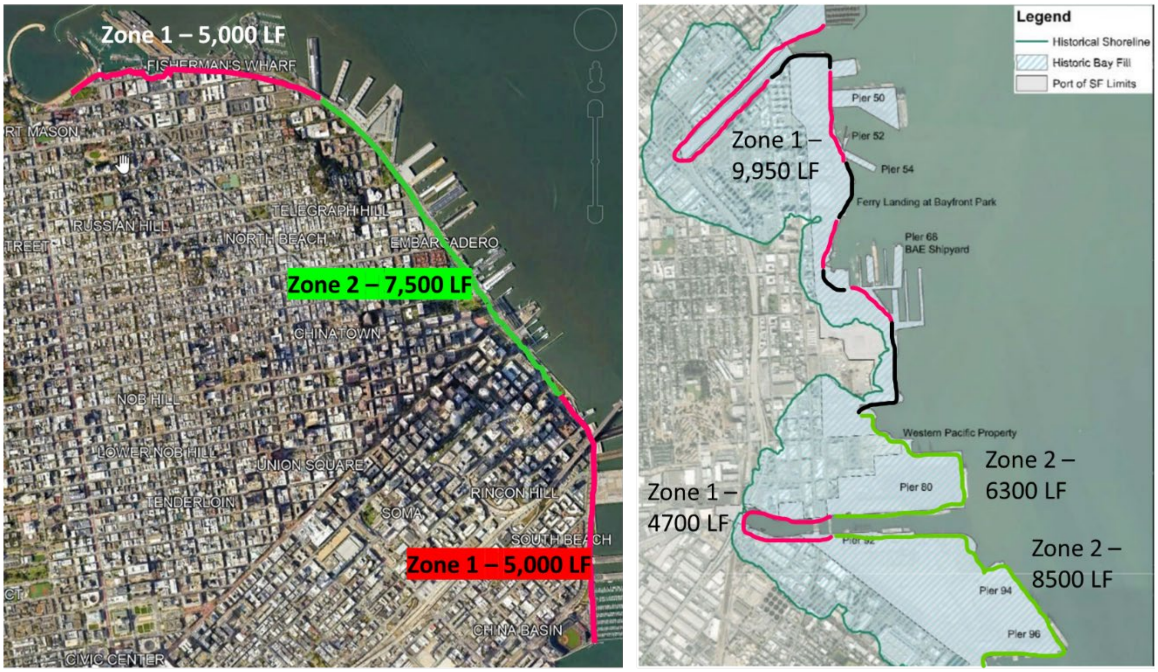


Figure B-8: Plan Views of the NWF and SWF With soil Zones 1 and 2 Delineated

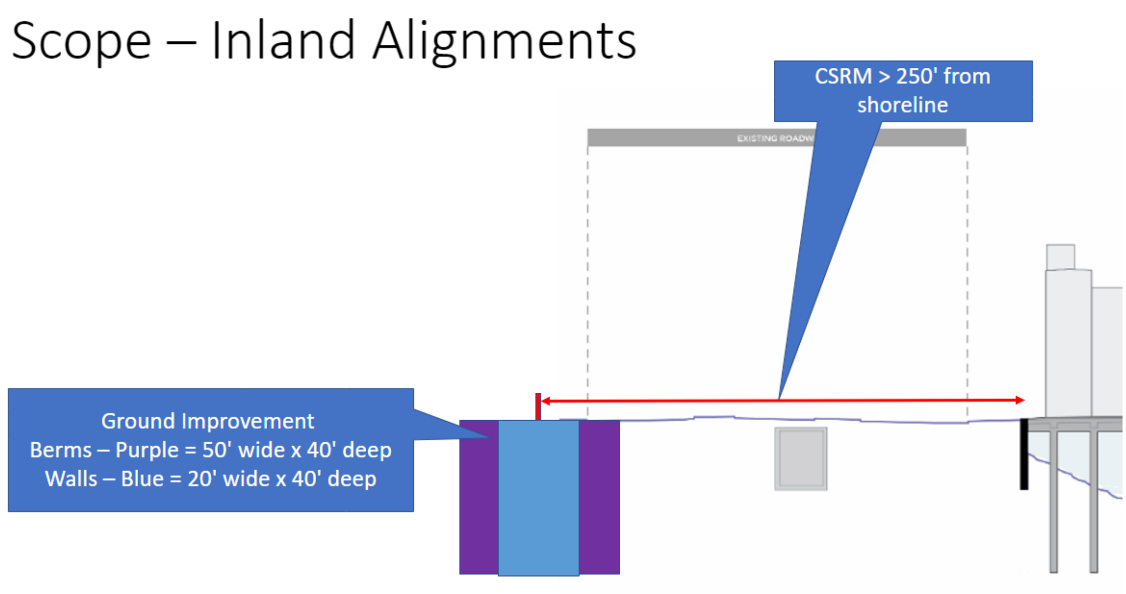


Figure B-9: Ground Improvement Requirements for Measures at Least 250 Feet From the Shoreline

B-1.2.3 Subsidence

The primary contributors to vertical movements for this project are plate tectonic associated with the plate boundary within which the site exists and traditional geotechnical consolidation settlement. These contributing effects need to be considered when determining sea level rise potentials and also when evaluating heights of proposed flood control measures.

The team researched various resources for subsidence information in the study area. U.S. Army Corps of Engineers (USACE) curves are based on a simple quadratic equation: E(t) = M*t + b*t^2 where E(t) is the eustatic sea level change (SLC), M is a constant for the global SLC (currently 1.7 millimeters per year [mm/yr]) plus the Vertical Land Movement (VLM) as determined at the gauge of interest, b is a constant relating the SLC to assumed SLC of 0.5, 1.0, and 1.5 meters by 2100, and t is time relative to 1992.

Estimating Vertical Land Motion from Long-Term Tide Gauge Records (Technical Report NOS CO-OPS 065) has a list of the estimated M and also VLM values for gauges of interest in the study area (San Francisco, Alameda, and Redwood City). The M values range from 0.82 mm/yr to 2.06 mm/yr. The average rate of VLM for the San Francisco gauge is -0.36 mm/yr. The team recognizes the contribution of tectonic plate movement to relative SLC is already accounted for in the development of the USACE RSLC curves used in the hydraulic analysis.

Consolidation settlement on the other hand is not considered in the relative SLC curves so must be accounted for in the engineering design. The VLM due to consolidation settlement will differ across the waterfront based upon the differing subsurface conditions, changing groundwater levels, and increased overburden stresses from general grading or added flood protection measures. Features or site changes that will increase the magnitude of consolidation due to the addition of overburden will address settlement in one, or more ways:

- Ground improvement – stiffen the effective soil matrix of the foundation soils by adding stiffer elements created by adding DSM elements, jet grout columns, or other means of cement-soil improvement.
- Overbuild – account for consolidation settlement by adding additional height to the flood control measure (typically only for levees).
- Balance fill approach – excavate and backfill with light weight fill such that the applied stress is equal to the existing stress within the supporting soils.
  A possible design issue associated with this method is that it will require use of light weight fill that is not susceptible to flotation during flood conditions.

The overbuild and ground improvement approaches have been adopted at this phase of the study, but these will be refined during PED based upon further subsurface exploration, analyses, and consideration of construction means and methods.

B-1.3 Numerical Models

B-1.3.1 Hydrology, Hydraulics and Coastal

The complete Hydrology, Hydraulics and Coastal (HH&C) analyses are detailed in Sub-Appendix B.1, which provides a full explanation of the calculations and modeling performed for this study.

B-1.3.2 Interior Drainage

Water levels in the Bay are expected to rise and coastal defenses are planned to be constructed to prevent flooding. The new defense structures will block storm water flowing overland from entering the Bay, and storm water that would normally drain through many of the controlled outflow locations will be countered by reverse hydraulic head from the high bay water levels. These impacts would increase the potential of interior flooding. To protect the area from flooding, new pumps and gravity flow structures will need to be placed throughout the study area to remove water that would otherwise be impeded by the impermeable line of defense. In this study, pumps will be placed in low spots throughout the city to move water from the inside of the coastal defense to the Bay.

High level modeling was performed using the Hydrologic Engineer Center River Analysis System (HEC-RAS) and its 2D modeling capabilities. The software was used to determine the general locations and sizes of pumps and culverts throughout the study area, with the goal to reduce the impacts of the coastal defenses on interior drainage by maintaining the ‘without project’ storm water conditions (i.e., drainage flows and storage).

The complete Interior Drainage analysis is detailed in Sub-Appendix B.1.4 and provides a full explanation of the calculations and modeling performed.

Section B-2. Existing Conditions

B-2.1 Shoreline Structures

B-2.1.1 Embarcadero Seawall

The Embarcadero seawall was built over 100 years ago. The seawall comprises 22 unique sections, some with or without a bulkhead wharf, and approximately 22 remaining finger piers. There is considerable variability along the 3-mile stretch of waterfront that forms the Embarcadero. The marine structures currently located along the waterfront were constructed with a variety of materials and construction styles, spanning over a century. The oldest marine asset on the waterfront is also one of the most unique and supports one of most recognizable buildings in San Francisco. Built circa 1894, the Ferry Building is supported on several large concrete piers, each founded on tight clusters of timber piles. Much of the seawall was built after 1900, sometimes in conjunction with a bulkhead wharf structure, typically adding 40 feet of additional space over the water.

The oldest remaining finger piers are in the South Beach area; built around 1910. They were founded on large, circular, concrete piers (also referred to as “cylinder piles”). Soon after, piers were added around Pier 33. Reinforced concrete piles were driven into the soils, typically in sizes of 16-inch to 20-inch square. In most of this area, the pier and bulkhead wharf were built concurrently and connected to each other by the reinforced concrete deck. During the 1920s, the Fisherman’s Wharf area was developed. Timber piles and timber decking were primarily used for marine structures in this area, as well as some small connecting wharves (also referred to as “infill” structures) elsewhere on the waterfront (such as between Pier 26 and Pier 28).

B-2.1.2 Large Concrete Pile Structures

Structures founded on large diameter concrete piles are typically located south of the Bay Bridge at Piers 26, 28, 30/32, 38, and 40. These structures are characterized by large concrete cylinder piles, that were cast in place during initial construction, circa 1908-1912. The piles range in size from 36 to 48 inches in diameter. The seawall in this area typically has a trapezoidal cross-section concrete bulkhead of varying depths. In some locations, the bulkhead is supported by timber piles. For example, around Piers 26-32, the bulkhead wall includes four timber piles at 3 feet on-center. Other locations to the south have no piles or two piles below the bulkhead. The rest of the bulkhead wharf (within the area of the piers) typically comprises a single cylinder pile and concrete encased steel beam that is seated on the bulkhead wharf.

B-2.1.3 Timber Piled Shoreline Wharves

The timber piles wharves are in two types. wharves in the South Beach area between Pier 24 and Pier 28. In these locations, the adjacent bulkhead wharf includes a trapezoidal concrete bulkhead wall and a timber pile with concrete jackets, built circa 1910. The connecting wharves were then added between 1927 and 1935 with eucalyptus piles and a timber deck. The second group of connecting wharves are near the northern end of the waterfront, including the wharves north of Pier 35. These are also timber deck, supported on timber piles, with an open area (no building) on top.

