SAN FRANCISCO WATERFRONT COASTAL FLOOD STUDY, CALIFORNIA

APPENDIX B – ENGINEERING

JULY 2026

USACE TULSA DISTRICT | THE PORT OF SAN FRANCISCO

US Army Corps of Engineers

PORT OF SAN FRANCISCO

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Acronyms and Abbreviations

Acronym	Definition
ADA	Americans with Disabilities Act
AF	Artificial Fill
ARA	Array of Alternatives
AWSS	Auxiliary Water Supply System
BART	Bay Area Rapid Transit
BLP	Boring Location Plan
CCSF	City and County of San Francisco
CFRM	Coastal Flood Risk Management
CGS	California Geological Survey
cfs	cubic feet per second
CPT	Cone Penetration Tests
CSRA	Cost and Schedule Risk Analysis
CSRM	Coastal Strom Risk Management
CWCCIS	Civil Works Construction Cost Index System
cy	cubic yard
DSM	Deep Soil Mixing
EM	Engineering Manual
FHWA	Federal Highway Administration
GIS	Geographic Information System
H	Horizontal
HEC-RAS	Hydrologic Engineer Center River Analysis System

HH&C	Hydrology, Hydraulics & Coastal
HTRW	Hazardous, Toxic, and Radioactive Waste
lf	linear foot
LiDAR	Light Detection and Ranging
LLS	Lower Layered Sediment
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
OBM	Old Bay Mud
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
RSLC	Relative Sea Level Change

SFFD	San Francisco Fire Department
SPK	USACE Sacramento District
SFMTA	San Francisco Mass Transit Authority
SFPUC	San Francisco Public Utilities Commission
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
ULS	Upper Layered Sediment
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
V	Vertical
VLM	Vertical Land Movement
WE	Wastewater Enterprise
WETA	Water Emergency Transportation Authority
YBM	Young Bay Mud

1.0 Existing Information and Data

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 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 typical cross sections over defined lengths. 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 design 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 formal plans and specifications for construction. The City and County of San Francisco (CCSF) partnered with U.S. Geological Survey (USGS) to complete an updated Quality Level 0 LiDAR survey, which was completed in 2024. This 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 in the design phase of this project.

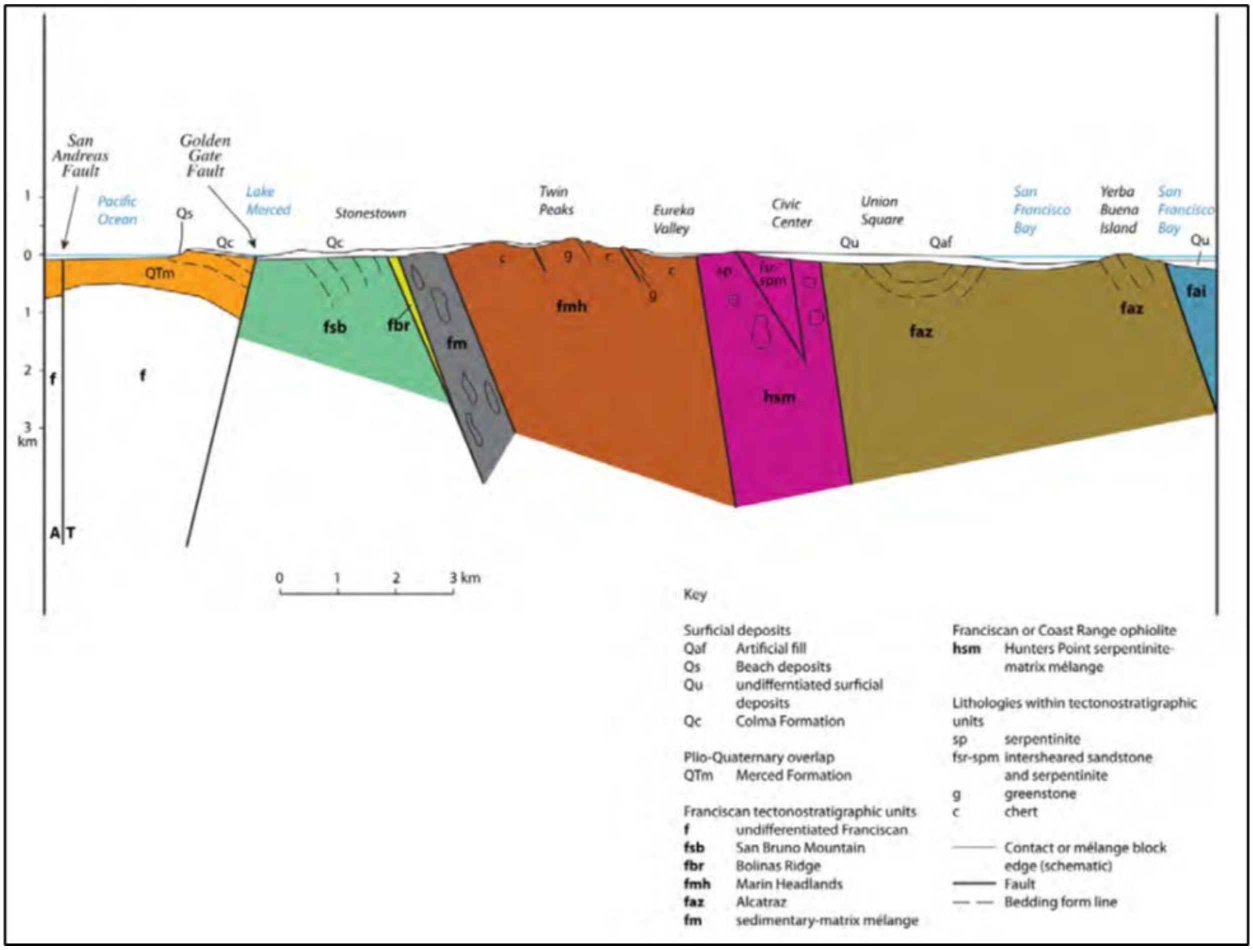
1.2 Geological and Geotechnical Assessments

1.2.1 Regional Geology

1.2.1.1 Basement Complex in San Francisco

The geology of San Francisco 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

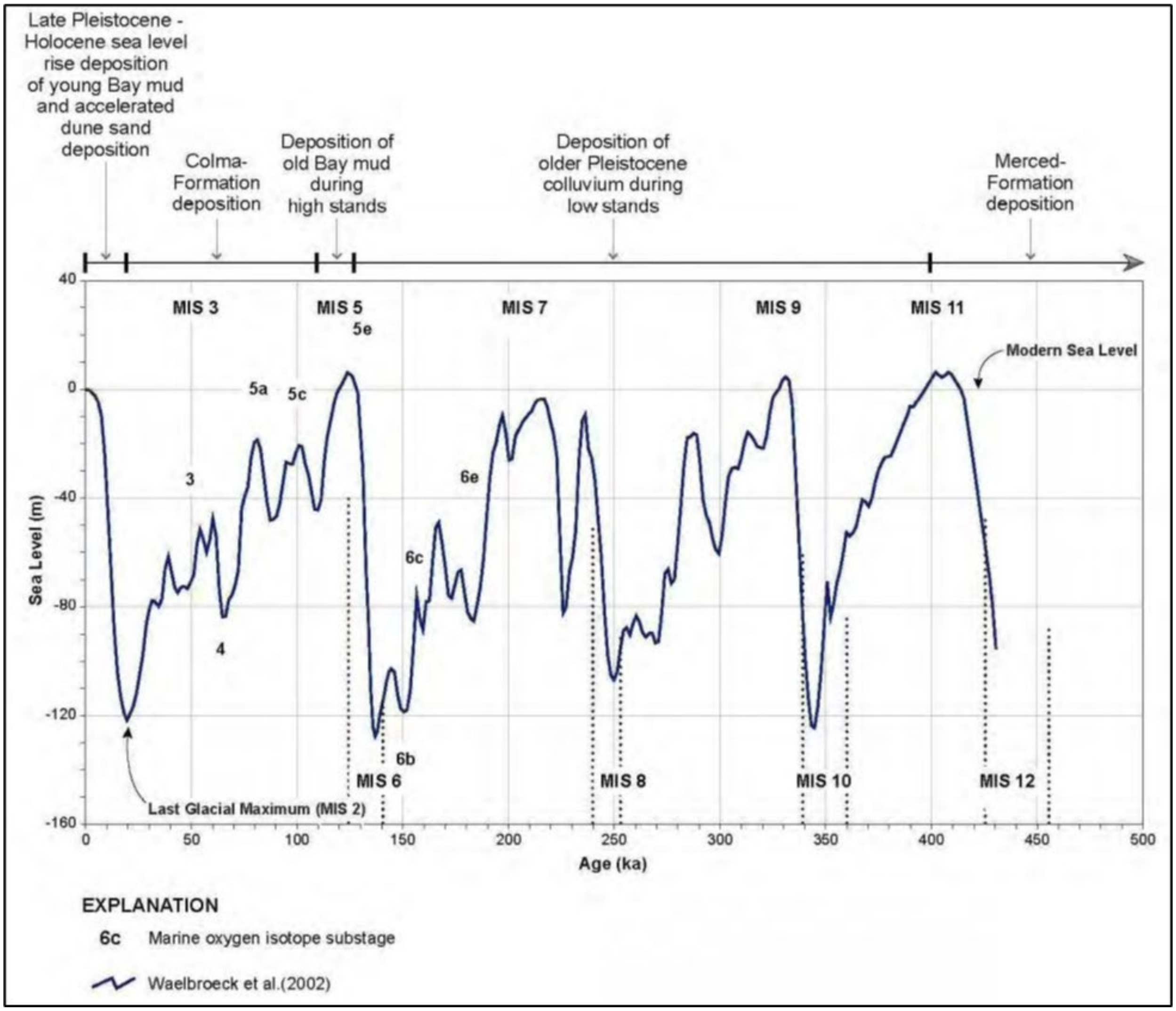
The Alcatraz Terrane crops out in northeast San Francisco, including Alcatraz and Yerba Buena Island, where it forms scattered hills that rise above the bay floor and flat-lying areas of the city. The hills probably represent erosion-resistant parts of a larger, but mostly hidden, northwest–southeast 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 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.

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**).

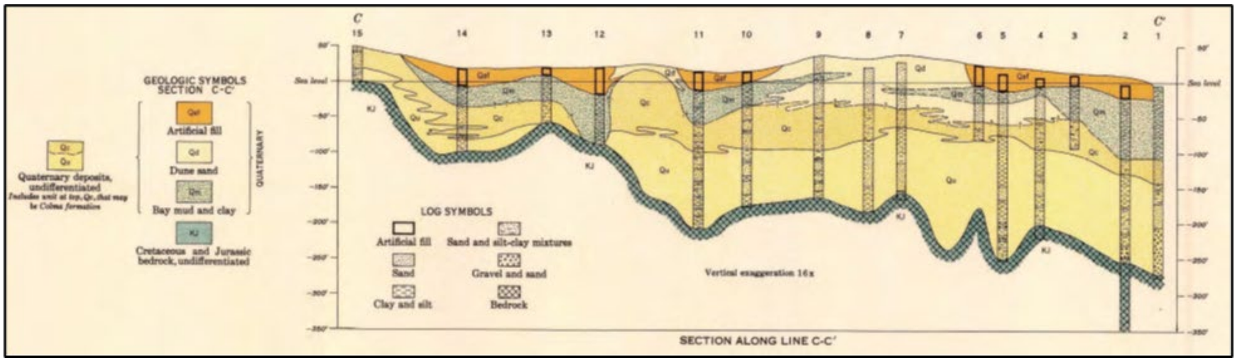
Nearly all the major Quaternary stratigraphic packages in the Bay Area owe their origin to a rising or lowering of the sea over these glacial–interglacial cycles. For example, sea levels in the late Quaternary fluctuated in elevation by over 330 feet globally between glacial (low sea level) and intervening 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 as detailed in Bartow and Johnson (2018).



Source: Bartow amd Johnson 2018

Figure B-3: Section Generally along Market Street

1.2.1.3 Lower Layered Sediments (LLS)

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 in **Figure B-3**. 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 pre-consolidated soils. Total densities range from approximately 115 to 140 pounds per cubic feet (pcf).

1.2.1.4 Old Bay Mud (OBM)

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 pre-consolidated. Total densities range from 100 to 135 pcf.

1.2.1.5 Upper Layered Sediment (ULS)

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.

1.2.1.6 Young Bay Mud (YBM)

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.

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.

1.2.2 Geologic Profiles

Geologic profiles detailing two general segments of the project length consisted of the Northern Waterfront (NWF) and the Southern Waterfront (SWF). It is noted that there is no engineering or geologic significance in dividing the geologic profiles into the NWF and SWF, as these general segments are solely a delineation denoted in previous evaluations of facilities performed by the Port. The NWF profile covers areas defined as Reaches 1, 2 and the northern segment of 3 in the feasibility report, and the SWF profile covers the areas defined as the remainder of Reach 3 and all of Reach 4. It is noted that there is some overlap in the profiles between the SF-Oakland Bay Bridge and Giants Stadium. Various strata identified in the geological profiles contained herein are labeled with the acronyms listed in previous sections.

The NWF geologic profile was developed by the Port's engineering consultants (Fugro/CH2M-Arcadis) as presented in the report titled Earthquake Hazard & Geotechnical Assessment Report, dated April 2020. **Figure B-4** provides an aerial view of the extent of the NWF geologic profile, and **Figure B-5** is the consultant generated geologic profile developed for the NWF. The NWF geologic profile was developed utilizing a combination of existing subsurface information collected from previous explorations for Port and private facilities and using soil test borings and Cone Penetration Tests (CPT) performed for their study.