B-2.1.4 Timber Piles with Concrete Jackets

The main distinguishing characteristic for this structural group is the pier piles, which include a timber pile driven to roughly the mudline elevation, then wrapped with a reinforced concrete jacket. Typically located within the northeast waterfront, including (but not limited to) Piers 9, 17, 19, and 23. These piers were typically built in the early 1930s, except for Pier 17, which was built in the 1910s. These piers mainly have concrete decks, except for Pier 17 which has a timber deck. Generally, the bulkhead wharf adjacent to these piers includes a shallow (approximately 10 feet deep) concrete stem wall supported on a 16-inch-square concrete pile, and four additional wharf piles that are 16-inch or 18-inch-square reinforced concrete. The bulkhead wharf structures in this area were typically built between 1910 and 1915.

B-2.1.5 Square Precast Piles and Concrete Deck

Structures typically located north of Battery Street and constructed prior to 1920 consist of concrete beams and decks supported on square precast concrete piles. The bulkhead wharf was constructed integrally with a shallow concrete stem wall supported on a single row of 16-inch-square precast piles.

B-2.1.6 Fisherman's Wharf

The structures at Fisherman's Wharf are of traditional timber construction, including driven timber piles, square timber pile caps, and timber decking. They may also include asphalt or concrete topping above the timber decking. These marine structures come in a variety of structural configurations, including two main types: marginal wharves – oriented adjacent to the seawall with a sloped soil (or rock dike) profile across its section, and finger piers – projecting out from a seawall or offset from the seawall with a fairly level soil profile. The structures support buildings such as restaurants.

B-2.1.7 Ferry Building

The foundations of the Ferry Building, which consists of construction unique from any other assets on the waterfront, consists of four rows of large unreinforced concrete piers supported by timber cribbing and clusters of closely spaced timber piles (approximately 3 feet on-center). The timber pile clusters are topped with concrete that appears to have been cast-in-place atop the cribbing, possibly with the use of dewatering and shoring. The pyramidal stepped concrete piers then transition to concrete arches in both the longitudinal and transverse directions between piers. The building columns appear to be supported directly above the piers. The landward edge of the Ferry Building is supported directly on a concrete seawall of similar construction.

B-2.1.8 Ferry Plaza

The Ferry Plaza was constructed circa 1971 and consists of a 14-inch-thick concrete deck supported by both plumb and batter square prestressed concrete piles. Unlike structures within RS-6, the prestressed piles are connected into the deck with mild steel dowels rather than developing the prestressing strands directly into the deck. Following observed structural damage from the 1989 Loma Prieta earthquake, the batter piles were repaired in 1995 with new reinforced concrete and bolted-steel channel sections at the pile-deck connections.

B-2.2 Utility Systems

B-2.2.1 General

Utilities have two general categories: public and private. Public utilities are owned, operated, and maintained by a city department or local municipality. These utilities generally need to follow city and county regulations and can be subject to public control. Private utilities are owned, operated, and maintained by an investor-owned company. These utilities are typically regulated at the state level but can have more flexibility in the management of their utility system based on investment decisions, operations, and customer rates.

The primary public utility agency in San Francisco is the San Francisco Public Utilities Commission (SFPUC), a department of the CCSF. This agency is composed of three service utility systems: Wastewater, Water, and Power. These utility systems are operated, maintained, and developed by the SFPUC, which serves residential and commercial accounts, as well as some municipal facilities.

The private utility companies contacted during the 2020 Multi-Hazard Risk Assessment by the POSF (CH2M/Arcadis Team 2020e) were dry utilities, which are either cable, electric, telephone, natural gas, television, or fiber optic. These utility companies include Pacific Gas and Electric Company (PG&E), Verizon, Comcast, and AT&T.

Due to privacy of business information, existing information was more readily available for the public utilities than private utilities. The following subsections summarize the existing utility information obtained and describes the differences in information between public and private utilities.

B-2.2.2 Combined Wastewater

The wastewater system in most of the study area is a combined sewer system operated by the SFPUC Wastewater Enterprise (WE). The primary wastewater system operated by the SFPUC is a combined sewer system which collects, transports, and treats stormwater and sanitary sewage through building roof drains, storm, and sewer laterals, and drain inlets (**Figure B-10**). The system is also essential for drainage for the CCSF. Together, the collection system (with its storage capacity) and outfalls prevent flooding of public streets, sidewalks, parks, and buildings.

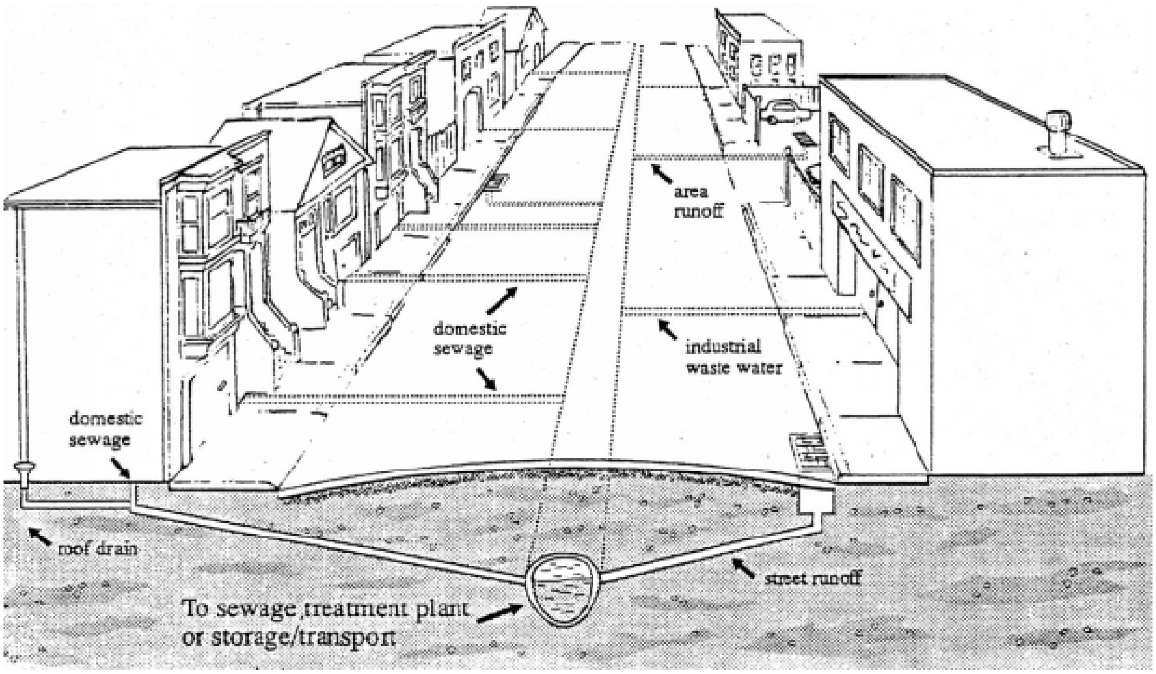


Figure B-10: Representation of Combined Sewer System

Within the study area, conveyance includes gravity mains, tunnels (deeper sewers), and transport storage boxes (approximately 15-foot by 20-foot large interconnected underground structures).

The combined wastewater system has two operational conditions, wet and dry. During dry operations the Southeast Treatment Plant (located outside of the study area) can treat all storm and wastewater for the bayside area. During wet operations the North Shore Pump Station and North Shore Force Main assist the otherwise largely gravity-fed system. The North Point Wet Weather Facility is activated for wet-weather flows when the main Southeast Treatment Plant approaches capacity. This facility discharges treated effluent to the Bay through the North Point Outfalls under Piers 33 and 35. However, if the transport storage boxes and both treatment facilities reach capacity during wet weather, the transport storage boxes discharge to the Bay through multiple combined sewer discharges (outfall/overflow structures). San Francisco's wastewater system varies in age from newly installed to over 100 years old, with portions of the system built prior to 1892.

B-2.2.3 Separated Wastewater System

Within certain pockets of the study area, the wastewater system managed by SFPUC or the POSF is not a combined sewer system but rather a separate sewer system. Sanitary sewer is directed to the SFPUC treatment plants while stormwater is managed within localized pockets before discharging to the Bay. These localized separate sewer areas mainly occur in the SWF neighborhood of Mission Bay and on the Port's industrial facilities at Piers 80 to 96. The plan impacts to these localized systems will be studied further in subsequent phases of the feasibility study or PED.

B-2.2.4 Low-Pressure Water (LPW)

The SFPUC WE operate the Hetch Hetchy Regional Water System (**Figure B-11**), providing water to 2.7 million customers in Alameda, Santa Clara, San Mateo, and San Francisco Counties. This system includes reservoirs and storage tanks, pump stations, fire hydrants, distribution pipelines, isolation valves, and automatic air valve (CH2M/Arcadis Team 2020e).

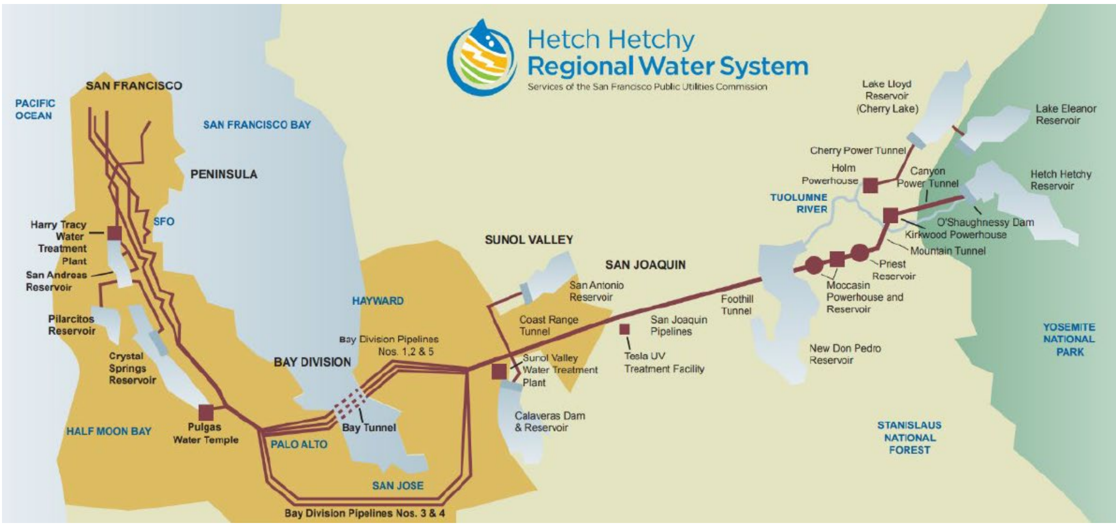


Figure B-11: Hetch Hetchy Regional Water System

The water is taken from the Hetch Hetchy Reservoir to the Bay Area, and transported over 167 miles through hydroelectric powerhouses, treatment facilities, tunnels, and pipelines. The SFPUC WE also manage the CCSF's water distribution system through its SFPUC City Distribution Division. This water distribution system serves both the low-pressure fire protection system and the domestic drinking water system. Within the study area water mains vary in age (1863-2016), and generally are cast iron or ductile iron, with some steel. There is a network of low-pressure fire hydrants and isolation valves present, and one critical automatic air valve on Sansome Street near the Transamerica Building. The only transmission line is to and from the Bay Bridge Pump Station, on the corner of Bryant Street and Main Street, which serves as the sole source of water for the Treasure Island and Yerba Buena water distribution system (CH2M/Arcadis Team 2020e).

B-2.2.5 Auxiliary Water Supply System (AWSS)

During the San Francisco Earthquake of 1906, the domestic water system was severely damaged, and there was insufficient water available to fight the fires. In response to this, City leaders determined that an independent fire protection system was needed in case the municipal water supply failed again. The AWSS was created to provide the San Francisco Fire Department (SFFD) with a high-pressure fire suppression water system. The AWSS is vital for protecting against loss of life, homes, or businesses in the event of a major earthquake or disaster, by providing an additional layer of fire protection beyond that of the LPW system. In 2011, the management of the AWSS was transferred from SFFD to SFPUC WE.

The AWSS is an important part of the firefighting defense and the backbone for fire protection in San Francisco, which operates in parallel with the LPW system. The system is highly redundant, being able to draw from Twin Peaks reservoir located within the city limits of San Francisco but west of the study area, Pump Stations No. 1 and 2 (bay water), cisterns, manifolds, and Portable Water Supply System (PWSS), all within the study area, with the use of fireboats and trucks.

Within the study area, the AWSS distribution lines generally end prior to the Embarcadero. The lines connect to a series of high-pressure fire hydrants. Fireboats and pump trucks can connect to manifolds and drafting points to deliver saltwater to the AWSS or provide water to fight fires. Two underground cisterns are on the edge of the program area. While Pump Station No. 1 is outside of the program area, it is important for the AWSS systemwide function as it pumps saltwater from the Bay to serve as a backup to the primary water source of the AWSS (series of reservoirs). Pump Station No.1 depends on a seismically retrofitted 5-foot-diameter reinforced concrete intake tunnel near Townsend close to Pier 38 within the program area. The PWSS provides an additional layer of redundancy. The fireboats Phoenix and Guardian are berthed at Firehouse 35 at Pier 22-½ (CH2M/Arcadis Team 2020e).

B-2.2.6 Natural Gas

Within the program area, the natural gas system is owned and managed by PG&E. Although Kinder Morgan is a major natural gas distributor in North America, Kinder Morgan confirmed it has no pipelines in the study area or San Francisco. There are no transmission lines within the program area, but there are over 7 miles of buried distribution pipelines and 111 manually operated gas valves within 500 feet of the seawall. PG&E has already retrofitted all gas distribution pipelines within the study area. Service can be isolated based on 14 isolation zones that intersect the program area (within 500 feet).

Natural gas is extracted and brought to storage facilities and compressor stations for processing. The natural gas is then distributed through an infrastructure system that includes transmission lines, regulator stations, and distribution lines. Transmission lines transfer natural gas from compressor stations to regulator stations. These lines are larger and operate at higher pressure. Regulator stations reduce the pressure of natural gas coming from the transmission lines and feed it into distribution lines. Distribution lines then carry natural gas from the regulators to the residential and business districts. Smaller pipes connecting the main distribution lines to customers are often referred to as service lines or “service laterals.” Within the study area, the natural gas system includes distribution lines, service lines, and emergency shutoff valves (CH2M/Arcadis Team 2020e).

B-2.2.7 Electric Power

The two primary organizations that provide power in San Francisco are PG&E (private) and the SFPUC (public). PG&E owns most of San Francisco’s power infrastructure. Its electric system is designed and built to deliver power to Northern and Central California, and it services commercial and residential customers. PG&E produces or buys its power from a mix of conventional and renewable generating sources. PG&E-owned generating plants make electricity by hydropower, gas-fired steam, and nuclear energy, and they buy electricity from over 400 independently owned plants and out-of-state producers. The electrical system consists of generators, transmission lines, substations, distribution lines, and service laterals also known as the energy grid.

The energy grid is composed of three principal stages: electricity production, transmission, and distribution. After production, electricity is transmitted by high-voltage power lines that connect power generation plants to substations. The primary power lines servicing San Francisco include the Transbay Cable, the transmission lines from Newark and the lines along the Peninsula.

Substations are important facilities in the energy grid network because they connect the transmission and distribution systems. Substations use transformers to lower the electricity voltage to a level acceptable for the distribution system. Some of the substations near the program area that service San Francisco include Embarcadero Substation and Potrero Substation.

The distribution system links electricity from the substations to SFPUC and PG&E customers. It includes high-voltage primary lines connected to lower-voltage secondary lines, and the distribution transformers that connect the two, lowering voltage from primary to secondary. In addition, the system employs switching equipment to allow the distribution lines to connect to one another, forming a variety of combinations and patterns according to need.

PG&E manages and balances the amount of electricity in the grid to maximize its system's efficiency and demand. It is transforming the management into a Smart Grid system, which will provide PG&E with real-time information to manage the system more efficiently (CH2M/Arcadis Team 2020e).

B-2.2.8 Telecommunications

Telecommunications is the sending of information, such as signals, messages, sounds, and images by wire, radio, fiber-optic, or other electromagnetic systems. Telecommunications companies provide voice or data transmission services via underground cables, overhead wires, or radio waves through cell sites. The individual components of each company's system can vary significantly, depending on available technologies, customer base, and region. This system is supported through a complex network of assets and services having hubs as the central distribution center.

There are multiple organizations that provide telecommunication systems in San Francisco: SFDT (public) and several private telecommunication companies, including, but is not limited to, AT&T, Verizon, Comcast, Sprint, CenturyLink, and XO Communications. These organizations own their own conduit space, share and lease conduit space from each other, or share and lease from PG&E.

SFDT provides telecommunication services to CCSF departments and agencies, but its critical function is public safety and emergency communications. The SFDT public safety network includes safety sirens, safety radio facilities, fire alarm pull stations/fire boxes, and underground conduits. SFDT maintains multiple two-way radio networks for first responders to coordinate life safety operations.

PG&E owns a telecommunications network throughout its service territory. This network supports the electric grid and gas system monitoring, supervisory control, and data acquisition (SCADA), and remote substation management (CH2M/Arcadis Team 2020e).

B-2.3 Mobility Systems

B-2.3.1 General

The seawall itself is the physical edge of downtown San Francisco. As such, it is the city's interface with the Bay, and the landing point from around the region. The CCSF and POSF have made significant investments to create a publicly inviting, pedestrian-oriented waterfront that draws 15-20 million visitors annually. CCSF investments in the 1990s to replace the Embarcadero Freeway with an urban boulevard, public transit, and pedestrian promenade have been reinforced with significant additional pedestrian and water transportation investments along the Embarcadero. These systems include throughways and connections for essential regional and state assets such as expanding regional ferry service, the Bay Trail that rings the Bay with contiguous cycling and pedestrian access, the region's major rail crossing that serves the East Bay and connections beyond, as well as the Embarcadero and other roadways serving business and recreational visitors connecting to various modes or accessing nearby Bay Bridge on-ramps.

B-2.3.2 Shared Road Network

Public Works and the San Francisco Mass Transit Authority (SFMTA) are the primary CCSF agencies responsible for the road network throughout San Francisco. The two agencies have different but shared responsibilities for public road asset planning, maintenance, and signage. The POSF works closely with Public Works and SFMTA with respect to the roads and sidewalks under POSF jurisdiction.

The shared road network includes the roadway and parking facilities for personal vehicles, taxis, and transportation network companies (e.g., Uber and Lyft), trucks, motorcycles, motorized and nonmotorized scooters, bicycles and other human-powered or electric-assist modes of transportation, and pedestrians. Public transit is also an important component of the shared road network and is addressed in more depth in the following sections. The Embarcadero bike and pedestrian infrastructure along the Embarcadero is used heavily by local workers, residents, and visitors, but is also integral to maintaining a continuous San Francisco Bay Trail, connecting cyclists and pedestrians throughout the Bay Area (CH2M/Arcadis Team 2020e).

B-2.3.3 Local Transit

SFMTA is a city agency that operates bus and rail lines with an annual budget of roughly $1 billion. It operates and maintains the following transportation services: light rail (subway and surface), historic rail, cable car, diesel bus (30-, 40-, and 60-foot articulated buses), and electric trolley bus. SFMTA is also responsible for paratransit operations (contracted to a third party) and taxicabs (private industry oversight). This mix of services represents the most diverse mix of modes and vehicles operated by a single agency in North America. SFMTA is an enterprise agency with dedicated funding sources and semi-autonomous oversight. The SFMTA Board of Directors is appointed by the mayor and confirmed by the Board of Supervisors.

The San Francisco Municipal Railway (Muni) system has evolved since the mid-1800s. Not just a commuter system, Muni connects with the city's many neighborhoods, as well as cultural, sporting, shopping, and entertainment venues. Today, the fleet features electric, hybrid and biodiesel-powered buses, electric light rail, historic streetcars, and iconic cable cars. Muni is recognized as one of the greenest transit fleets in the world.

Bay Area Rapid Transit (BART) also serves as an important provider of intracity transit along the Market and Mission Street corridors. Between Embarcadero and Civic Center stations, BART service is paralleled by Muni bus and subway, but BART capacity is essential to meeting demand along Market Street and in the entire corridor (CH2M/Arcadis Team 2020e).

B-2.3.4 Regional Transit

This section describes the regional transit providers, specifically those routes that provide service into and out of the city. Peak-hour regional commute trips to downtown San Francisco rely heavily on public transit.

POSF property plays a key role in the infrastructure for BART, Golden Gate Ferry, and the Water Emergency Transportation Authority (WETA) San Francisco Bay Ferry. Regional buses are important links for commuters, connecting the city via the Bay Bridge (AC Transit and Capital Corridor), the Golden Gate Bridge (Golden Gate Transit), and from the peninsula, primarily via U.S. Route 101 (SamTrans). An estimated 290,000 trips per day pass through the BART Transbay Tube and Embarcadero Station. The BART system connects San Francisco directly to the East Bay and the Peninsula, reaching as far south as Millbrae Station.

San Francisco's waterfront is a primary connection point for most of the 15,000 daily ferry commuters arriving on Golden Gate Ferry and WETA's San Francisco Bay Ferry.

Regional buses are also important links for commuters, connecting the city via the Bay Bridge (AC Transit and Capital Corridor), the Golden Gate Bridge (Golden Gate Transit), and from the Peninsula, primarily via U.S. Route 101 (SamTrans). AC Transit's connections bring more workers into the city than any other agency, averaging more than 14,500 daily trips to downtown in 2019. AC Transit estimates that demand is even greater, and ridership is limited by capacity. The remaining bus operators provide important geographical links but have significantly less ridership. Of the regional bus routes, only Golden Gate Transit has stops within the study area (Fisherman's Wharf area). Other regional buses do not pass through the program area, except for AC Transit buses passing over the Bay Bridge (CH2M/Arcadis Team 2020e).

Section B-3. Engineering Evaluation

B-3.1 General

Model areas (reaches) were needed for Economics and Cost Engineering to develop benefit cost ratios along the study area. Four reaches were developed for the G2CRM evaluation. Topography and interior drainage were the driving factors in establishing the reach boundaries. **Figure B-12** illustrates the four reaches with the boundaries of Reach 1 shown in yellow, Reach 2 in blue, Reach 3 in red, and Reach 4 in green. Notation is made in the feasibility text for the NWF and the SWF. The NWF consists of Reaches 1, 2, and part of 3 (north of Mission Creek). The SWF consists of Reach 3 (south of Mission Creek) and Reach 4.

- Reach 1: Includes Aquatic Park, Fisherman’s Wharf and Piers 35 to 31.
- Reach 2: Includes the Embarcadero, Financial District and Rincon Point.
- Reach 3: Includes South Beach. China Basin, Mission Bay, and Dogpatch.
- Reach 4: Includes Central Waterfront, Islais Creek, and India Basin.