Figure B-4: Aerial view depicting the extent of the NWF geologic profile

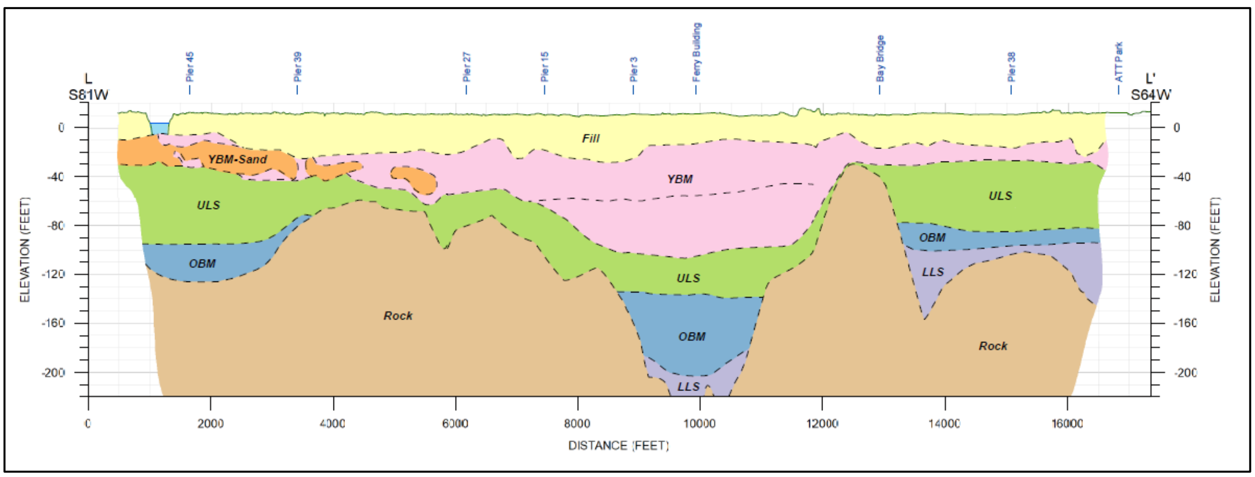


Figure B-5: Geologic Profile for the NWF

The SWF geologic profile was developed utilizing advanced geotechnical modeling techniques, including Google Earth for spatial data visualization, Openground for subsurface data management, and Leapfrog for geological modeling. Historical geotechnical data from previous studies by the Port and private ventures were compiled and analyzed to develop a comprehensive 3D model to advance the understanding of the site's geology to refine the flood protection design. The historical reports ranged in age from approximately the 1940s to the present day.

A total of 205 historical geotechnical reports from Reach 3 (89 reports) and Reach 4 (116 reports) were provided by the Port and were utilized to create a comprehensive Boring Location Plan (BLP) in Google Earth. A total of 587 borings were extracted from the geotechnical reports, and a kmz file was created in Google Earth that included the location of each boring, along with boring depth and link to the individual geotechnical report of origin. The borings were georeferenced using historical maps from the reports and marked using place markers. The BLP **(Figure B-6**) provides a visual representation of all the boring data utilized across the SWF project area.

Figure B-6: BLP of the borings from the 205 historical geotechnical reports

Given the extent of the SWF area and the level of effort required to input the geotechnical data into a usable format, the PDT developed criteria to filter the 587 borings into a representative, yet manageable, data set. The filtering criteria consisted of selecting borings generally on or near the study alignment, on a 500-foot spacing grid with preference toward deeper borings that terminated in bedrock and with sufficient details to differentiate the various geologic strata. Using these criteria, a total of 270 borings, as shown in **Figure B-7**, were selected for processing into OpenGround and then subsequent incorporation into the Leapfrog model.

Figure B-7: BLP of the 270 borings selected to develop the SWF geologic profile

The historical geotechnical reports and associated boring logs were provided in PDF file format. To utilize the information for our modeling purposes, the individual geotechnical data information on each boring log was manually input into an Excel spreadsheet in a format that allowed for importing into Openground. The data fields included the coordinates developed in Google Earth, boring surface elevation, boring total depth, and geology grouping as defined on the individual report boring logs. The subsurface strata as defined in the borings consisted of 6 geological groups including: AF, YBM, ULS, OBM, LLS, and Bedrock. These geologic classifications were defined in the preceding sections of this report and are consistent with those used in the NWF geologic profile.

The initial stage of the works in Leapfrog consisted of developing a georeferenced contour map of the project area ground surface. A 1-foot contour Geographic Information System (GIS) map was provided by the Port for our use. This GIS model was converted into CAD format to import into Leapfrog. The 3D contour ground surface was then developed for the project. Next, the filtered boring data was imported from Openground into Leapfrog to enable the development of the subsurface 3D geologic model. Leapfrog software has multiple ways to group or delineate the various geologic zones across the 3D model, with an important input being the deposition order of the various geologic groups defined from youngest to oldest. Since the various geologic groups are not continuous across the entire 3D model, it is necessary to refine how the program will model isolated ‘bubbles’ of various layers within other layers. Reasonable modeling results required engineering/geologic judgment and iteration of various modeling routines within Leapfrog to arrive at the final interpreted model. As a check of the 3D geologic model, several sections were cut corresponding to the historical fence diagrams in the historical geotechnical reports. These are provided below for comparison (**Figure B-8** and **Figure B-9**) and indicate relatively consistent results. The 3D model can be viewed in a multitude of ways to visualize the model to include cutting sections through any portion of the model or visualizing only certain geologic layers, for example. As delineated in **Figure B-10**, a cross-section cut along the entire shoreline is provided in **Figure B-11**. Shorter increments are provided in **Figure B-12** to **Figure B-20**, as delineated in **Figure B-11**, where the end of the increments labeled ‘A’ are in the northernmost orientation. Two independent consultants, who work extensively in the Bay area and regularly perform engineering work for the Port, provided a cursory review of the model and provided general comments and informal advisory input, which were incorporated as appropriate. In general, the development of this 3D model was found to be an appropriate tool for the feasibility stage with the expectation that it will be refined and updated as additional subsurface information is developed in the design phase of the project.

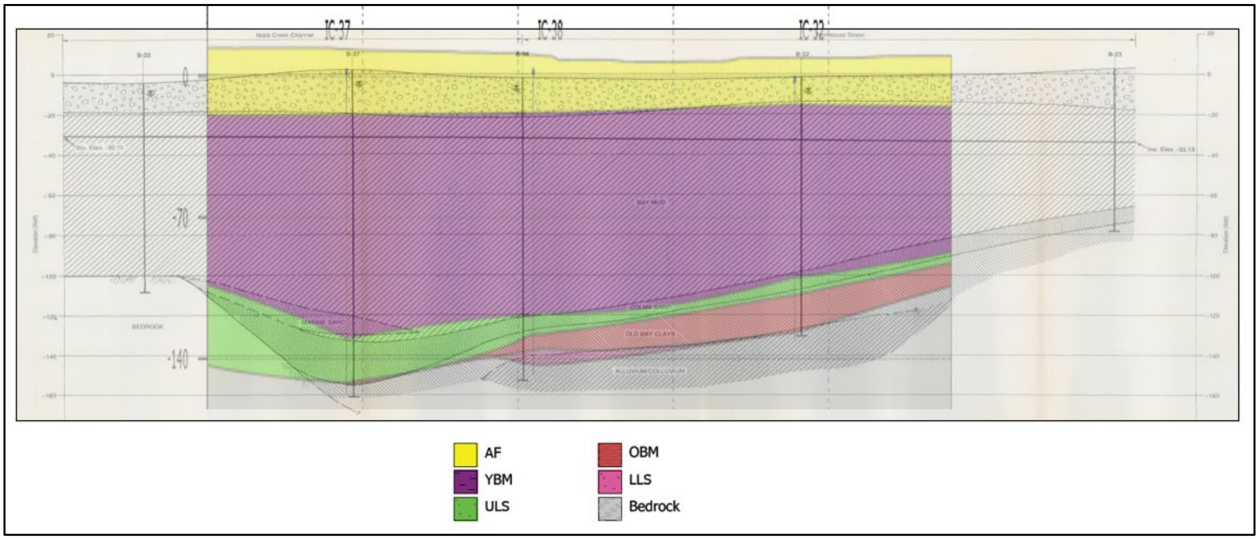


Figure B-8: Comparison #1 of Leapfrog (in color) and historic geologic sections (grayscale)

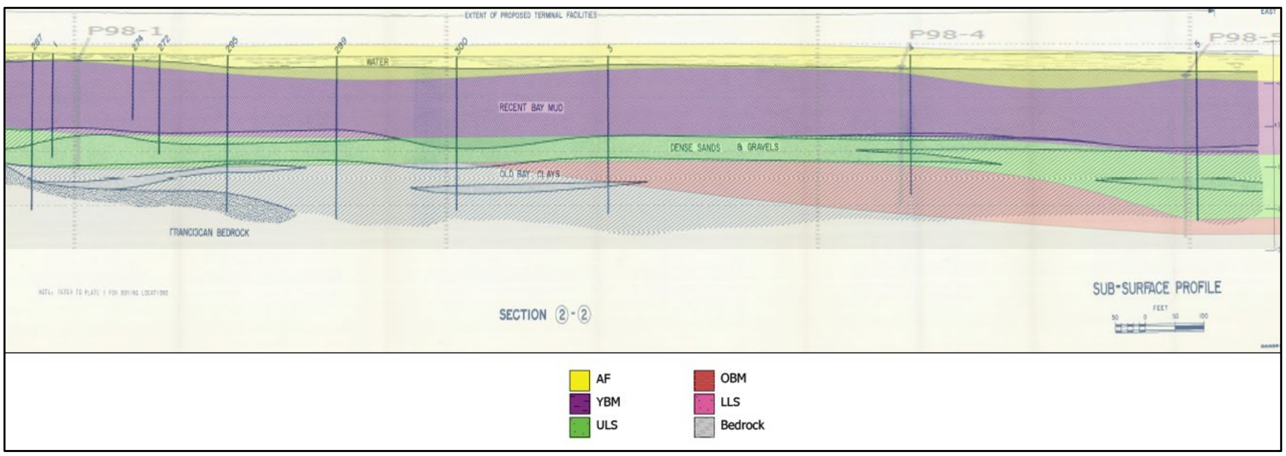


Figure B-9: Comparison #2 of Leapfrog (in color) and historic geologic sections (grayscale)

Figure B-10: Aerial view delineating the geologic profile location

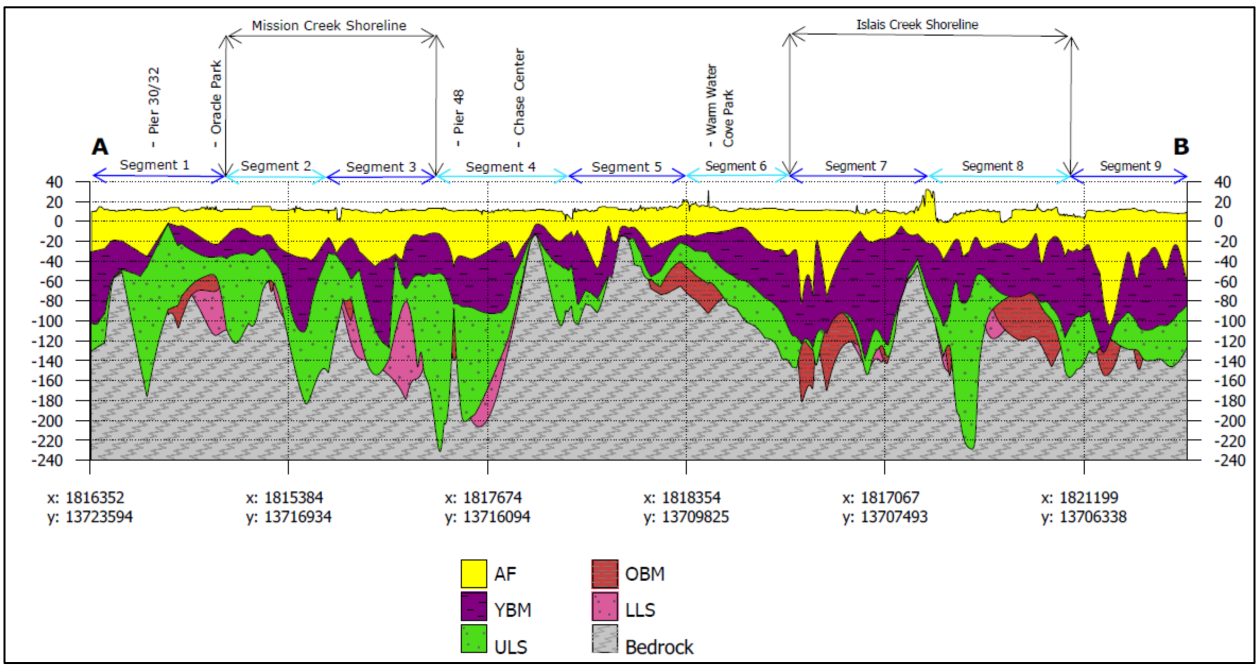


Figure B-11: Leapfrog developed geologic profile for the entire SWF (A-B = north-south)

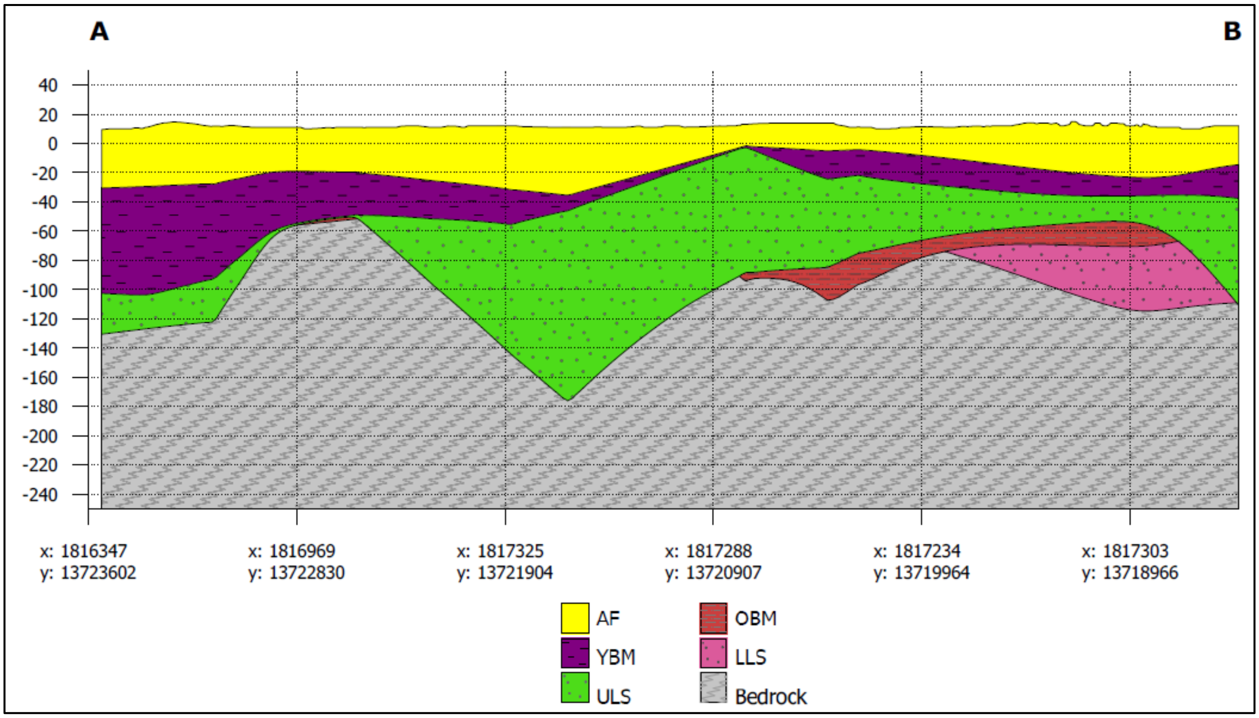


Figure B-12: Segment 1 geologic profile for the SWF (A-B = north-south)