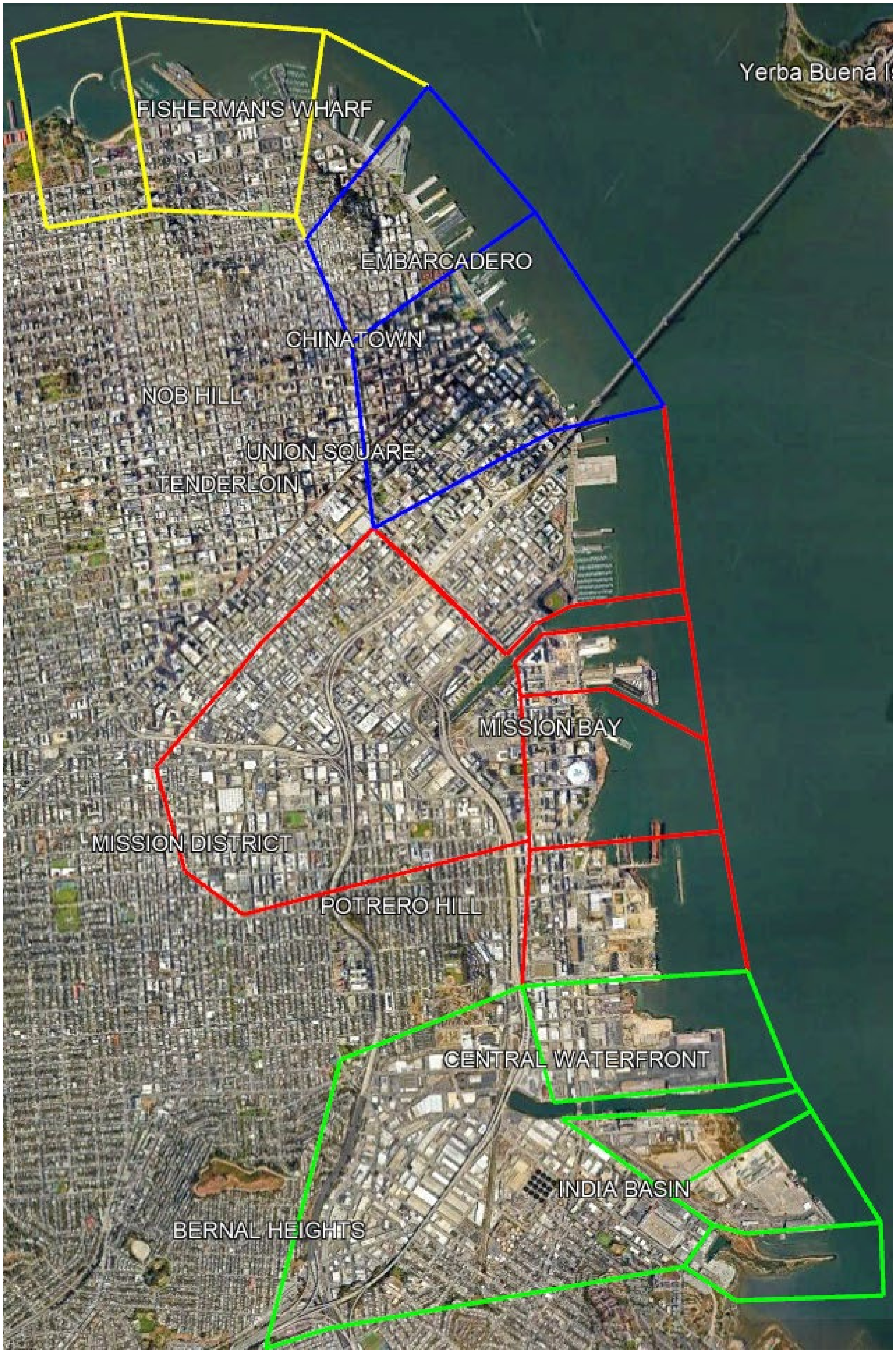


Figure B-12: Map of Four Reaches

B-3.2 Project Alignment

Five structural alternatives were developed consisting of lines of protection being both along and inland of the coastline of the study area. Along the NWF and around Mission Creek, the alignments of each alternative generally follow a similar path due to the constraints of existing roadways and structures, which are iconic and historic, and the overall viewshed which dictate the objective to minimize the construction footprints and disturbance of these areas. The alignment of three of the five alternatives approximately align with the existing seawall while the other two are located approximately 25 feet and 50 feet bayward of the existing seawall. The alignments along the SWF vary from approximately following the existing shoreline to being setback several hundred to several thousand feet from the existing shoreline depending on the assumptions of the alternative relative to future land use, development, and environmental benefits. Plan view representations that show the approximate footprint and measure type of the structural and nonstructural alternatives can be found in Sub-Appendix B.4.

Based on the comprehensive benefits approach, the Total Net Benefits Plan (TNBP) was developed by selecting various components from the initial alternatives which were developed and evaluated for the purpose of maximizing overall comprehensive benefits to the project. The TNBP was selected to include the first actions of Alternative B in Reach 1, Alternative G in Reach 2 and Alternative D in Reaches 3 and 4. These alternatives are described in detail in the Integrated Feasibility Report and Environmental Impact Statement. A plan view representation that shows the approximate footprint and measure type of the TNBP can be found in Sub-Appendix B.5.

The real estate requirements, both temporary during construction and permanent after construction, were considered and estimated when developing the project alignments. For earthen levees, 20 feet beyond the estimated final width was included for design tolerances when considering the need for temporary access during construction.
For structural walls, the temporary construction footprint was assumed to be 25 feet wide overall. Ideally, permanent easements should include a minimum of 20 feet beyond the final footprints for access to the flood control elements for maintenance and to maintain the vegetative free zones; however, given the highly developed nature of the project extent, this extent may only be possible in limited areas. In some instances, both temporary and permanent footprints would need to be the equivalent due to real estate issues and constraints.

B-3.2.1 Optimization Changes

As the feasibility study continues to further evaluate and refine the TNBP, it is anticipated further refinements to various components in the current design will be developed for the purpose of optimizing costs and benefits for the project which will be beneficial to the Federal Government, the local sponsor, and the citizens of San Francisco.

B-3.3 Engineering Aspects of Study Measures

The preliminary designs of individual protection measures are predominately based on engineering judgement and on available designs utilized on previous USACE Coastal Flood Risk Management (CFRM) projects and studies conducted by the local sponsor; however, it is noted that much of the engineering effort to date was focused on refining selection and sizing of appropriate protection measures (levee or wall) along the project length which considered existing topography, infrastructure (structures, roadways, and utilities), viewshed, and likelihood of local acceptance. For additional information about Natural and Nature Based Features (NNBFs) considered during plan formulation refer to *Appendix I: Engineering With Nature*. Discussions are included on future work required during the Pre-construction Engineering and Design (PED) phase. The geotechnical and structural aspects of the various feasibility study measures are discussed below.

B-3.3.1 Levees

Levees have a trapezoidal cross-section with a wide base that slopes up to a narrower crest over a height designed to raise the shoreline defenses against flooding risks. Levees would be constructed of imported impervious earthen fill material, assumed to be procured from a commercial source near San Francisco. The side slopes of the levees developed for this study have side slopes no steeper than 3H:1V, but isolated areas will include side slopes ranging from 5H-10H:1V for vehicle passage in parking lots and on piers to 12H:1V to accommodate Americans with Disabilities Act (ADA) accessibility requirements.
The levees would be constructed using heavy machinery (i.e., bull dozers, track hoes, roller compactors, etc.). The required levee height and width is dependent on local existing elevations, the final flood side design side slope and configuration, and the crown width which was assumed to vary between 10 to 20 feet for this feasibility study. During PED, all levees should be designed in general accordance with Engineering Manual (EM) 1110-2-1913 *Design and Construction of Levees* (USACE 2000). A typical levee is shown on **Figure B-13**.

Table B-2: Levee Footprint Requirements

Levee Height (feet)	10-Foot Crest	20-Foot Crest
	3H:1V	3H:1V
	Total Width (feet)	Total Width (feet)
1	16	26
2	22	32
3	28	38
4	34	44
5	40	50
6	46	56
7	52	62

Levee Height (feet)	10-Foot Crest	20-Foot Crest
	3H:1V	3H:1V
	Total Width (feet)	Total Width (feet)
8	58	68
9	64	74
10	70	80

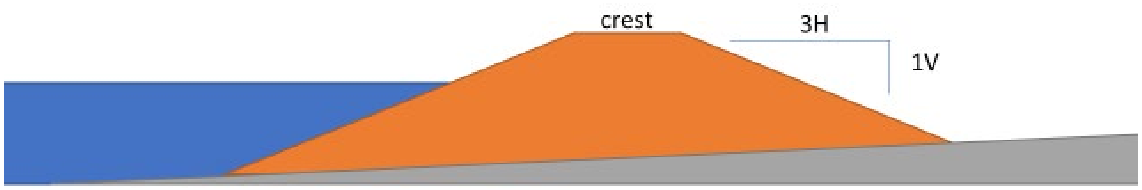


Figure B-13: Typical Cross Section of Levee

The foundation material along many areas where levees might be utilized is AF placed between the late 1800s and the 1950s as portions of the Bay were infilled for development. The nature of this fill is known to vary in composition from earth fill consisting of dune sands, dredged bay clay/silt, mud waves of existing soft bay muds, and quarry waste sand/rock to manmade debris consisting of sunken boats, construction debris, demolished structures from the 1906 earthquake/fire, and general landfill rubbish. Future PED geotechnical site investigations will need to attempt to better identify subsurface conditions in these AF areas which are subject to change significantly within short distances due to the nature of AF subsurface conditions. Conservatively, the team assumed these fill materials could be somewhat permeable and included seepage barriers beneath all earthen levees greater than 4 feet high, even in areas outside of known AF areas. The 4-foot levee height criteria were developed considering the assumed 2-foot design wave height and the judgement that 2 feet or less of still water height would be unlikely to create a seepage issue. The seepage barrier was assumed to be a 20 feet deep vinyl sheet pile, extending several feet into the levee section, and driven along the length of the levees for the purpose of minimizing underseepage potential in the potentially highly variable soils during flood conditions.

B-3.3.2 Ecotone Levees

Ecotone levees are typically constructed with the flood side slopes gently sloped towards a tidal marsh. They connect the levee to the marsh surface and can provide high quality transition zone habitat when vegetated with appropriate native plants. Ecotone levees would be constructed of commercially sourced imported impervious earthen fill material, assumed to be procured near San Francisco. The topsoil would need to be specified based on the vegetation type anticipated to cover the slope. The slopes of ecotone levees typically vary between 10H:1V to 30H:1V (**Figure B-14**). In the isolated areas where real estate is not restrictive, levees (ecotone levees) were included in the selected plan with these typical dimensions of flood side slopes and vegetation. Future designs will need to reconcile the fact the water surfaces in this project are driven by SLC and as such the assumed marsh conditions will not exist throughout decades of this project while the anticipated sea levels changes occur at a relatively slow rate. This will likely impact the ability to grow and maintain appropriate marsh-type vegetation on the flattened flood side slopes. Where an ecotone levee has been proposed, the vinyl sheet pile was not included because the overall footprint at the levee base is deemed to be sufficiently wide to limit water from seeping through the levee toe (landward side). Further information about ecotone levees and other NNBFs considered during plan formulation can be found in *Appendix I: Engineering With Nature*.