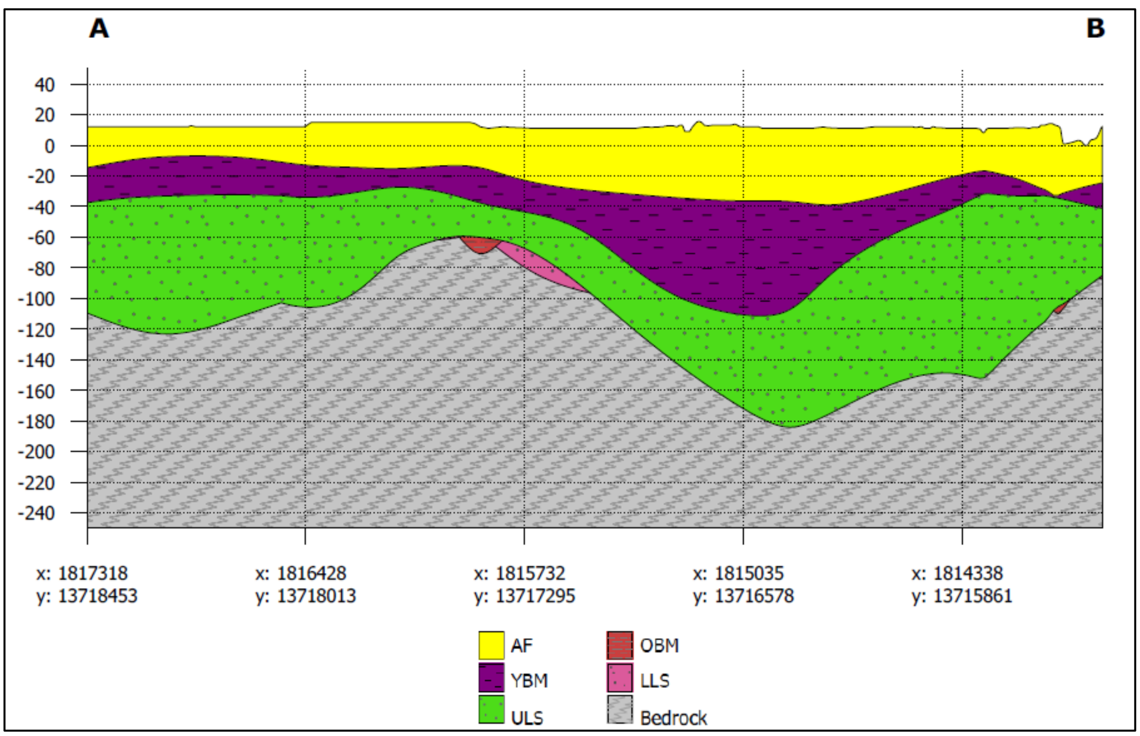


Figure B-13: Segment 2 geologic profile for the SWF (A-B = Mouth of Mission Creek-Inland End)

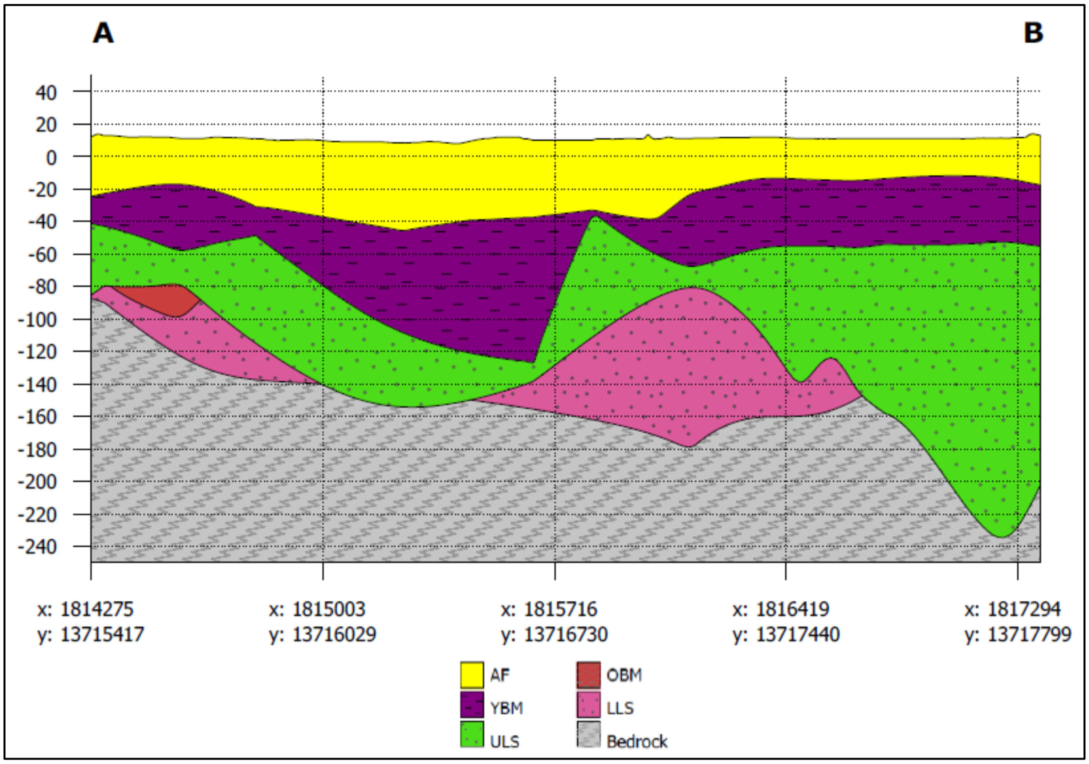


Figure B-14: Segment 3 geologic profile for the SWF (A-B = Inland End-Mouth of Mission Creek)

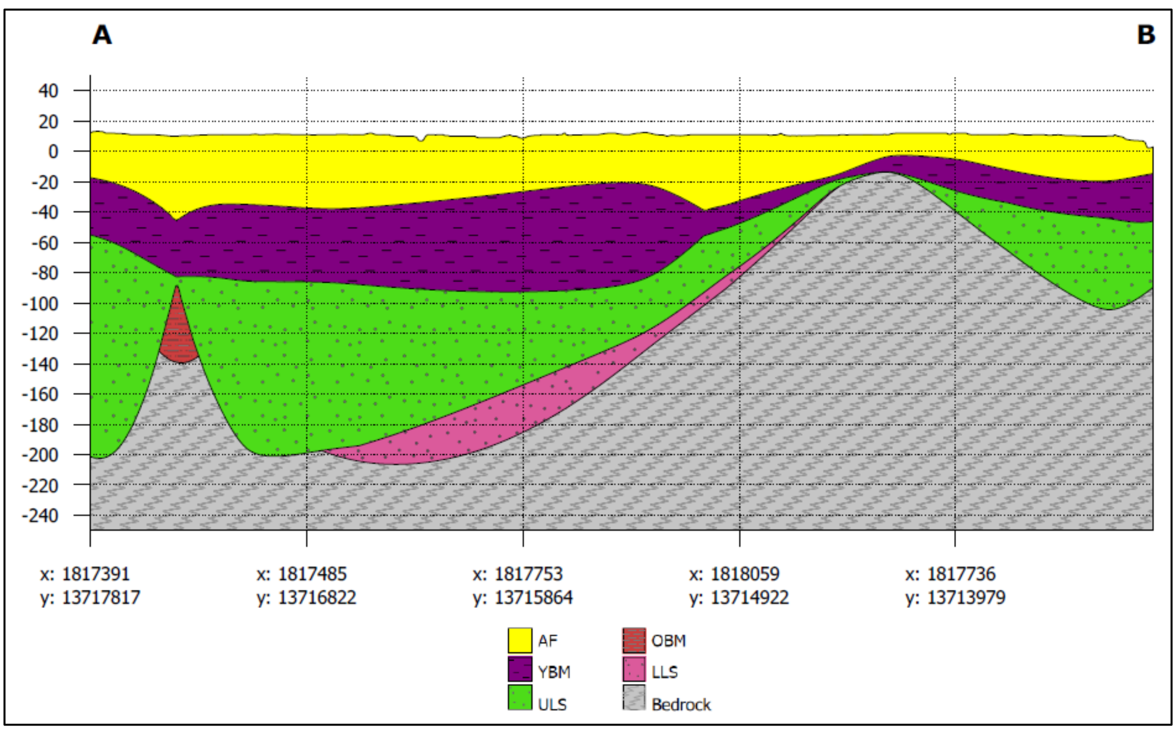


Figure B-15: Segment 4 geologic profile for the SWF (A-B = north-south)

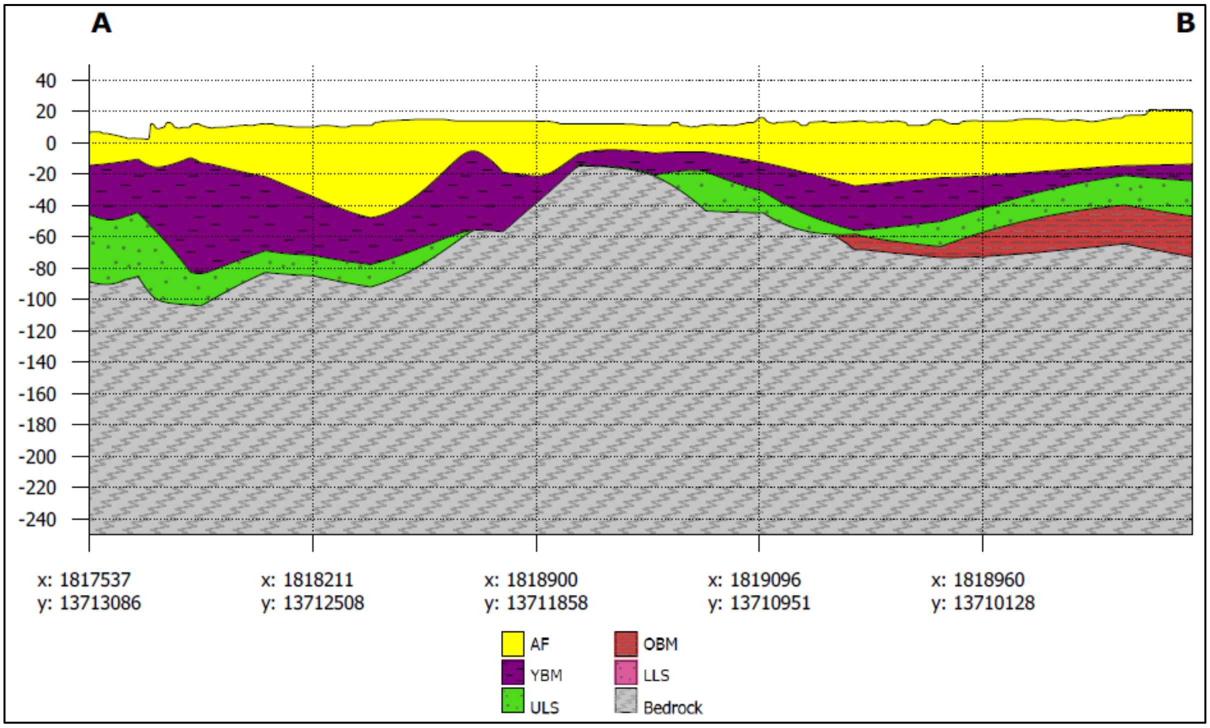


Figure B-16: Segment 5 geologic profile for the SWF (A-B = north-south)

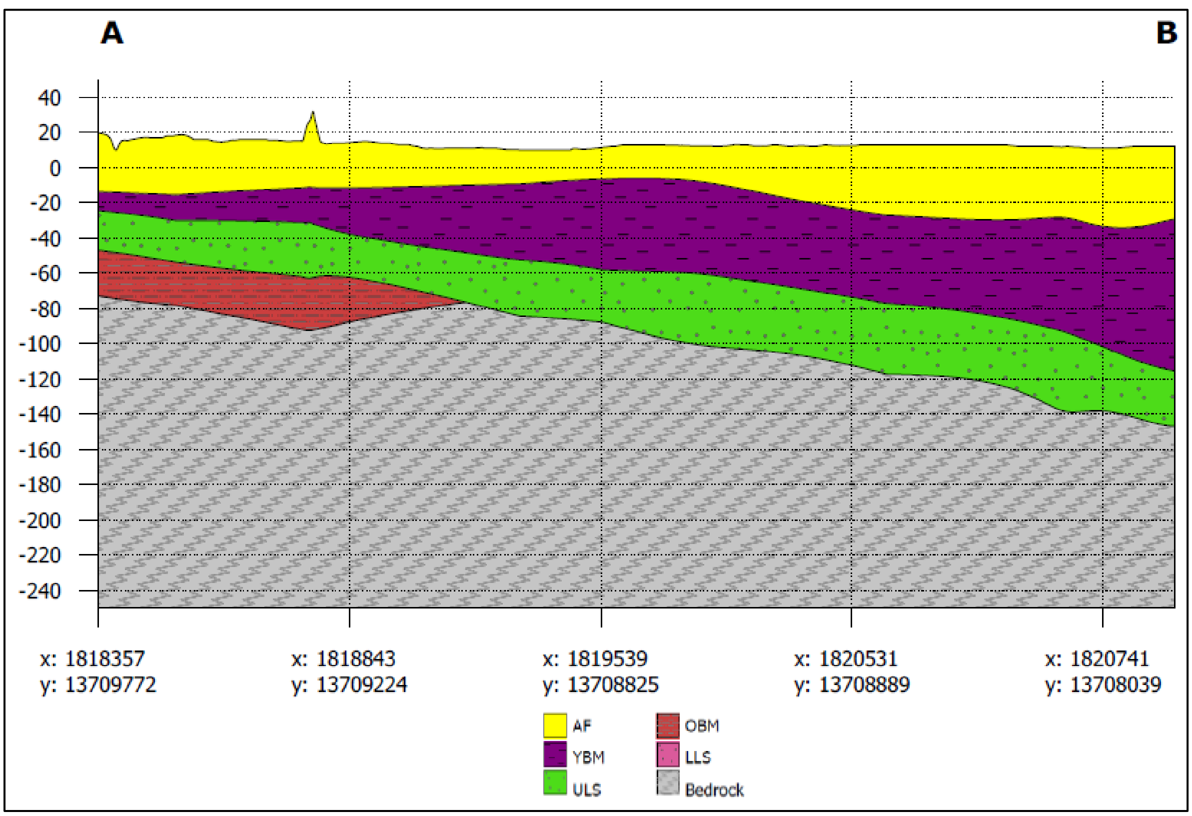


Figure B-17: Segment 6 geologic profile for the SWF (A-B = north-south)