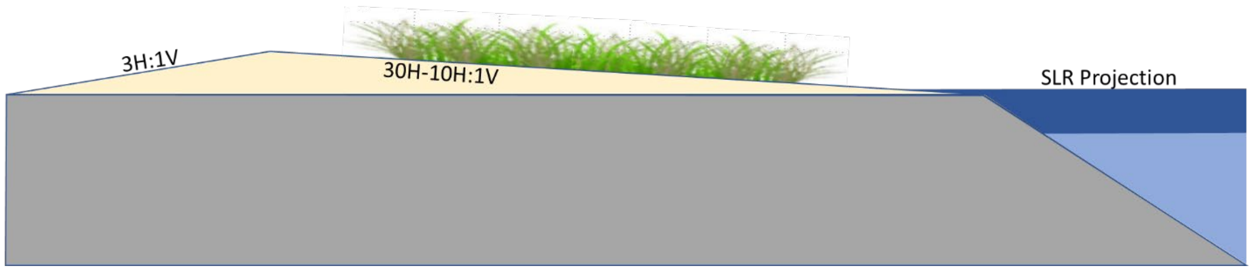


Figure B-14: Typical Cross Section of Ecotone Levee

Table B-3: Ecotone Levee Footprint Requirements

Ecotone Levee Height (feet)	Slope		
	10H:1V	20H:1V	30H:1V
	Total Width (feet)	Total Width (feet)	Total Width (feet)
1	13	23	33
2	26	46	66
3	39	69	99
4	52	92	132
5	65	115	165
6	78	138	198
7	91	161	231
8	104	184	264

B-3.3.3 Water Management Structure

Alternative F considered the potential to build water management structures across both Mission and Islais Creeks. Initially, the water management structures would consist of a passive gate system (sector or miter gates) with the structure modified in the future to be converted to a pump station as SLC makes the passive operation unmanageable. Multiple advantages would be provided with these structures in place to include: (1)

elimination of several thousand feet of flood protection along the creek banks, (2) eliminate the need to raise or replace the bridges that cross the creeks, (3) eliminates modifications to the existing overflow interior drainage/sewer system that currently discharges into the creeks, (4) eliminates the need for additional pump stations for interior drainage which are required for all other alternatives, (5) act as a storage retention basin for high water events, (6) maintain long term connectivity of bay and creek ecosystems, and (7) is adaptable as SLC dictates future bay water levels. The adaptability component of the water management structures is they would initially be constructed as passive gates with foundations and configuration such that in the future the gates can be removed, and the structures converted to a pump station.

The intent for the operation of the passive gate system is the gates would be left open to allow tidal exchange of the creeks with the bay tides. In PED, the number and/or size of the gates would be determined to maintain similar inflow/outflow volumes in the creeks. The open gates posture would persist until bay water levels and storm forecasts indicate closure of the gates (for a few hours, maybe a day) is warranted to avoid flooding the areas around the creek banks. The gate closing would be done at low tide to maximize storage in the creeks for inland storm runoff.

The adaption phase would be mandated as SLC increases the low tide water levels such that adequate lowering of the protected side water levels behind the gates is no longer achievable to provide functionality of the inland drainage system. Current H&H evaluations indicate the Mission Creek Pump Station and Islais Creek Pump Station would require pumping capacities of 600 cubic feet per second (cfs) and 1,200 cfs, respectively. Once the water management structures are converted to pump stations the operations would likely consist of intermittent pumping (predominantly associated with heavy precipitation events) of the protected creek areas into the Bay to maintain acceptable water levels to allow functioning of the overflow interior drainage system.

The water management structures are assumed to be supported on deep foundations with the base of the structures at approximately elevation –25 to –30 feet based on existing creek bottom elevations. Underseepage mitigation measures under the structures will be provided as determined in the PED phase. The structures will be approximately 600 feet in length to extend from bank to bank. The ends of the structures will be designed to tie into the adjoining flood protection elements which currently consist of earthen levees.

B-3.3.4 Curb Walls

There are lengths of the reaches that comprise existing bulkhead piers or wharves which are currently at elevations of approximately 11 to 12 feet. The most cost-effective way to raise these structures by up to about 2.5 feet for flood protection is a curb wall. Gate structures were also included with these measures to allow access by vehicles or pedestrians during non-flood periods. Depending on actual SLC over the project life, this flood protection measure could provide the necessary level of protection, or it could be a temporary measure extending the life of the existing infrastructure before more extensive measures (raising or abandoning) are required.

As envisioned for this feasibility study, the curb wall is a small concrete structure that is fixed to a concrete deck or capping beam along the perimeter of the main structure. The curb wall could consist of either cast-in-place concrete or precast concrete elements which are attached mechanically, both of which would include some type of water stop integrated to ensure an acceptable seal against seepage. The design of the curb wall would ensure it is anchored to the deck to prevent overturning or sliding when loaded. Future design evaluations could consider the use of a glass or clear polymer panel to provide similar functionality; however, only concrete walls were included in the study. A typical curb wall is shown on **Figure B-15**.

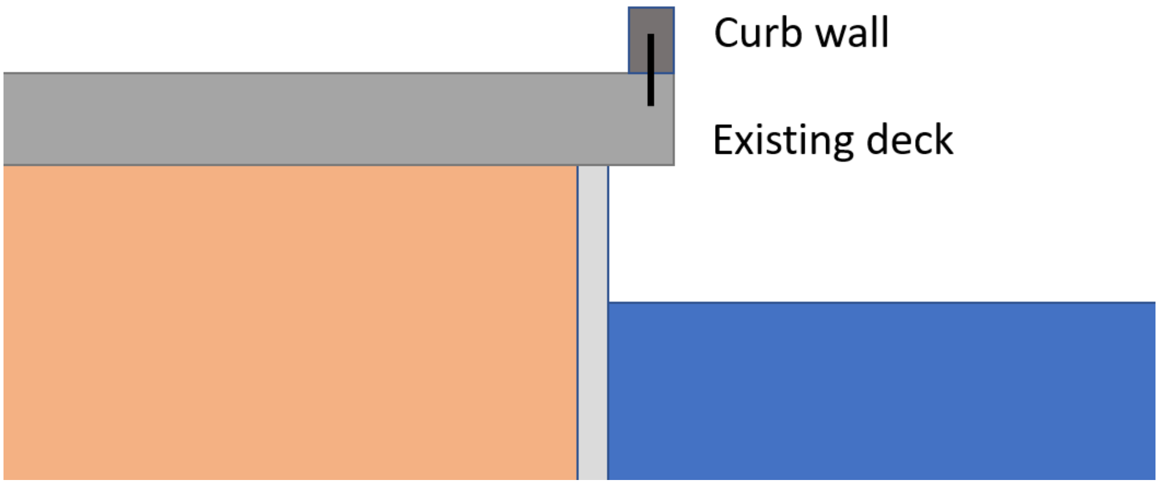


Figure B-15: Example of Curb Wall

B-3.3.5 I-Walls (Cantilever sheet pile wall)

Where the chosen flood protection measure needed to be a structural wall due to real estate constraints and the required flood protection height was about 4 feet or less, an I-wall (cantilever sheet pile wall) was typically utilized for the protection measure. The I-wall consists of a steel sheet pile section (typically a PZ- or NZ-type section) for structural lateral stability which is cast into a reinforced concrete capping wall for corrosion and impact protection. A concrete splash pad is also required on the protected side of the I-wall to protect against scour from any potential overtopping during flood events. Based on previous design experience, the stability design assumed the depth of embedment of the steel sheet pile section to be a minimum of 2 times the height of wall above the ground surface, with a minimum sheet pile length of 10 feet for wall stick up heights less than 3 feet. I-walls should be designed in accordance with EM 1110-2-2502 *Retaining and Flood Walls* (USACE 1989).

B-3.3.6 T-Walls and L-Walls

Where the chosen flood protection measure needed to be a structural wall due to real estate constraints and the required flood protection height was about 4 feet or greater, either a concrete T-wall or L-wall was utilized for the protection measure. Typical examples of these type walls are shown on **Figure B-16**. At this design level, the T-walls and L-walls are assumed to be founded on deep foundations consisting of either steel H-piles or pipe piles which could be driven either vertically or battered depending on the design loads and underground site constraints. A steel sheet pile cutoff, extending 20-30 feet vertically beneath the wall base, will be installed to reduce underseepage and uplift pressures on the structure. The pile foundations are assumed to consist of two rows of piles which are 50 feet in length and placed on 5-foot centers along the wall length. These design estimates will be refined in the PED phase. T-walls and L-walls should be designed in accordance with EM 1110-2-2502 *Retaining and Flood Walls* (USACE 1989).

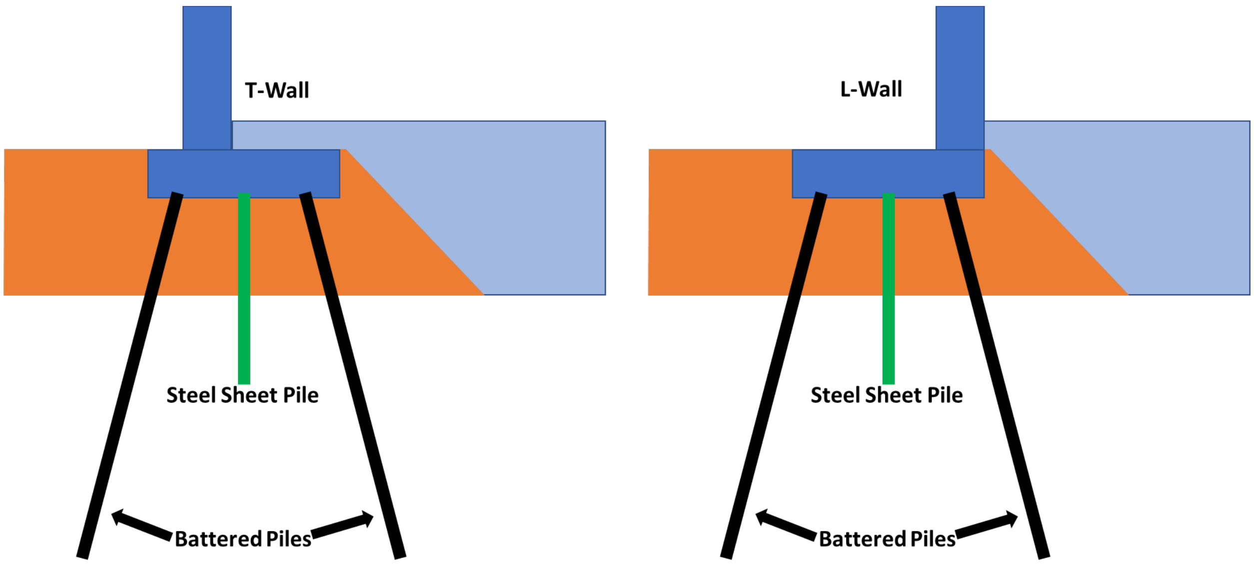


Figure B-16: L and T-Walls

B-3.3.7 Combi Wall

In the NWF, there were two alignments considered which constructed the line of protection further out into the Bay. These were evaluated to include alternatives which limited impacts to the Embarcadero promenade and roadway but increased impacts to the Bay and wharf/pier structures. These alternatives considered moving the line of protection either 25 feet or 50 feet bayward. At these offsets, the cantilever height of the wall above the bottom of the Bay will be approximately 25 feet to 40 feet high.

To accommodate construction of a wall in the Bay, without using a cofferdam and dewatering means and methods, it was proposed to utilize a cantilever wall (Combi wall) consisting of driven, large (66-inch diameter) circular, prestressed concrete pipe piles driven approximately 8-12 inches apart, which essentially form a tangent wall. The overall lengths of these circular piles were estimated to be 120 feet, which provides an embedment depth of 2 times the stick-up height (40 feet of stick-up and 80 feet embedment). After driving, the soils within the circular piles will be removed and the piles tremie filled with concrete and reinforced as necessary to resist the design lateral loads. Two 18-inch square prestress concrete piles will be driven on a diagonal adjacent to the circular piles and grout will be tremie placed into the space between the circular and square piles within the water depths to mitigate potential movement of either water or soils through the wall. Once the wall is constructed the zone between the new wall and the existing line of protection will be backfilled in the wet with granular material which will support the reconstructed wharf slab and existing wharf structure. It was assumed wall segments would be constructed at 500-foot intervals extending perpendicularly from the main wall alignment to the shoreline to help accommodate and contain backfilling of the wall in manageable lengths.