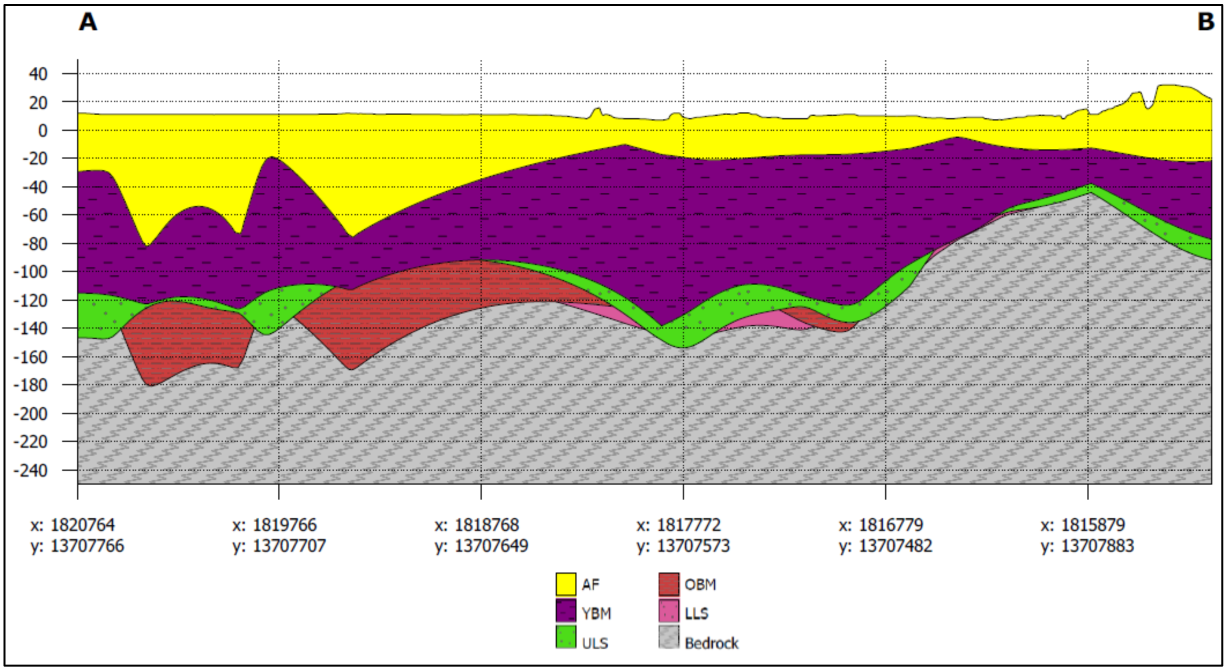


Figure B-18: Segment 7 geologic profile for the SWF (A-B = Mouth of Islais Creek-Inland End)

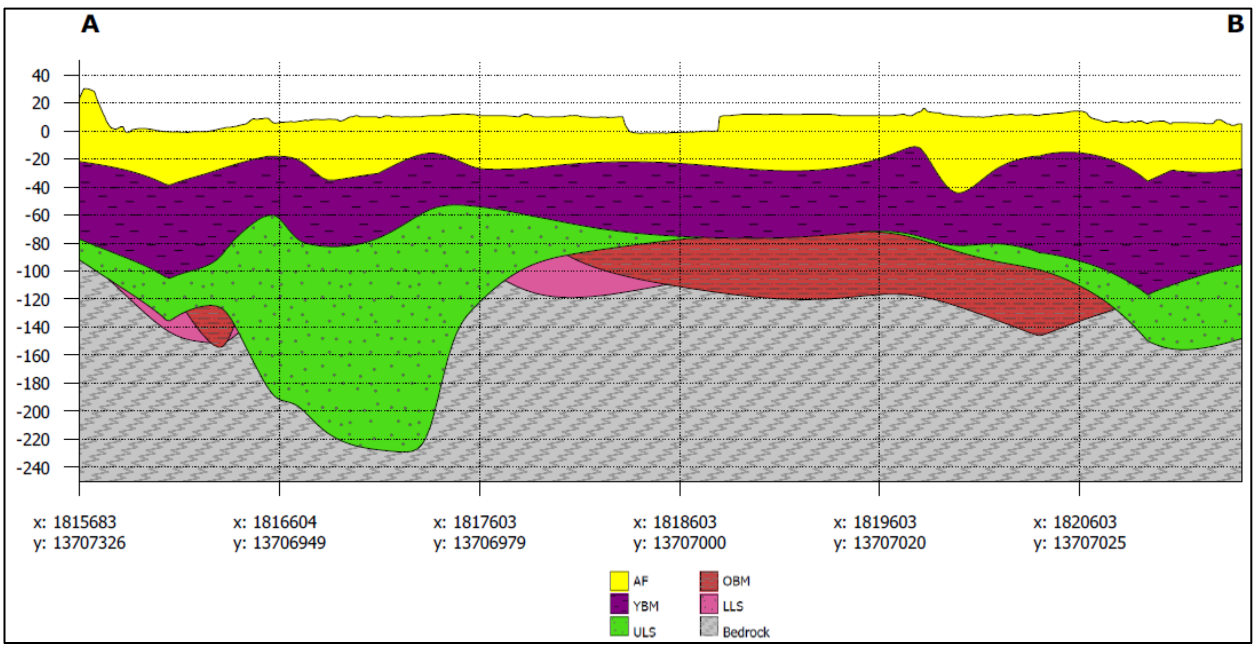


Figure B-19: Segment 8 geologic profile for the SWF (A-B = Inland End-Mouth of Islais Creek)

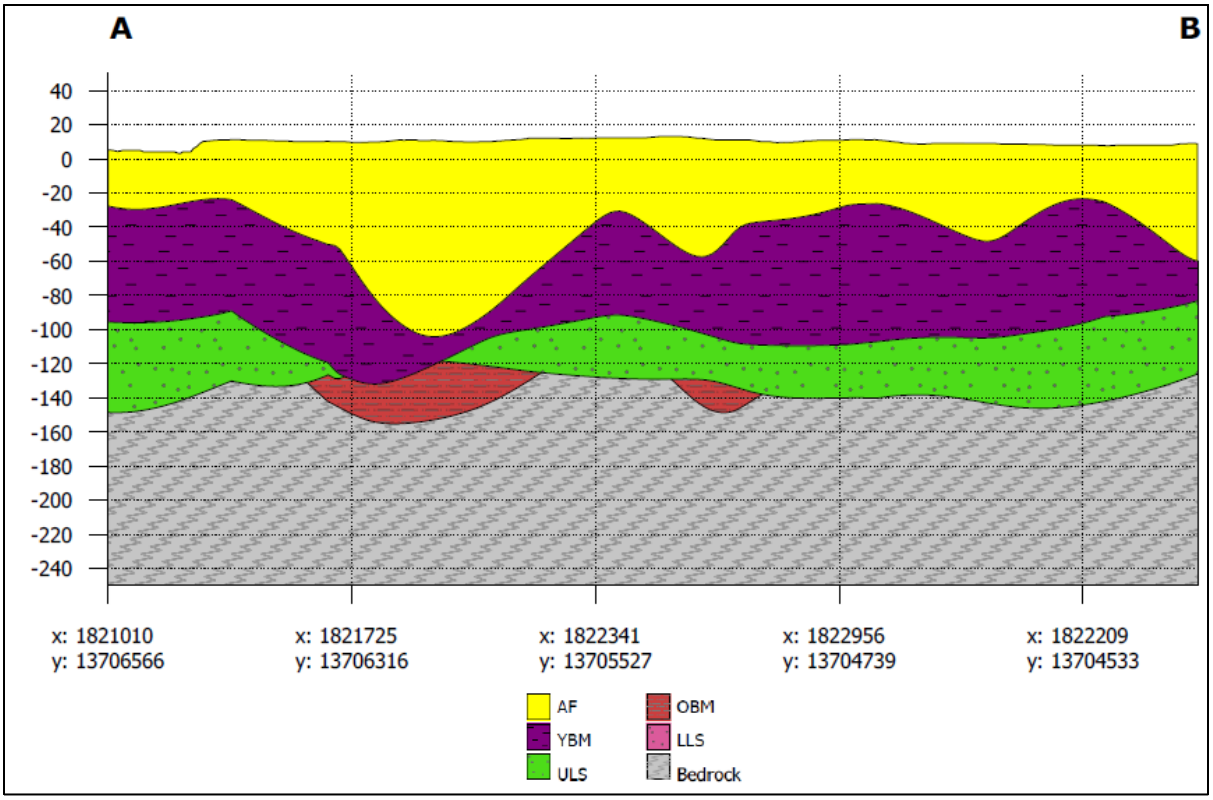
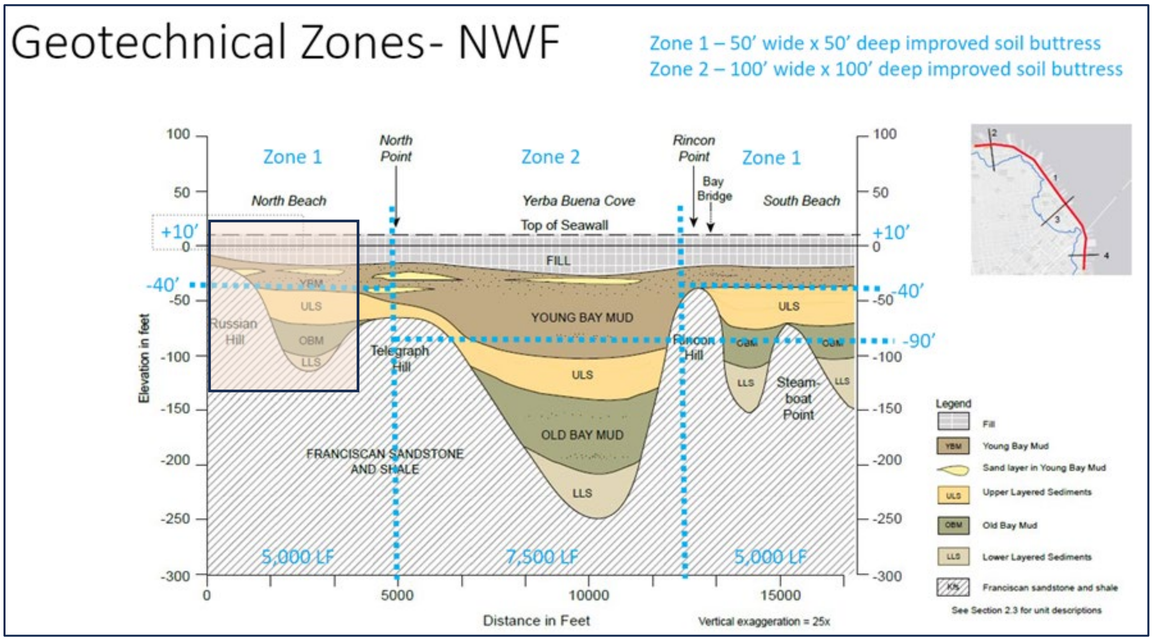


Figure B-20: Segment 9 geologic profile for the SWF (A-B = north-south)

1.3 Ground Improvement Footprint

Selection of the ground improvement footprint and location was based on achieving a stable foundation considering both static and dynamic loading conditions as provided in ER 1110-2-1806, EM 1110-2-1913, EM 1110-2-2100, and EM 1110-2-2502 for support of the earthen and structural flood protection features. The PDT delineated two generalized ground improvement zones (designated Zone 1 and Zone 2) along the project length based on the overall thickness of the AF and YBM deposits which account for virtually all the subsurface vertical and lateral deformation potential due to static and dynamic loading. **Figure B-21** depicts a geologic profile along the NWF area which illustrates the overall thickness of the deposits over this part of the project and also indicates the distribution of Zones 1 and 2. Currently, the northern most Zone 1 area will not be part of the initial project features since this area is designated to be a non-structural solution in the Recommended Plan (area has been shaded in the figure); however, if future structural flood control measures are constructed in this area due to higher rates of RSLC then ground improvements will be required as indicated in conjunction with the subsequent adaptive action. Similarly, **Figure B-22** to **Figure B-30** depict geologic profiles along the SWF area with the assigned stretches of Zones 1 and 2. The areas in the SWF that show no Deep Soil Mixing (DSM) ground improvements are areas where the ground surface is currently above or will be raised to at least the initial flood protection elevation outside of the Federal project; therefore, no new flood protection features are required as a part of this project. The performance of these features outside of the Federal project have not been evaluated as part of this feasibility study, as the PDT assumes that these will be designed in accordance with applicable local building codes and standards and will satisfy the USACE design requirements.



Source: CH2M/Arcadis Team 2020c

Figure B-21: NWF Geologic Profile with Soil Zones 1 and 2 Delineated

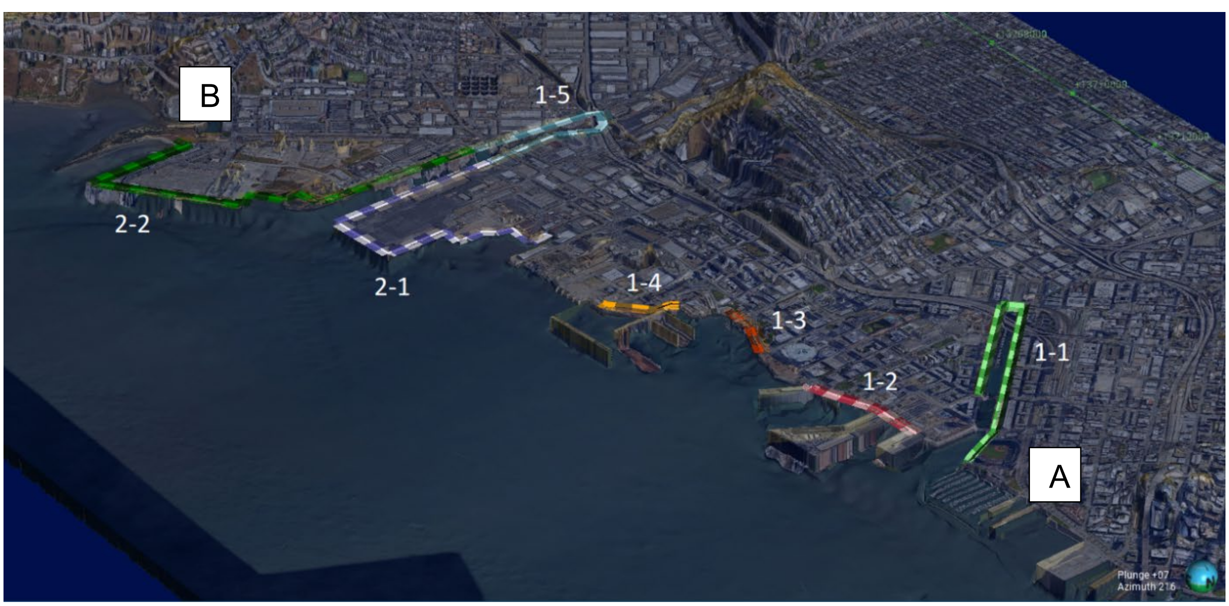


Figure B-22: Plan Views of the NWF and SWF With soil Zones 1 and 2 Delineated

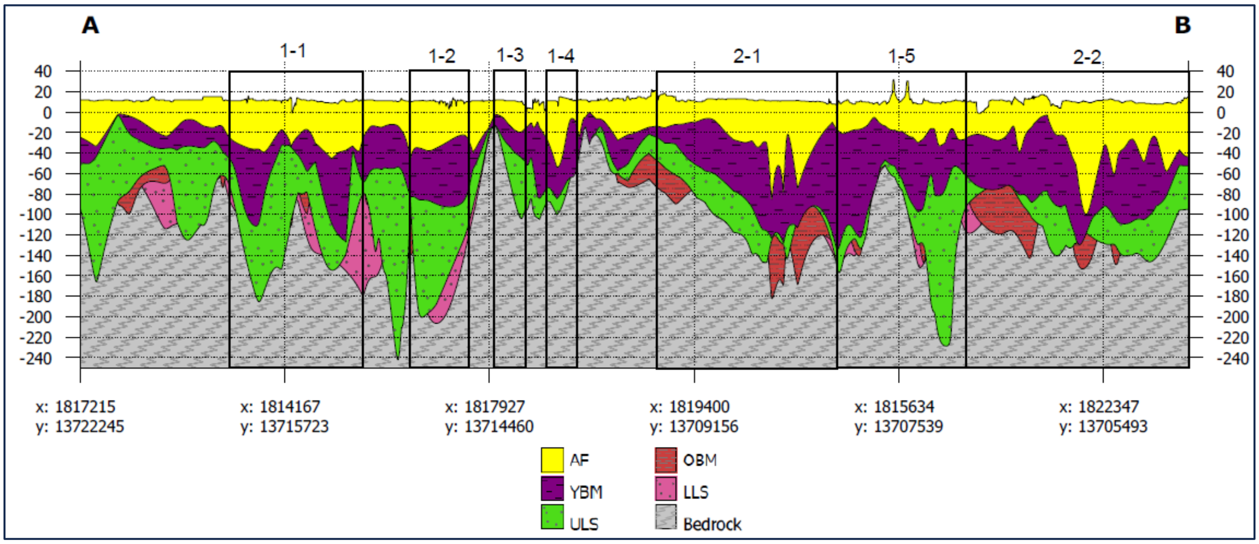


Figure B-23: SWF geologic profile with Ground Improvement segments (A-B = north-south)

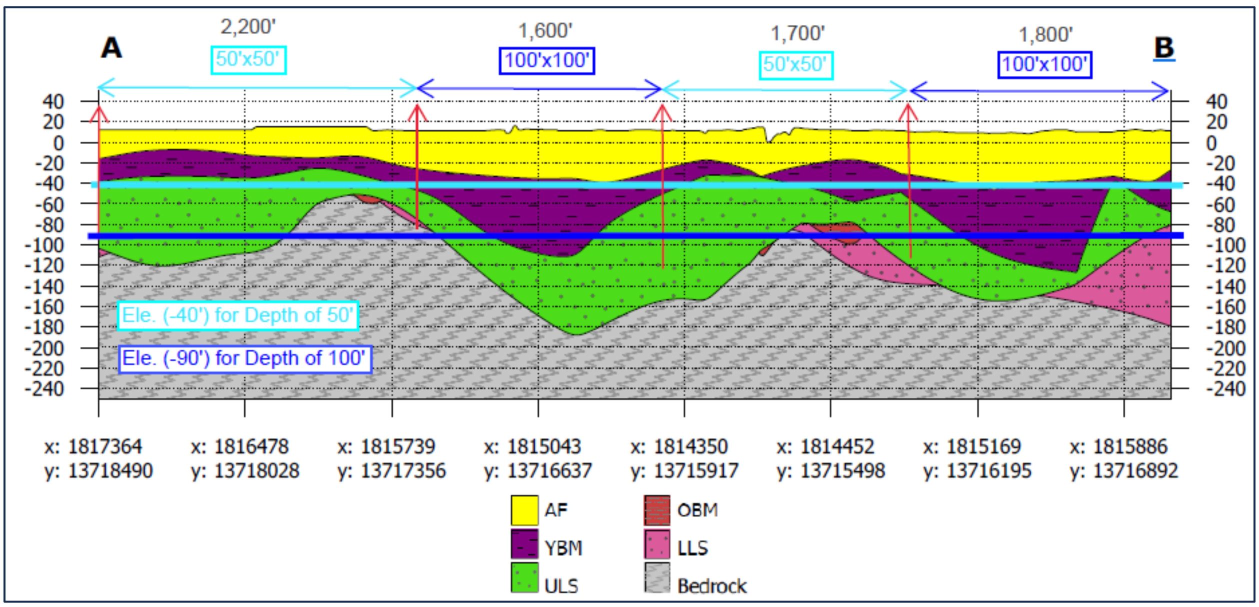


Figure B-24: SWF geologic profile with Ground Improvement segment 1-1 (A-B = north-south)

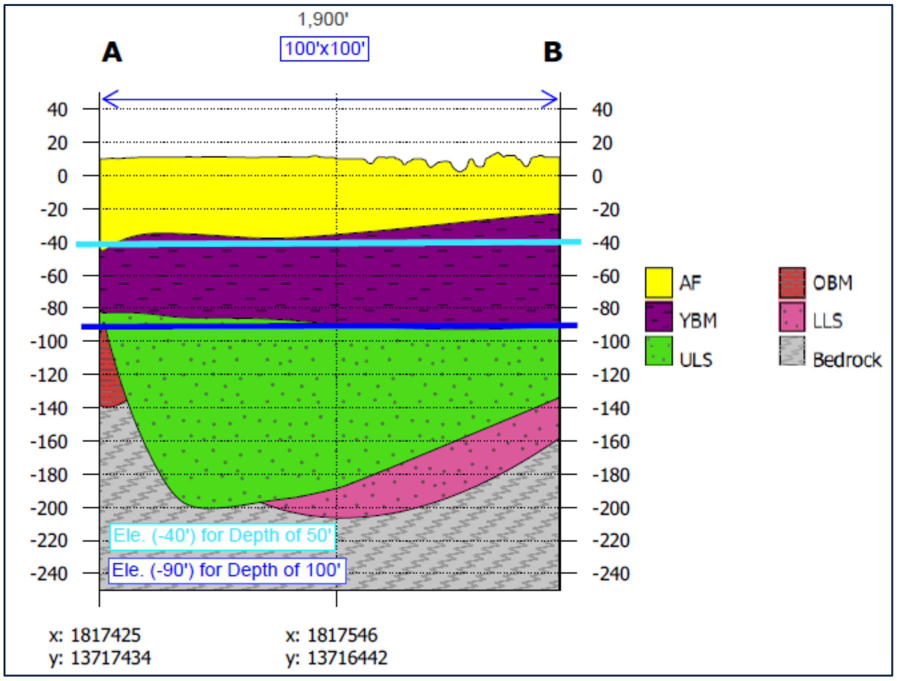


Figure B-25: SWF geologic profile with Ground Improvement segment 1-2 (A-B = north-south)

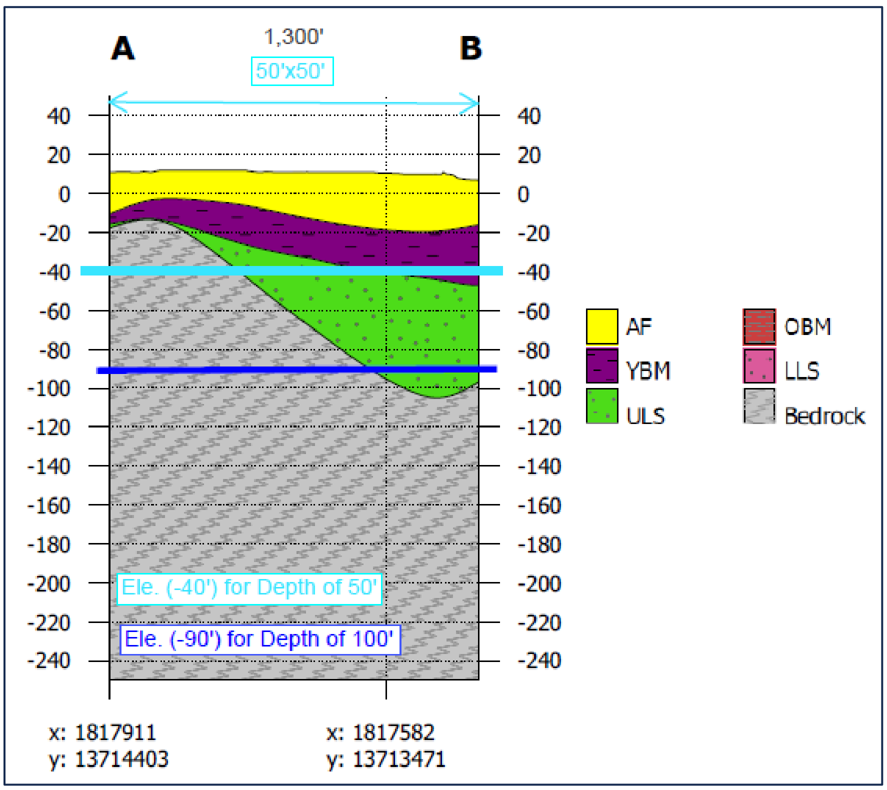


Figure B-26: SWF geologic profile with Ground Improvement segment 1-3 (A-B = north-south)

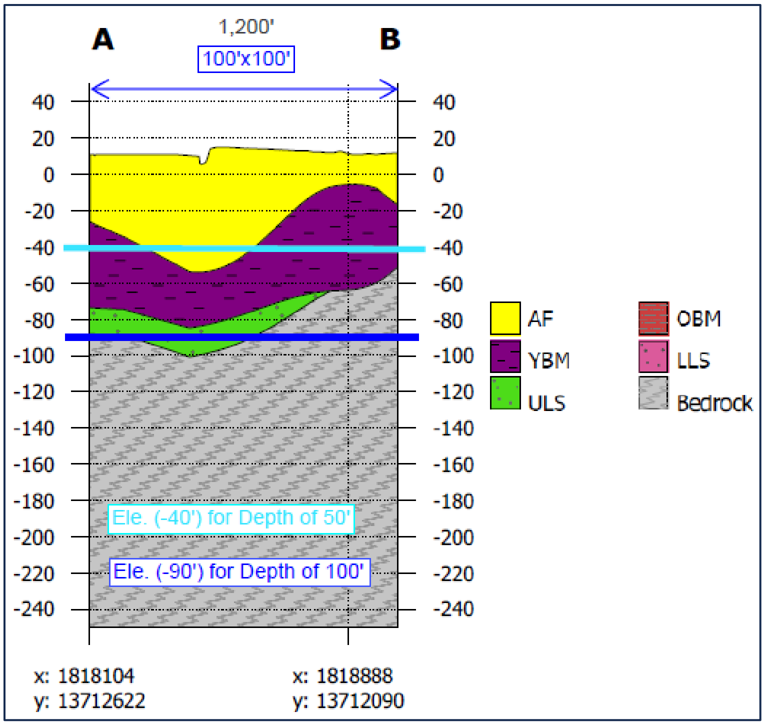


Figure B-27: SWF geologic profile with Ground Improvement segment 1-4 (A-B = north-south)

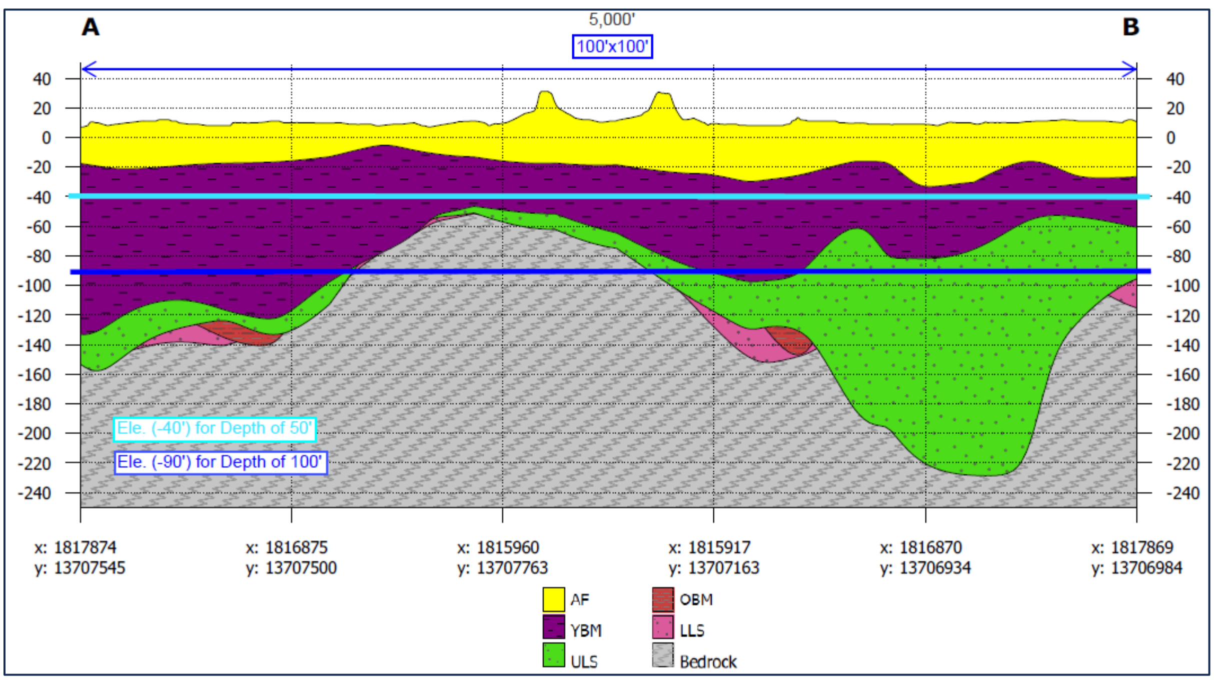


Figure B-28: SWF geologic profile with Ground Improvement segment 1-5 (A-B = north-south)

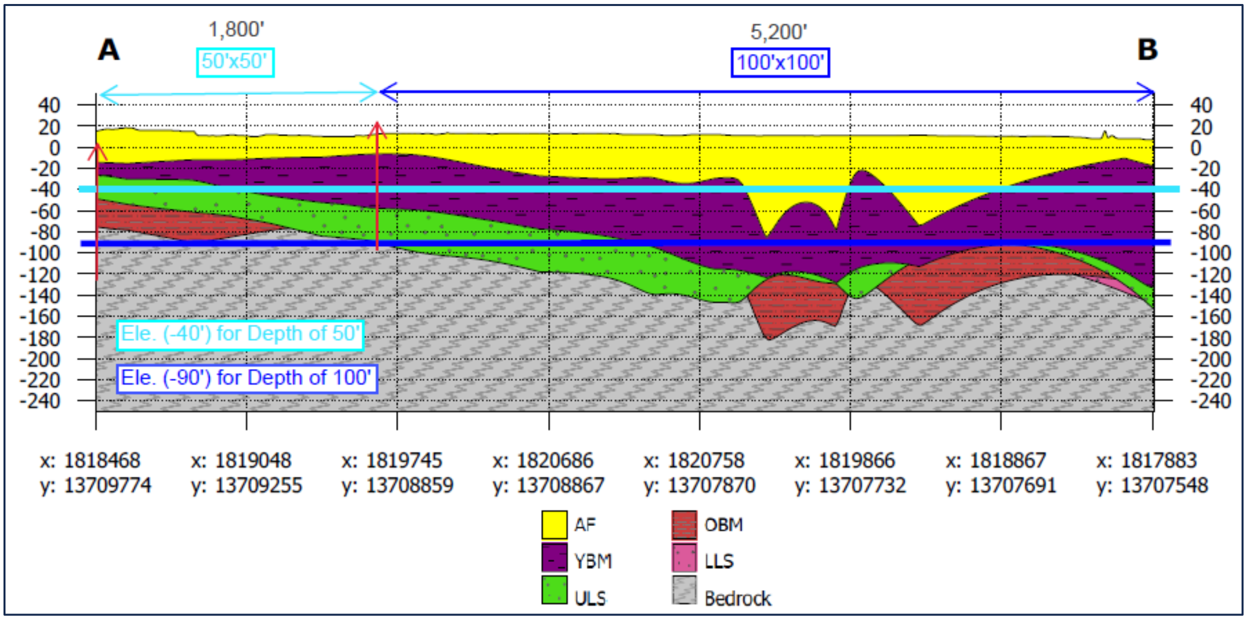


Figure B-29: SWF geologic profile with Ground Improvement segment 2-1 (A-B = north-south).