Two phases of ground improvements consisting of DSM will be required during the construction process. The first phase of DSM will be constructed within the existing YBM stratum prior to backfill being placed to stabilize these soils and minimize consolidation settlements due to the weight of the backfill soils. The second phase of DSM will be required to densify and strengthen the backfill soils to ensure they can support the new wharf slab and structure with minimal settlements.

This wall concept was adapted from the Surge Barrier structure constructed in New Orleans as a part of the CFRM protection constructed after Hurricane Katrina. These design estimates will be refined in the PED phase. A general concept of this measure is shown on **Figure B-17**.

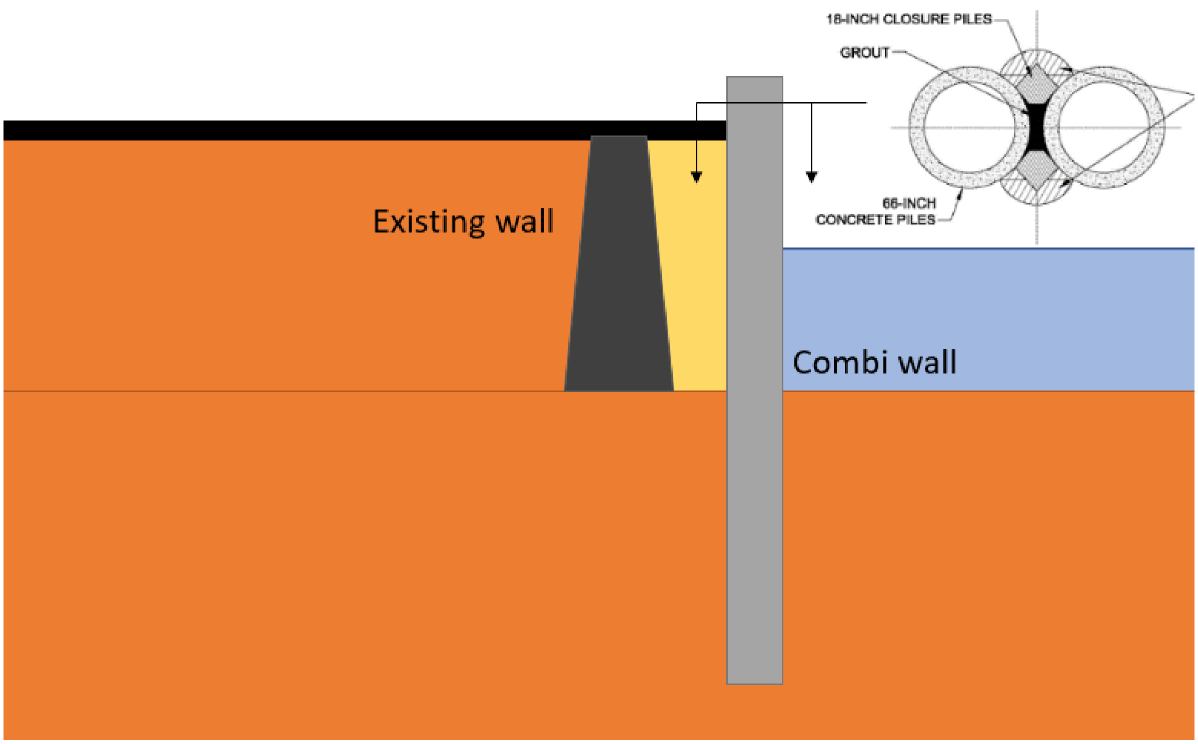


Figure B-17: Example of Combi-Wall

B-3.4 Land Gates

Gates are required to complete the line of flood protection in the event of a flood to provide for the safety of the public and infrastructure from inundation of flood waters. In non-flood conditions the gates remain open to allow passage of vehicles and/or pedestrians through the flood protection without the need for ramps which require additional real estate and fill material. In flood conditions the gates are closed to provide flood protection and to restrict passage of pedestrians and vehicles.

B-3.4.1 Vehicle Gates

A vehicular gate is required at opening where the new flood protection measure crosses an existing roadway or where access is required to the piers. The gates will be open most of the time and will only be closed when water levels are forecast to potentially inundate protected areas for coastal storm events. Vehicle gates that accommodate pedestrian passage will have to be ADA compliant. Types of vehicle gates include swing gates, lift hinge gates, roller (slide) gates, and pivot flood gates. For this study, roller gates are the preferred option since they deploy relatively easily and without concern for potential clearance of nearby obstacles, with swing gates potentially used where protection heights and gate openings are relatively small.

B-3.4.1.1 Swing Gates

Swing gates are relatively easy to deploy and operate; however, the main drawback to swing gates are the large clearances required to be able to close them. The gates are typically structural steel elements with a steel plate skin which are attached to a reinforced column of a flood wall with hinges on one side. To close, the gate is swung around on the hinges and into place, then secured to the wall on the opposite side of the opening. Two swing gates are used for larger openings with center post reinforcing as necessary. Compressible seals along the bottom and sides of the gate provide a seal to resist water infiltration. Typically swing gates do not require powered equipment to open/close and can be set manually with enough people. Depending on the size, heavy lifting equipment may be needed for opening/closing and placing the additional, removable supports required. An example of swing flood gates is shown on **Figure B-18**.

Source: courtesy of Flood Control International

Figure B-18: Typical Swing Flood Gate)

B-3.4.1.2 Roller Gates

Roller gates (also known as slide gates) are also simple and relatively easy to operate. Roller gates are also constructed using structural steel members with plate steel skins. The gates are deployed using rollers attached to the gate which roll across a track along the gate sill. Roller gates for this project are proposed to be constructed as one gate section. When closed, compressible seals along the bottom and sides of the gate seal against the wall and sill to resist water infiltration. Depending on the height and width, additional bracing can be placed on the dry side of the gate to help it withstand the pressure of the water. An advantage of roller gates is they do require the same clearance as swing gates. Roller gates can have the option for a manual cranking system to close depending on size, but many times a motorized opening/closing device is required. **Figure B-19** shows a typical sliding gate in the open position.

Source: courtesy of Flood Control International

Figure B-19: Typical Roller Gate

B-3.4.2 Gate Maintenance

Annual maintenance of the various flood gates is recommended to ensure the gates are in proper working order and that personnel responsible for deployment of the gates are familiar with the operation procedure and are knowledgeable of the location of any external components required to properly deploy the gates. Maintenance of the gates should focus to ensure the seals are in acceptable condition and should be replaced as needed. Other components such as rollers and tracks should be inspected and cleaned and lubricated as necessary. For gates deployed using motors, the mechanical and electrical components should be operated annually, and normal maintenance and lubrication performed. It is anticipated the structural component of the gates will last for the life of the project; however, periodic inspections should also include visual inspections for fractures of welds and connections as well as inspection of all structural steel part of the gates for potential corrosion due to the coastal marine environment.

A draft operations and maintenance (O&M) manual has been started during this feasibility phase. This draft O&M manual provides a preliminary operations process, maintenance, and inspection timeline, etc. This draft plan will be further developed during the PED phase as more detailed information is developed about each individual gate during the design portion.

B-3.5 Storm Surge Gates

The storm surge gates are elements associated with the water management structures which would be located across Mission and Islais Creeks. Storm surge gates are generally larger scale flood protection measures placed across major inlets with the intent of restricting water surges from reaching large areas of the shorelines. By restricting surging water at the inlet, large quantities of shoreline protection can be eliminated.
In normal (non-flood) conditions, open storm surge gates allow the free passage of water, enabling regular navigation and natural water exchange in tidal inlets. For flood conditions sufficient to inundate protected areas behind the barrier, these gates can be closed against storm surges or high tides (including sea level rise) to prevent flooding. Two types of flood barrier gates are commonly used which consist of sector gates or miter gates. For this project it was assumed that sector gates would be utilized.

B-3.5.1 Sector Gates

A sector gate is a pie-slice shaped structure in plan that rotates about its point to close a water inlet (river, creek, bay, etc.) off from rising flood waters. The gates typically consist of a pair of gates that, when closed, meet in the center of the gate opening. These are robust mechanically operated structures that are very effective at holding water loads under dynamic conditions (i.e., storm events).

Sector gates protect New Orleans as part of the Hurricane Storm and Damage Risk Reduction System that were enhanced by USACE after Hurricane Katrina. These navigable sector gates allow marine vessels to traffic through the levees and floodwalls and are shown on **Figure B-20**.

Figure B-20: Bayou Bienvenue and Bayou Dupre Sector Gates

B-3.5.2 Miter Gates

Miter gates are historically very popular on canals due to their basic hydraulic functionality. A pair of closed miter gates meet at an angle like a chevron pointing upstream and only a very small difference in water-level is necessary to squeeze the closed gates securely together. A disadvantage with miter gates is if the head is reversed then the gates are pushed open. An example of a miter gate is provided on **Figure B-21**.

Source: courtesy of Jacobs Engineering Group

Figure B-21: Queensborough Barrier, UK at Low Tide

B-3.6 Storm Drainage Check Valves and Gates

As discussed in Section B-2.2.2, the storm water drainage system for the area of San Francisco considered in this project is combined with the sewer system. The combined water is treated and then discharged into the Bay via short outfalls, largely by gravity flow. In some area pumps are required to assist the flow, as discussed in Section B-5.

If no action is taken, the expected increase in sea level in the Bay over the next few decades will reverse the hydraulic head, thus allowing flow of sea water into populated areas through the storm water system. This can be prevented by closing the storm water drains using valves and gates in a high-water level event. The method of preventing backflow within the system at specific locations will be further refined during PED.

B-3.6.1 Check Valves

Check valves restrict the flow of water to one direction, so they will allow stormwater to flow out through the system normally but will close and cut off all flow once water levels on the outlet side reach a point that it would flow to the protected side of protection. The valves open automatically to allow water to flow out from the protected side of protection once the water levels on the outlet are low enough.

B-3.6.2 Flap Gates

A flap gate on stormwater outlets is a method of automatically closing the outlet when a reverse hydraulic head occurs. This allows storm water to free flow from a pipe structure while preventing the high tides from the Bay to backflow into the storm drainage system causing flooding in protected areas. Flap gates are beneficial when water stages fluctuate with a few feet above and below a stage that would require gates to be open and closed frequently over a prolonged period. A basic example is shown on **Figure B-22**.

Source: courtesy of USACE Norfolk District

Figure B-22: Flap Gate

B-3.6.3 Sluice Gates

Sluice gates are used in a variety of water control applications, including flood control, all over the world, and are relatively low maintenance due to their simple design; however, they require some level of manual or mechanized operation. They are available in a variety of sizes and can be placed side by side to maximize the flow when open and minimize negative effects like flow restriction or scouring. As shown on **Figure B-23**, sluice gates function as a simple metal gate that can be raised and lowered on a vertical track to seal an opening in the protection. The sluice gates can be placed in areas where a tidal creek or marsh freely flows in and out during normal tide cycles and closed during times of high-water levels in the Bay.

Source: courtesy of USACE Norfolk District

Figure B-23: Pair of Sluice Gates

B-3.7 Gate Closure Procedure

Gate closure procedure will be finalized during PED phase and dictated in the O&M Plan; a draft version is currently being written. Typically, all gates will remain open and will only be closed when required due to coastal storm flooding events. For the land gates, the operational procedures will need to consider the time needed to close gates in reaction to water level to address overall operation and evacuation needs. This may result in different thresholds in the different areas of the city.

Section B-4. Pump Stations

The city’s wastewater collection system is designed to maximize gravity flow based on the topography of the area. In cases where the collection system cannot drain by gravity flow, pumps and force mains are used to move water throughout the system.
The San Francisco combined storm sewer system currently has 26 pump stations throughout the city. Of the 26 pump stations, 12 of the pump stations are located adjacent to the San Francisco Bay and are inside the city’s Sea Level Rise Vulnerability Zone. The Sea Level Rise Vulnerability and Consequences Assessment (CCSF 2020) was completed in 2020 and identifies portions of the wastewater system that would be compromised due to changes in San Francisco Bay tidal conditions and potential damage that may occur as sea level rise occurs in the future.

The 12 pump stations located within the study area are shown on **Figure B-24**, which indicates the pumping capacity and whether the pump is used in wet and dry operations. Six of the 12 are active during dry weather operations and all 12 are active during wet weather operations. During dry weather operations the pump stations are primarily used to move water collected throughout the system to the southeast water treatment plant where the water is treated and discharged through effluent drains to the Bay. During wet weather operations both the North Shore and Southwest water treatment plants are in operation. The drainage system also includes large underground storage boxes along the shoreline to help detain excess flow that would otherwise overwhelm the two water treatment plants. When the storage boxes are full, and the treatment plants are at maximum capacity the system also has multiple gravity overflow structures that can operate to provide relief to the combined system allowing untreated effluent to flow directly into the Bay and reduce interior flooding.

Table B-4: List of Current Pumps in the Study Area

Pump Name	Pumping Capacity (MGD)	Operation (wet or all weather)
Channel	103	All Weather
Bruce Flynn	110 (new 150)	Wet
North Shore	150	All Weather
Mariposa	15	All Weather
Davidson	1	Wet
Rankin	3	Wet
Merlin Morris	9.2	Wet
Harriet-Lucerne	7.3	Wet
Twentieth Street	3	All Weather
Berry Street	9.2	Wet
Booster	110	All Weather
Southeast Lift Station	50	All Weather

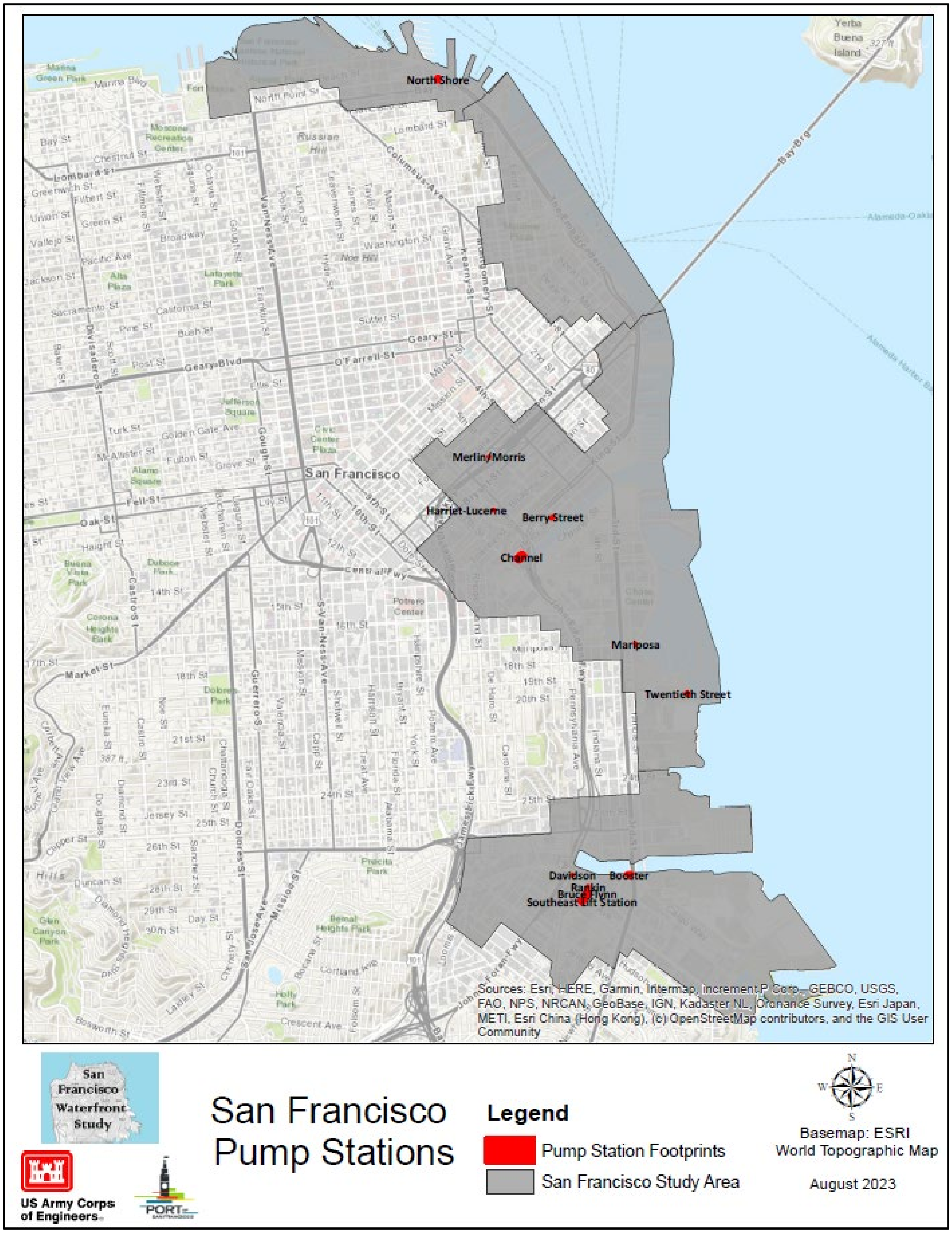


Figure B-24: Location and Names of Pumps in the Study Area

For this study the coastal flood protection system is assumed to be implemented in two phases over the 100-year project life. During the first implementation phase of the TNBP, the existing pump stations and gravity flow systems are acceptable in Reaches 1 and 2; however, 2 additional pump stations will be required in both Reaches 3 and 4 (total of 4 pump stations) in order to reduce the excess flooding caused by installation of the flood protection system. When the second implementation phase of construction occurs at some future time, the bay water levels will be such that gravity flow structures will be mostly ineffective. During this second stage, Reaches 1 and 2 will require construction of 3 pump stations, and Reaches 3 and 4 will require additional pumping capacity achieved by either building new larger pump stations or modifying the existing pump stations with larger capacity pumps. **Table B-5** provides a summary of the additional pumps and total capacity per reach for the pumps estimated as part of the feasibility level analysis for the TNBP.

Table B-5: Size and Pump Estimates for the Total Benefits Plan

Reach	New Pumps – Total Pump Capacity	Action
Reach 1 (Northshore)	No additional pumps	First Action
	1 new pump – 300 cfs (200 MGD)	Second Action
Reach 2 (Northshore and Channel)	No Additional Pumps	First Action
	2 new pumps – 730 cfs (480 MGD)	Second Action
Reach 3 (Channel)	2 new pumps – 800 cfs (520 MGD)	First Action
	2 new pumps upsize 1 pump – 1050 cfs (680 MGD)	Second Action
Reach 4 (Islais Creek)	2 new pumps – 700 cfs (450 MGD)	First Action
	2 new pumps upsize 2 pumps – 2900 cfs (1880 MGD)	Second Action

For additional detail on the interior drainage analysis and pumping requirement for the future with project alternatives see Sub-Appendix B.1.4.

Section B-5. Civil Design

Civil Design efforts will be extensive for this project during the PED phase to properly layout the flood control measures, particularly when considering the site grading, demolition, utility identification and relocation, transportation design (roadway, rail, and sidewalk), traffic control plans, and ADA compliance layout. These details will require much effort and coordination across multiple engineering disciplines and with local, state, and federal agencies. Civil Design efforts to date have been limited in scope.

Section B-6. Mechanical and Electrical Requirements

To date only very broad M&E consideration have been included in the designs. The various components of the flood control structures are anticipated to be refined during PED. The bulk of the M&E efforts during PED are anticipated to be focused on pump stations and gate structures.

Section B-7. Hazardous and Toxic Materials

The shoreline area in San Francisco was initially developed as an industrial port zone which contained a large variety of manufacturing facilities. There was little if any environmental regulations for control or disposal of hazardous or toxic substances during much of the heyday of industrial usage of this area. As a result, virtually all the project area has some level of contamination. As this study progresses, a formal Hazardous, Toxic, and Radioactive Waste (HTRW) survey will be necessary to evaluate the extent of remediation that will be required to construct the proposed flood control structures. Per federal regulations, the nonfederal sponsor is responsible for providing a clean site before any construction can begin.

Section B-8. Alternatives

A total of 7 alternatives were developed for the San Francisco waterfront which consisted of a do nothing alternative, a non-structural alternative, and 5 structural alternatives (formulated to address various SLC potentials over the project life). Each of the five structural alternatives were formulated to be adaptable in the future except for Alternative C, which was specifically formulated to only address SLC associated with the low USACE sea level rise curve estimate. Each alternative is described in full in the Integrated Feasibility Report and Environmental Impact Statement and *Appendix A: Plan Formulation*. Plan view representations of the alternatives are provided in Sub-appendix B.4, and Sub-appendix B.5 for the TNBP.

Section B-9. Quantity Estimates

All quantities were provided to the cost engineer by the design engineers. The PDT engineers responsible for developing the quantities met as group and peer reviewed each other’s numbers prior to providing as final for estimating. Additional details for all quantities in each alternative as well as the TNBP are included in *Appendix C: Cost Engineering*.

Section B-10. Cost Estimates

The baseline cost estimate for the proposed measures, tentative selected plan and the recommended plan were developed using MCACES in the Civil Works Work Breakdown Structure format. Quantities were calculated and provided by the design engineers on the PDT. Real Estate costs for permanent and construction easements and acquisition are currently being calculated. They will be based on parcel data provided by the officials with the CCSF and by the local sponsor. The total real estate costs will be developed and provided by USACE Real Estate personnel. Utility relocations were based on available data and assumptions, more detailed data will need to be obtained in PED phase. The cost estimate for each feature was escalated to the midpoint of construction using the most current indices for the Civil Works Construction Cost Index System (CWCCIS), EM 1110-2-1304 (USACE 2000). For this project, an Abbreviated Risk Analysis (ARA) was performed on a 5% design. Since the design level is so low (5% design), this could inherently result in cost uncertainties that are captured by higher cost contingencies. A Cost and Schedule Risk Analysis (CSRA) is scheduled to be performed after the Tentatively Selected Plan (TSP) is approved. For more information on the Cost Estimates and the Total Project Cost Summary (TPCS) and ARA performed on this project, refer to *Appendix C: Cost Engineering*.

Section B-11. Engineering Risk and Uncertainty

Risk is a measure of the probability (or likelihood) and consequences of uncertain future events. Risk analysis is a decision-making framework that explicitly evaluates the level of risk if no action is taken and recognizes the monetary/non-monetary costs and benefits of reducing risks when making decisions. A variety of variables and their associated uncertainties may be incorporated into the risk assessment of a CFRM study. Design conditions for major coastal and flood protection projects are often vague and design parameters contain large uncertainties.