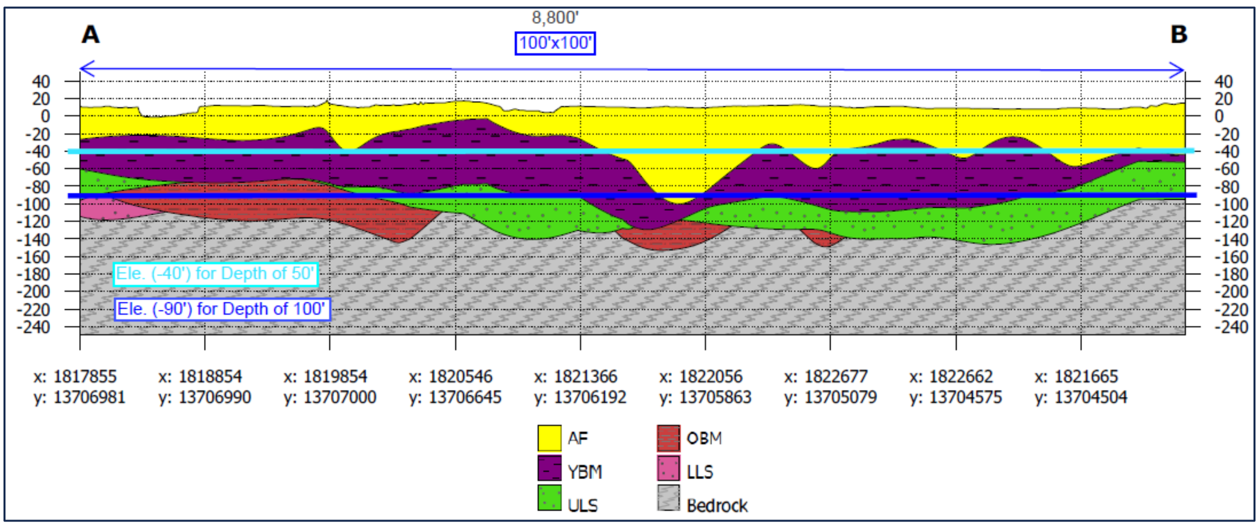


Figure B-30: SWF geologic profile with Ground Improvement segment 2-2 (A-B = north-south)

Areas designated as a Zone 1 have a general overall average thickness of AF and YBM of approximately 50 feet, while Zone 2 areas have a general overall average thickness of approximately 100 feet. The simplified assumption of two zone ground improvement depth dimensions of 50 feet and 100 feet were based on the approximate depth to competent bearing material (either ULS or OBM) and to provide a reasonable estimate of the overall volume of ground improvements necessary for development of this feasibility study cost and construction duration estimates. The PDT recognizes that some areas within these delineated zone reaches have combined thickness of AF and

YBM soils that are either a little less than or greater than the 50 feet or 100 feet assumptions; however, the zone reaches were determined with the intent that the overall average ground improvement volume in each reach would be reasonably captured (i.e. within a given zone some areas would require a little deeper ground improvement and some areas would be a little shallower).

In several of the initial project structural alternatives (Alternatives F and G) the line of protection retreated from the shoreline in areas of the 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 approximately 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 projected 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 panel 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-31**). The reduced depths were based on geologic information that indicates the AF and YBM thicknesses decrease as the distance from the shoreline increases.

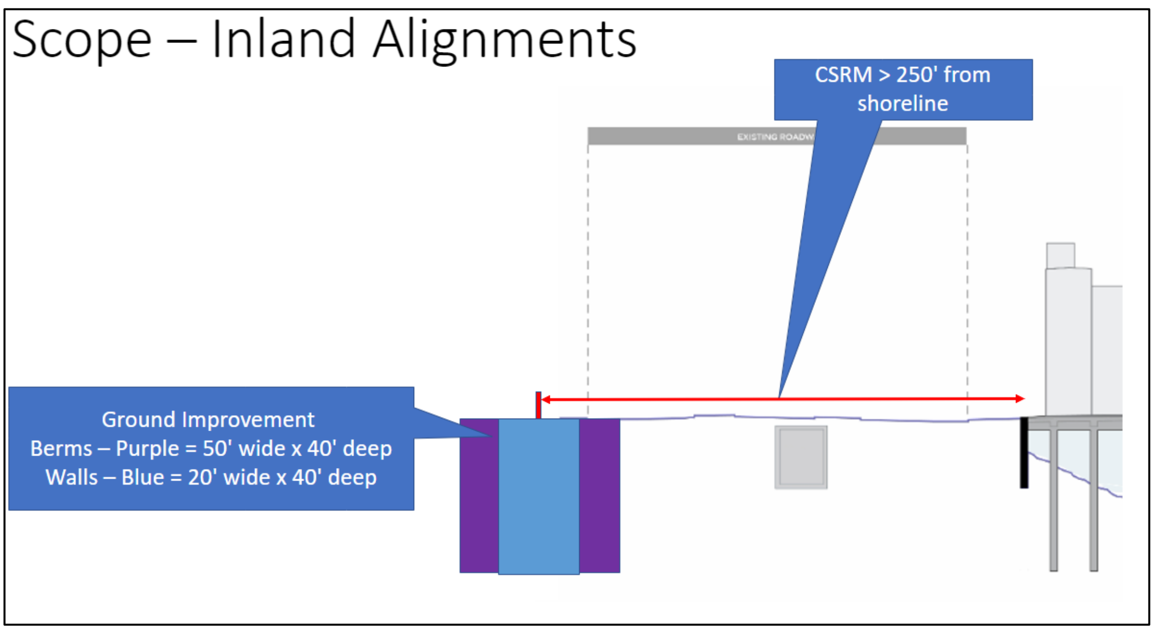


Figure B-31: Ground Improvement Requirements for Measures at Least 250 Feet from the Shoreline

1.3.1 Ground Improvement Engineering/Constructability Formulation

The PDT considered multiple potential solutions to provide a stable foundation for earthen and structural flood protection features for both static and dynamic load cases. Typical solutions consist of either a structural method (piles, shafts, etc.) or ground improvement method (deep soil mixing, jet grouting, permeation grouting, vibro-replacement, dynamic compaction, etc.). The PDT determined that the most appropriate and representative method to consider for the feasibility study was a deep soil mixing (DSM) buttress, with panel layout perpendicular to the shoreline alignment. The basis of this selection considered:

1) continuity of construction techniques and integration of a system that could satisfy both static and dynamic loading conditions in one design solution. This considered the fact that some of the initial flood protection alternatives evaluated had substantial amounts of fill placed in the bay which would require ground improvements for settlement control and DSM was the preferred solution for that condition,
2) review of recently designed and constructed private development projects located in the waterfront area which utilized DSM buttresses of similar proportions for both static and dynamic design solutions,
3) PDT's engineering judgment based on previous project experience for various design solutions with both static and dynamic loading conditions,
4) review of seismic hazard evaluations and preliminary designs recently performed by the Port's engineering consultants (reviewed by an independent expert review panel) for the waterfront which considered multiple solutions with DSM being identified with a high level of confidence that it would sufficiently provide a stable foundation, and
5) incorporating a level of uncertainty with a solution that the PDT judged would provide a reasonable feasibility level estimate of both construction duration and cost in the likely event that future designs change to an alternate design solution.

Formulation of the proposed coastal storm risk management measures were informed by several factors including urban design considerations along the historic urban waterfront with the intent of minimizing impacts on the future viewshed. The impact to public and private infrastructure systems such as buildings, public rail transit, roadways, and utilities were also considered. In almost all cases, the footprint disturbed by the project construction was governed by visual considerations related to the gradual grade change considering viewshed and not by the DSM buttress size.

Based on results from the Port's preliminary engineering studies, southern waterfront geotechnical analysis, designs for multiple private and public construction projects along the waterfront, and the PDT's engineering judgment, the current design estimates are based on utilizing DSM stabilization buttress panels 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) to stabilize the foundation and shoreline soil to be consistent with the ER 1110-2-1806 performance requirements for both the OBE and MDE ground motions and also static loading conditions. **Table B-1** provides the design parameter assumptions for the DSM buttress panels developed by the PDT in this study. These design assumptions are consistent with those utilized in other projects along the waterfront and in the general San Francisco Bay area (Oakland International Airport, Potrero Power Station Development, etc.). The DSM design methodology and requirements as provided in the USACE DSM design manual (dated April 2011) and the FHWA DSM design guide (dated October 2013) were utilized in the development of these initial design parameters.

Table B-1: Design parameters for the DSM buttress panels developed by the PDT

$UCS_{,DSM}$=	150	psi	*(Design strength of mixed material)*
Diameter=	3	ft	*(individual columns)*
Overlap=	9	in	*(overlap of columns)*
Panel Spacing=	9	ft	*(center-to-center spacing)*
a_s=	0.3	Table 2 of USACE DSM Design Manual	*(area replacement ratio)*
a_e=	0.15	Table 1 of USACE DSM Design Manual	*(overlap area ratio)*
	0.45	Total mixed area ratio, $a_s + a_e$	

***per USACE & FHWA design guidance, tensile capacity of the DSM panel is neglected.*

The PDT recognizes that the use of full width DSM buttresses may not be the final solution in all places along the project length, as delineated herein, due to subsurface conditions related to the rock dike in the NWF area. This uncertainty in final solution has been considered in the cost estimate by increasing the unit rate for DSM construction in the rock dike area. Alternate solutions or combined solutions (structural and ground improvements) may provide a more efficient construction solution, or the future designers may elect to straddle the rock dike area with ground improvements in the final design configuration.

The PDT also intentionally elected to neglect any potential resistance from the pile foundations supporting the existing finger piers. The rationale for this assumption is numerous and includes the following:

1) inclusion of these foundation elements in any design of future flood protection features would mean that those foundation elements could not be removed in the future which is a concern since the Port has estimated that most of the finger piers will be beyond their useful life by about the year 2040,
2) foundations would need to be maintained by the NFS for the design life of this project,
3) foundation elements are intermittent and not continuous along the extent of the waterfront and it was considered more advantageous to consider a uniform solution along the waterfront, and

4) a high level of uncertainty exists in estimating the available resistance capacity of these foundation elements given: a) the age of these foundation elements (~75 yrs +/-), b) material variability in the elements (timber, concrete, steel), c) uncertainty of pile tip depths, d) uncertainty of both initial and current material dimensions and properties, and e) developing reliable means of estimating any additional deterioration potential over the next 100-year project life given the adverse marine environment.

Considering the proposed ground improvement constitutes the largest single construction feature for this project, it is highly recommended that consideration be given to conducting test sections of the various methods (DSM, jet grouting, permeation grouting, etc.) in the AF and rock dike subsurface conditions to evaluate the effectiveness, constructability, and production rates of these methods. Performing these test sections early in the design phase will provide very valuable information to inform the design, construction schedule, and construction cost.

To supplement the PDT ground improvement design, the PDT arranged for a preliminary sensitivity analysis using finite element methods be conducted by the Sacramento District (SPK) for two typical sections to investigate alternate DSM buttress dimensions. The draft summary report prepared by SPK is attached to this feasibility report. For comparison purposes, the design assumptions developed by SPK for the alternate buttress dimensions is provided in **Table B-2**. The design strength of mixed material (UCS, DSM) and area replacement ratio (as) were assumed in the SPK evaluations, and the individual column diameter, overlap, panel spacing, and overlap area ratio values were estimated based on design tables in the USACE DSM design manual to achieve the assumed area replacement ratio. These results indicated the potential for reducing the panel width to approximately 45% of the panel heights compared to the PDT's estimate of equal panel height and width.

Table B-2: Design parameters for the DSM buttress panels developed by SPK and PDT (Table B-1) for comparison

Design parameters for the DSM buttress panels developed by SPK.

$UCS,_{DSM}$=	400	psi	*(Design strength of mixed material)*
Diameter=	3	ft	*(individual columns)*
Overlap=	18	in	*(overlap of columns)*
Panel Spacing=	6	ft	*(center-to-center spacing)*
a_s=	0.5	Assumed in design	*(area replacement ratio)*
a_e=	0.39	Table 1 of USACE DSM Design Manual	*(overlap area ratio)*
	0.89	Total mixed area ratio, $a_s + a_e$	

Design parameters for the DSM buttress panels developed by the PDT.

$UCS,_{DSM}$=	150	psi	*(Design strength of mixed material)*
Diameter=	3	ft	*(individual columns)*
Overlap=	9	in	*(overlap of columns)*
Panel Spacing=	9	ft	*(center-to-center spacing)*
a_s=	0.3	Table 2 of USACE DSM Design Manual	*(area replacement ratio)*
a_e=	0.15	Table 1 of USACE DSM Design Manual	*(overlap area ratio)*
	0.45	Total mixed area ratio, $a_s + a_e$	

***per USACE & FHWA design guidance, tensile capacity of the DSM panel is neglected.*

The PDT recognizes the optimization of DSM buttress ratios will be a critical step during design and expects components of the sensitivity analysis to inform refined designs during that phase. The following list provides relevant information noted when comparing the PDT's estimated parameters and the sensitivity analysis which should be considered during the design optimization phase.