B-11.1 Life Safety

An abbreviated Semi-Quantitative Risk Assessment (SQRA) for this planning study for the San Francisco Waterfront is scheduled to be conducted in first quarter 2024. This risk assessment will be in accordance with ER 1110-2-1156 *Safety of Dams – Policies and Procedures,* ECB 2019-15 *Interim Approach for Risk-Informed Designs for Dam and Levee Projects*, and draft EC-1165-2-218 *Levee Safety Program – Policy and Procedures* (USACE 2014, 2019, 2021). This effort will consist of a facilitated Potential Failure Mode Analysis (PFMA) and a risk assessment of the potential failure modes judged to be risk drivers.

Prior to completion of the SQRA on the selected plan, a set of qualitative life-safety metrics were developed to evaluate the expected performance of the future with project alternatives. These metrics include both a score for coastal life-safety as well as earthquake life-safety due to the incidental seismic benefits that construction of the CFRM system will have on existing, seismically vulnerable structures. The details of these qualitative metrics can be found in Sub-Appendix B.2 and Sub-Appendix B.3.

Section B-12. Constructability

Construction, particularly in the NWF, will be near/along roadways, mostly the Embarcadero. This will negatively impact traffic and require temporary lane/street closures. Careful project phasing and traffic control are vital components to the work and critical to ensure continued access around the waterfront.

Construction will also occur near and adjacent to structures, some being historic. This leads to smaller, congested work areas, typically taking longer and costing more to complete. Another concern is safety as these areas are more dangerous to the workers.

Weather will impact work and the schedule during construction. The table below shows the anticipated adverse weather delays based on the National Oceanic and Atmospheric Administration (NOAA) for the project location. This is the baseline for monthly weather-related delays.

Table B-6: Anticipated Weather Days

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
3	3	3	1	0	0	0	0	0	1	3	3

Many of the construction activities for the proposed measures produce potentially damaging vibration and noise levels, including pile driving and removal, concrete and asphalt demolition, vibratory compaction, and excavation. Most construction vibrations, except for pile driving, will dissipate relatively quickly. In general, vibratory pile drivers will produce lower vibration levels than impact pile drivers. When pile driving is occurring near or adjacent to structures, monitoring will be required at these locations to ensure settlement/movement is not excessive.

Construction issues may also be encountered due to the highly variable subsurface conditions which will underlie virtually all the flood control measures. In particular, foundation elements which will extend through the AF materials or rock dikes associated with the original seawall construction could require substantial effort to penetrate the fill or may require some alternate foundation system reevaluated to be use as a substitute. The soft soil conditions along the project site could also cause construction issues related to site accessibility and mobility of heavy construction equipment.

B-12.1 Resiliency and Adaptability

Due to sea level rise and the harsh marine environment along the waterfront, measures will be taken to ensure the line of protection can adapt to the changing conditions as well as reduce required maintenance and increase line of defense lifespan. The items listed below were considered by the PDT and will be incorporated during the PED Phase.

- Sub/superstructure to accommodate future raise
- Wider base for levees to accommodate future raise and lessen disruptions
- Utilize durable materials to increase measure lifespan

B-12.2 Increasing Measure Height

The foundation of all T-walls will be designed and constructed with adaptation in mind to be able to support agreed upon future protection heights. Battered piles will be driven into the more stable Upper Layered Sediments underlying the YBM where possible to accommodate future adaptations. During PED, the concrete reinforcement should consider the forces resulting from the additional height as well as planning to only dowel into the existing structure when increasing the stem wall height.

Levees will be constructed with a wider initial footprint to support future height raises. The first phase construction will disrupt a larger area but allows for all future work to be accomplished within the established footprint and limits disruptions outside the work area during any subsequent construction activities.

B-12.3 Corrosion Mitigation

This project is being constructed along the shoreline in a heavily corrosive environment. Therefore, during PED, corrosion mitigation measures should be considered and implemented where practical to reduce required maintenance and ensure longevity of the measures. Examples of corrosion mitigation include:

- Corrosion resistant rebar, such as galvanized, epoxy coated, or FRP composite
- Corrosion resistant sheet piles, such as prestressed concrete, vinyl, or FRP composite
- Corrosion inhibiting admixtures for concrete
- Stainless steel or aluminum for railings and hardware.

Section B-13. Engineering with Nature

USACE is interested in using NNBFs within coastal resilience and CFRM projects. The PDT selected several general NNBF to utilize for the project which included enhancing existing wetlands, ecotone levees, coarse beach, living seawall, ecological armoring, and creek enhancements. The PDT then evaluated the project length for sections where it would be reasonable to implement these NNBF. Many sections where it was most reasonable to implement the NNBF were in the SWF. These features are discussed in more detail in *Appendix I: Engineering with Nature*.

Section B-14. Operation and Maintenance

A draft O&M manual has been started. This preliminary O&M manual will need to be further refined during PED phase as more information is gathered and specific details of each measure are designed.

The local sponsor should be prepared to carry out maintenance activities on all flood risk management structures yearly or more often if required. Regular maintenance is critical as various types of issues will escalate exponentially when left unchecked.

There are many ongoing requirements of which one should be aware. For example, debris and unwanted growth need to be removed from the areas adjacent to floodwalls and levees. Local sponsor will need to periodically install closure structures as required by the inspection and levee safety program. Vegetated levees must be maintained, and no trees shall be planted on or within 15 feet of a flood control measure. It is also noted that O&M also applies to NNBF since they are also part of the designed flood control system. These NNBF will likely require additional effort to monitor and maintain over time given their location and potential regular inundation.

Section B-15. Preconstruction Engineering and Design Considerations

Due to the study area size, schedule, and funding constraints, there is further geotechnical analysis and design required during the PED phases. Some of this work, such as subsurface exploration, will need to start at the beginning of PED to obtain the necessary information to complete geotechnical and structural analyses. The work required during PED is discussed in detail below.

B-15.1 Subsurface Investigation

Subsurface information is needed to provide the designers better data beyond the assumptions made by the PDT. The POSF completed a geotechnical exploration program of the Embarcadero roadway and nearshore area in 2018 to inform a planning level risk assessment of the waterfront. This exploration included nearly 90 explorations which were a mix of Cone Penetration Tests (CPTs), Sonic Drilling, Mud-Rotary Borings, Field Vane Shear tests, and Suspension Logging to characterize the AF, YBM and Upper Layered Sediments. Most of the fill along the TNBP alignment would be categorized as AF as described above. The 2018 explorations showed a high level of variability in the AF, thus indicating additional investigations are required to fully characterize the subsurface. Borings should at a minimum extend into the Upper Layered Sediments to determine the depth of YBM, which is a known factor in shoreline stability.

The 2018 explorations did not include the SWF; therefore, these reaches will need to be thoroughly investigated to better characterize the foundation conditions early in the PED phase.

CPT soundings supplemented with standard penetration test (SPT) borings should be performed. The SPT borings will be used to verify the soil behavior type determined during CPT data reduction. Undisturbed samples should be collected and tested. The testing should consist of drained and undrained shear strength determination, consolidation, and soil classification tests (Atterberg limits and grain size distribution). Seismic refraction tests should also be performed to measure shear wave velocities.

B-15.2 Verify Utility Locations

Exact utility locations and type will need to be verified during PED. Geotechnical explorations conducted in 2018 by the POSF revealed a substantial number of unmarked, unlocated utilities lie below the surface of the Embarcadero roadway. Prior to boring, utility service alerts and independent location using Ground Penetrating Radar were employed, but nearly 50% of the holes identified as clear of utilities were found to contain an obstruction. Reach 2 and the northern portion of Reach 3 should be the first priority as penetrations through the flood protection will be necessary to maintain service to the piers and the NFS experience with prior exploration below the roadway. Consideration should be given to determining if multiple utilities can cross the barrier in a single location to minimize the total number of potential penetrations.

B-15.3 Detailed Surveys

Although several surveys utilizing various topographic intervals is readily available, most of the data is outdated, specifically in the SWF. Detailed surveys will be completed to finalize the measure alignment, extents, and improve quantities for estimating costs. As the local sponsor continues to collect survey information such as LiDAR data in partnership with USGS, this will be used to the maximum extent.

B-15.4 Final Interior Drainage Analysis

The interior drainage analysis completed prior to the release of the draft report is based on a simplified overland flow model. The subsurface drainage system was considered but time did not allow for the PDT to obtain the needed information to fully model and include. Additionally, the interior drainage analysis was completed prior to hybridization of structural and non-structure plans, which will influence the hydraulics of the overland flow as well as the hydrologically connected subsurface drainage infrastructure. Effort will be made to include these revisions to the interior drainage model prior to release of the final report, to ensure the scope of interior drainage infrastructure associated with the coastal defense system meets the requirements. The PDT expects that during the PED phase the interior hydrology should be more accurately modeled to account for surface and subsurface flow interactions as well as the variable tidal boundary condition at the shoreline, which will ensure the pumps and requisite conveyance infrastructure are adequately sized and strategically placed.

B-15.5 Design of Urban Landscape and Transportation Corridor

For evaluation of alternatives, it was assumed the alternatives would reinstate the existing surface conditions, such as identical number of traffic lanes, transit tracks and landscape design as a baseline for estimating cost. The PDT assumes the design of the urban landscape and transportation corridor above and adjacent to the CFRM system will be further developed during the PED phase.

Section B-16. References

Aagaard, B.T., et al. 2016. Earthquake Outlook for the San Francisco Bay Region 2014-2043 (Ver. 1.1, August 2016), USGS Fact Sheet 2016-3020.

Bartow, G.W., and Johnson, K.A. 2018. Geology of San Francisco, California. Association Engineering Geologists. 13th International Association of Environmental & Engineering Geologists Congress in San Francisco, CA.

CH2M/Arcadis Team. 2020a. Buildings & Marine Structures – Earthquake Exposure, Vulnerability, and Consequences Report. Port of San Francisco, San Francisco, CA.

CH2M/Arcadis Team. 2020b. Earthquake Hazard and Geotechnical Assessment Report. Port of San Francisco, San Francisco, CA.

CH2M/Arcadis Team. 2020c. Geotechnical Interpretive Report. Port of San Francisco, San Francisco, CA.

CH2M/Arcadis Team. 2020d. Multi-Hazard Risk Assessment Northern Waterfront and Embarcadero Seawall Summary Report. Port of San Francisco, San Francisco, CA.

CH2M/Arcadis Team. 2020e. Utility and Mobility Systems – Earthquake and Coastal Flood Exposure, Vulnerability, and Consequences Report. Port of San Francisco, San Francisco, CA.

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

U.S. Army Corps of Engineers (USACE). 1975. Final Composite Environmental Statement, Maintenance Dredging, Existing Navigation Projects, San Francisco Bay Region, California.

U.S. Army Corps of Engineers (USACE). 1989. EM 1110-2-2502 *Retaining and Flood Walls.*

U.S. Army Corps of Engineers (USACE). 2000. EM 1110-2-1304 *Civil Works Construction Cost Index System (CWCCIS).*

U.S. Army Corps of Engineers (USACE). 2000. EM 1110-2-1913 *Design and Construction of Levees.*

U.S. Army Corps of Engineers (USACE). 2014. ER 1110-2-1156 *Safety of Dams – Policies and Procedures*.

U.S. Army Corps of Engineers (USACE). 2016. ER 1110-2-1806 *Earthquake Design and Evaluation for Civil Works Projects*. May 31.

U.S. Army Corps of Engineers (USACE). 2019. ECB 2019-15 *Interim Approach for Risk-Informed Designs for Dam and Levee Projects*,

U.S. Army Corps of Engineers (USACE). 2019. ER 1100-2-8162 *Incorporating Sea Level Change in Civil Works Programs*.

U.S. Army Corps of Engineers (USACE). 2021. EC-1165-2-218 *Levee Safety Program – Policy and Procedures.*