- **Figure B-32,** from the USACE DSM design manual (dated April 2011), shows that increasing the design strength from 150 psi to 400 psi will require approximately 3.5 times more cement binder leading to a higher unit cost (2x unit cost per oral communications with specialty contractors), increase mixing time, and additional spoils.
- Per the USACE DSM design manual (dated April 2011) and the FHWA DSM design guide (dated October 2013), it is difficult to consistently achieve the required DSM design strength greater than about 250-300 psi. Selection of the final design strength in the design phase should consider constructability issues and reliability of the contractor to consistently achieve the required strengths within the allowed variability.

- Both the USACE DSM design manual (dated April 2011) and the FHWA DSM design guide (dated October 2013) require the use of a factor of 0.8 be applied to the composite DSM design strength to account for the reduction from peak unconfined strength to residual confined strength in consideration of strain compatibility issues with unimproved soils. Applying this factor to the sensitivity analyses would equate to increasing the initial design strength from 400 psi to 500 psi to maintain the composite strength used in the analyses.
- The replacement ratio of 0.5 assumed in the sensitivity analysis will require closer spaced panels (6 feet vs 9 feet center-to-center spacing for 3 feet diameter columns) and greater overlap of columns than assumed in original plan formulation to achieve this replacement ratio (i.e. increased double mixing/dosing of about 40% of each panel volume). The closer panel spacing, combined with the greater overlap, will equate to a greater volume of mixed soils (quantity used to determine cost) even with the narrower width and slightly deeper panels proposed in the sensitivity analysis.
- The use of tensile capacity in the DSM panels varies across design manuals, but typical USACE and FHWA design guidance recommends neglecting tensile resistance in the panels.
- The sensitivity analysis indicated system displacements of about 9 inches were acceptable when subjected to the OBE ground motions, which was based on the criteria presented by the NFS's consultant evaluations of non-flood protection infrastructure. Per ER 1110-2-1806, the performance criteria for the OBE evaluation are little to no damage and continued function without need for extensive repairs. Future risk assessments should be conducted during design to establish reasonable displacements consistent with the OBE performance criteria for both earthen and structural flood protection measures.
- Future analyses should consider internal panel stability issues of crushing of the DSM panel toe, shearing on vertical planes within the panels, and extrusion of soft ground between panels.
- Additional sensitivity analyses should be conducted in the design phase to consider tradeoffs in DSM material strength, panel configuration, constructability issues, and other impacts to the project (real estate, viewshed, utilities, environmental, etc.).

Consistent with the PDT evaluations, the sensitivity analysis draft report also suggested consideration of other mitigation alternatives including structural or ground improvements both as single solutions or in combination in various areas of the project that could provide a more cost-effective solution than DSM panels alone. These alternatives should be further evaluated during the design phase to optimize how the coastal defense system meets performance requirements. For this feasibility study, the DSM buttress alternative provides a consistent solution with a high likelihood of meeting performance requirements and quality objectives, and an increased confidence in providing a reasonable feasibility level construction duration and cost estimate.

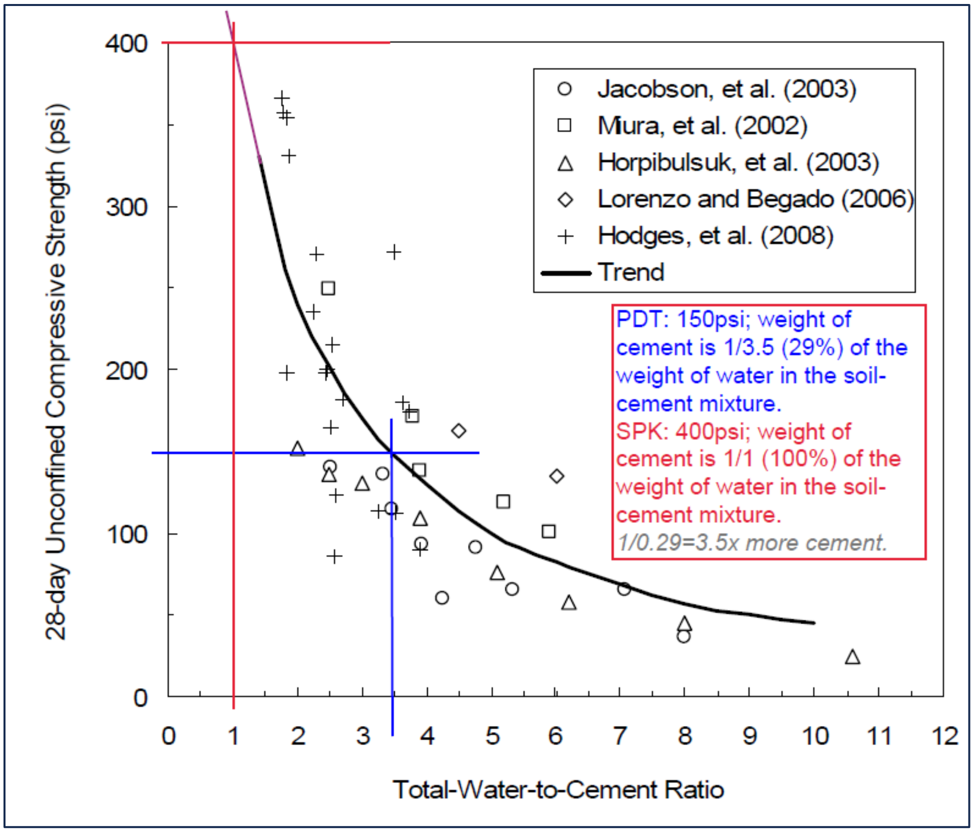
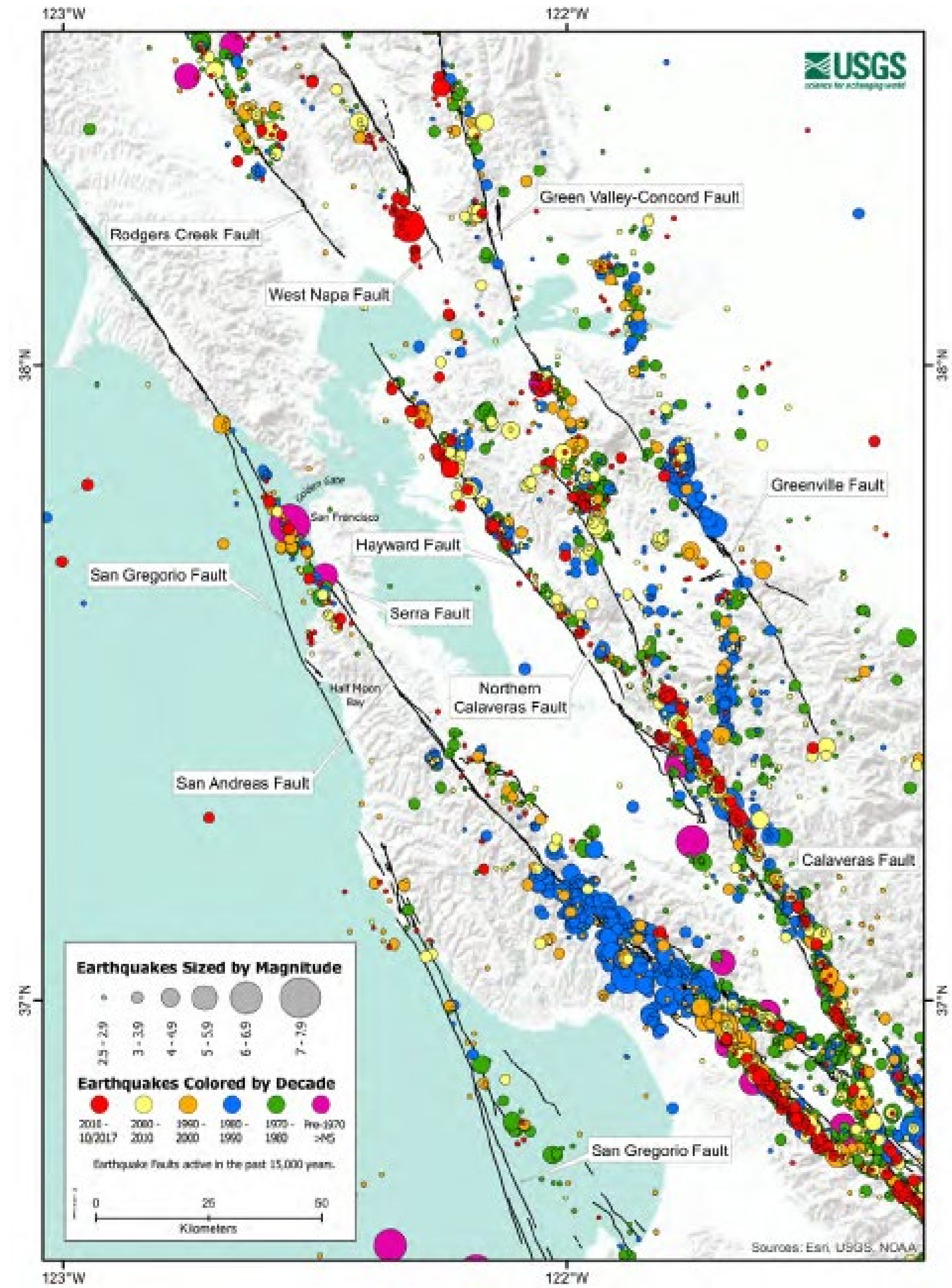


Figure B-32: Design Strength Parameter Assumption

1.4 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-33**) 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-33: 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, paleo-seismological 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-34**). 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%) is the most likely to produce a large earthquake damaging to San Francisco over this time. **Table B-3** 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-3: 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-34: 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 2024). 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 has a minimum ground motion with a 50% probability of exceedance in 100 years (144-year return period). Higher ground motions can be considered for the OBE as determined in the design phase. 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. Adaptation is a key component of the formulation strategy such that foundation elements included as part of the First Actions are expected to be in service for 100 years or more, therefore resulting in a high likelihood that an OBE level event occurs during the design life. 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 should have a minimum 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. An initial life safety risk assessment was performed during the feasibility study phase that confirmed this assumption, however, further design detail will be required to complete the formal life safety risk assessment during the design phase to confirm the classification of the flood control measures. In the project vicinity, the mean deterministic MCE is approximately equal to the 975-year return period event, therefore there will not be a substantial difference in the MDE for a normal or critical structure.

Ground motion information was obtained from the USGS Earthquake Hazard Toolbox website based on the 2023 USGS seismic hazard model results for the project site (see **Table B-4**). The PGA values are based on an estimated site class of DE with a Vs,30 value of 185 meters per second per available information in the waterfront area. The Performance Range column is provided in the table to indicate typical return period ground motions corresponding to the various design earthquake levels dependent on final classification of project features as normal or critical and economic considerations. These acceleration values indicate significant ground motions can be expected at the project site, even for the OBE event defined at a return period of 144 years.

Table B-4: 2023 USGS seismic hazard results for the project site

Return Period (yrs)	Site Class	PGA (g)	Performance Range	M_w Mean
145	DE	0.335	OBE	7.0
225	DE	0.398	OBE	7.1
475	DE	0.498	OBE	7.2
975	DE	0.614	MDE/MCE	7.2
2475	DE	0.756	MCE+	7.3
10000	DE	1.012	MCE+	7.3

+ - higher epsilon values above mean MCE

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-35** 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.

Figure B-35: California Geological Survey's Seismic Hazard Zones Map Showing Zones of Liquefaction and Earthquake-Induced Landslides

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 will be a major design effort in the design phase. The current seismic design mitigation methods are judged to be a reasonable, feasibility-level design that will satisfy the requirements of ER 1110-2-1806 (USACE 2024) and provide a suitable quantity and cost estimate for this feasibility study which the PDT expects will be refined and optimized in the design phase.

1.4.1 Subsidence

The primary contributors to vertical movements for this project are plate tectonic associated with the plate boundary within which the site exists and also traditional geotechnical consolidation settlement. These contributing effects need to be considered when determining relative sea level change potential and 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 (RSLC), M is a constant for the global RSLC (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 RSLC to assumed values RSLC of 0.5, 1.0, and 1.5 meters by the year 2100, and t is time relative to the year 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 PDT surmises that the contribution of tectonic plate movement to relative RSLC 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 RSLC 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.

All three approaches have been adopted at this phase of the study, but these will be refined during the design phase based upon further subsurface exploration, analyses, cost and consideration of construction means and methods.

1.5 Numerical Models

1.5.1 Hydrology, Hydraulics and Coastal

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

1.5.2 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 will increase the potential of interior flooding. To protect the area from flooding, new pumps and gravity flow structures are expected 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, modeling and interior drainage scope for the Recommended Plan.

2.0 Existing Conditions

2.1 Shoreline Structures

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).

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.

2.1.3 Timber Piled Shoreline Wharves

The timber piles wharves are in two types. The first type consists of 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.

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.

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.

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.

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.

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.

2.2 Utility Systems

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.

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-36**). The system is also essential for drainage for CCSF. Together, the collection system (with its storage capacity) and outfalls prevent flooding of public streets, sidewalks, parks, and buildings.

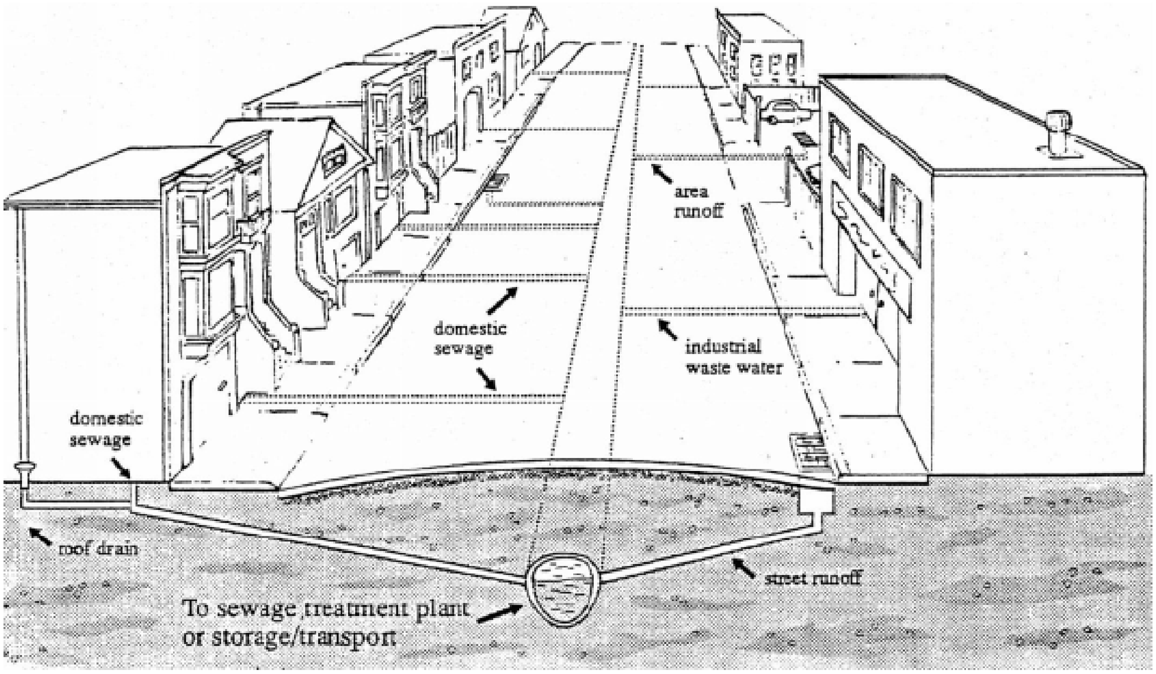


Figure B-36: Representation of Combined Sewer System

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.

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 design phase.

2.2.4 Low-Pressure Water (LPW)

The SFPUC WE operate the Hetch Hetchy Regional Water System (**Figure B-37**), 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).

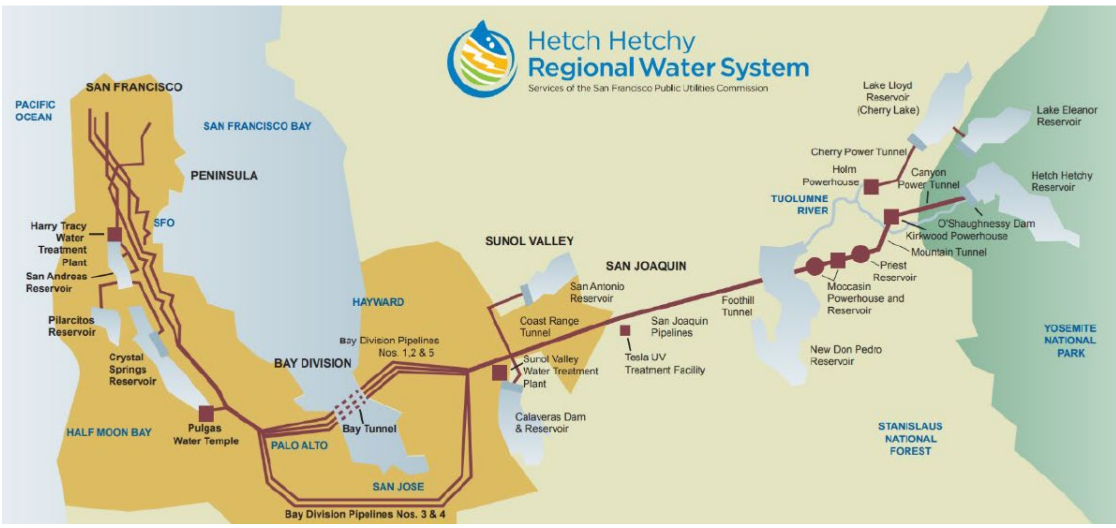


Figure B-37: Hetch Hetchy Regional Water System

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).

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).

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).

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 includes 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).

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).

2.2.9 Utility System Quantity Estimate

To estimate the scope of Study phase impacts to existing infrastructure and develop a Class 4/5 cost related to relocation and protection of the existing utility systems within the project footprint, the PDT used available GIS database information provided by stakeholder agencies and publicly available information (Google Earth/Maps) to lay out proposed relocation and protect-in-place various infrastructure systems. The intent was to maximize secondary lines direct to service locations to the maximum extent practicable while relocating primary feed lines to centralized utility corridors within readily accessible roadway and median alignments. The benefit of this strategy is a reduction in future building/facility closures, expandable corridor footprint, and reduction in full roadway closure in favor of single-lane traffic control. No field validation of the existing utility systems or detailed investigation of as-built drawings within the study area were performed during the feasibility study, therefore a high level of uncertainty remains with respect the affected utility systems. As such, the cost and schedule risk should be considered as significant risk with a high level of certainty that schedule and cost will be impacted until further field verification can be conducted

2.3 Mobility Systems

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. For scoping the Recommended Plan, transportation systems are recognized in order to consider the constructability and systematic impacts that the plan may have outside of the immediate footprint. This influences temporary access and detour requirements that will be incurred as part of constructing the plan.

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).

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).

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).

3.0 Engineering Evaluation

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 watersheds were the driving factors in establishing the reach boundaries, noting that these boundaries are not discrete and/or separable in all instances. **Figure B-38** 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 29.
- 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.

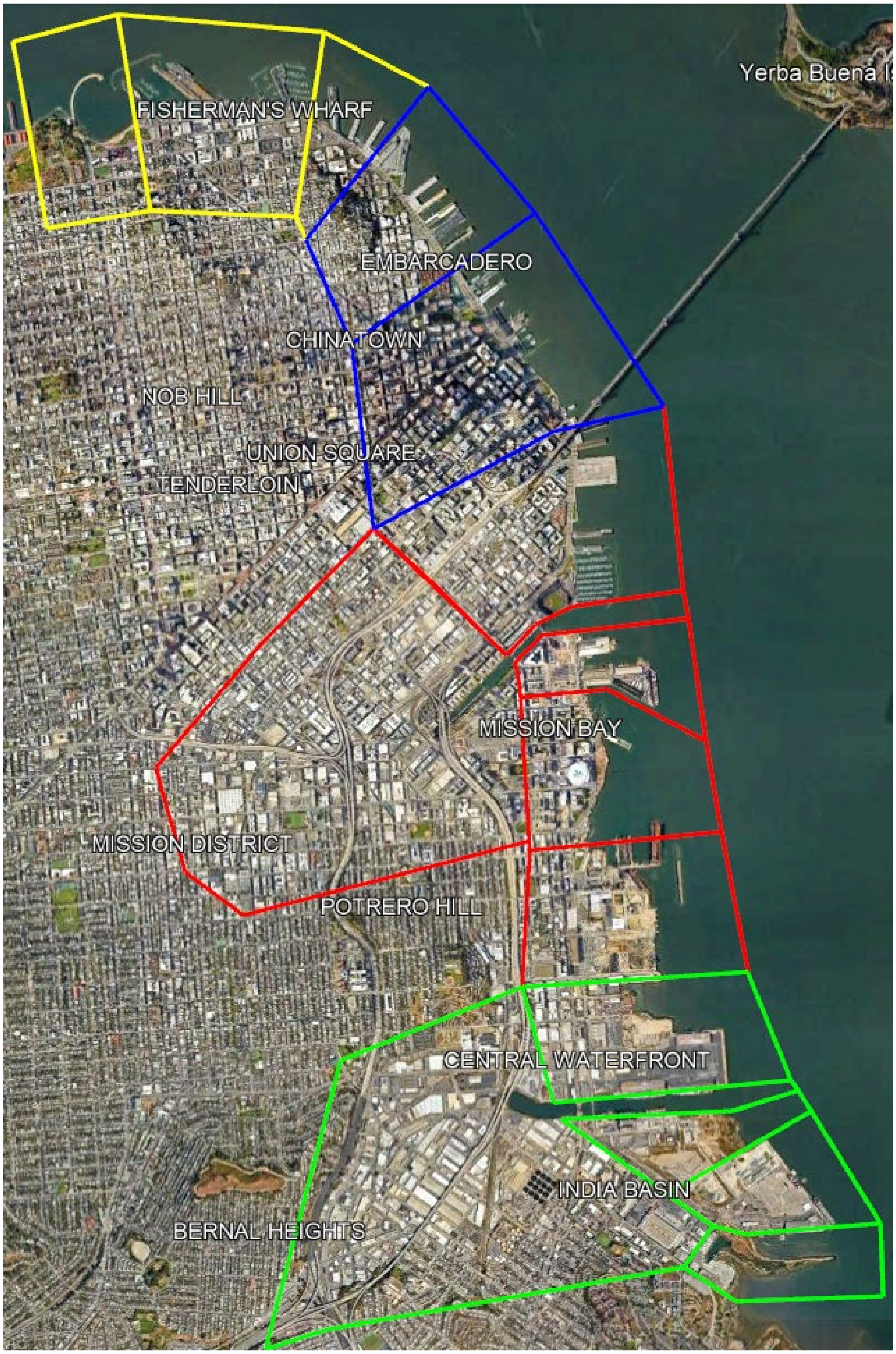


Figure B-38: Map of Four Reaches

3.2 Alternative Development & Alignments

Due to the uncertain nature of RSLC, plan formulation was conducted in a manner that spanned the range of potential RSLC projections. Alternatives were developed that targeted lower, moderate and higher levels of RSLC and utilized distinct RSLC values and a set adaptation horizon midway through the period of analysis. This allowed for relative comparison of alternatives, without the need to optimize all alternatives for each RSLC curve. During plan formulation 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 alignments 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.

This study has also considered the beneficial use of several proposed or in-construction private developments and city parks in the SWF (see **Figure B-39**) which will be raising the ground surface to meet current City of San Francisco RSLC planning requirements which are consistent with our proposed initial coastal flood control elevations. Since the ground surface of these private developments and city parks will meet our planned flood protection design elevations, we have assigned these areas to be omitted from requiring flood control measures and our new structural flood control measures will tie into these high ground areas. Future design phase evaluations should determine these sites are consistent with the final coastal hydraulic design elevation requirements and comply with the requirements of other applicable USACE Engineering Regulations and Engineering Manuals.

Figure B-39: Map of future or in-construction private developments or city parks

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.

In the design phase, it is anticipated further refinements to various components in the current design will be developed for the purpose of optimizing real estate requirements for the project which will be beneficial to the Federal Government, the local sponsor, and the citizens of San Francisco.

3.3 Setting Crest Elevation

The crest elevation of the alternatives was informed by the coastal engineering analysis and plan formulation approach that identified a range of RSLC targets for evaluation of multiple alternatives across the 100-year period of analysis. To set crest elevations and utilize an adaptation approach, the alternatives were categorized into those targeting higher or lower RSLC projections. Alternatives C and D were biased toward the lower

RSLC projections while Alternatives E, F and G were biased toward the higher RSLC projections. The first project action is assumed to be constructed around year 2040, then be adapted in the year 2090 to address actual RSLC. As such, the crest elevations are tied to a stepwise application of RSLC based upon the targeted RSLC projection. Lower RSLC targets assumed 1.5 feet of RSLC built into the first 50 years, while the higher RSLC targets assumed 3.5 feet of RSLC built into the same period. Crest elevation is dependent upon the application of these RSLC values in combination with a stillwater level (SWL) and approximated wave run-up value. SWLs are a function of the storm event return period, with approximately 2 feet difference between the annual event and the 1% annual exceedance probability (1% AEP or 100-yr storm) event for this project site. Due to the relatively small difference in elevation between these events, the 1% AEP SWL elevation was utilized in defining the crest elevation. The stillwater level also varies by approximately 6 inches across the project length; however, to simplify possible variabilities during the feasibility study the maximum SWL for the 1% AEP was applied to the entire project length. Finally, the contribution of wave run-up to the water surface was estimated based upon the results of the wave sensitivity evaluation that utilized paired historic water level and wave information from the FEMA study as presented in Appendix B1.3. The estimated freeboard for the effective wave action above the 1% AEP SWL is estimated as 2 feet, recognizing that larger isolated waves and associated runup is known to occur along the shoreline. For the feasibility study and evaluation of economic justification, the crest elevation is set in a manner to target protection from a 1% AEP coastal event, where 1% wave hazards do not occur concurrent with 1% AEP stillwater levels. **Figure B-40** shows the stepwise application of RSLC based upon the adaptation approach and **Figure B-41** lays out a flowchart to arrive at the assumed crest elevations. The PDT expects that further coastal engineering analysis and data collection will be performed in the design phase, such that crest elevations will be refined to ensure the constructed coastal flood defenses mitigate risk from the 1% AEP coastal event for the expected service life of the infrastructure prior to adaptation.

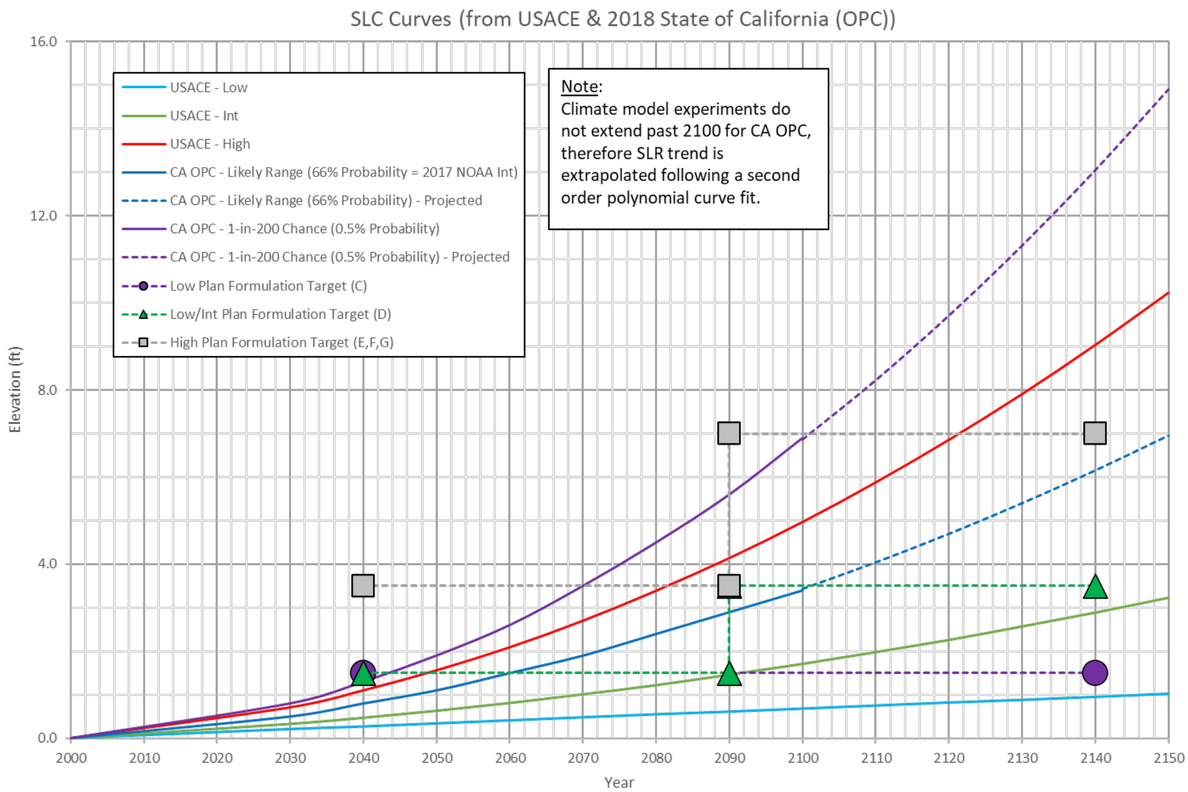


Figure B-40: Plot to show RSLC and Application of Targets for Plan Formulation

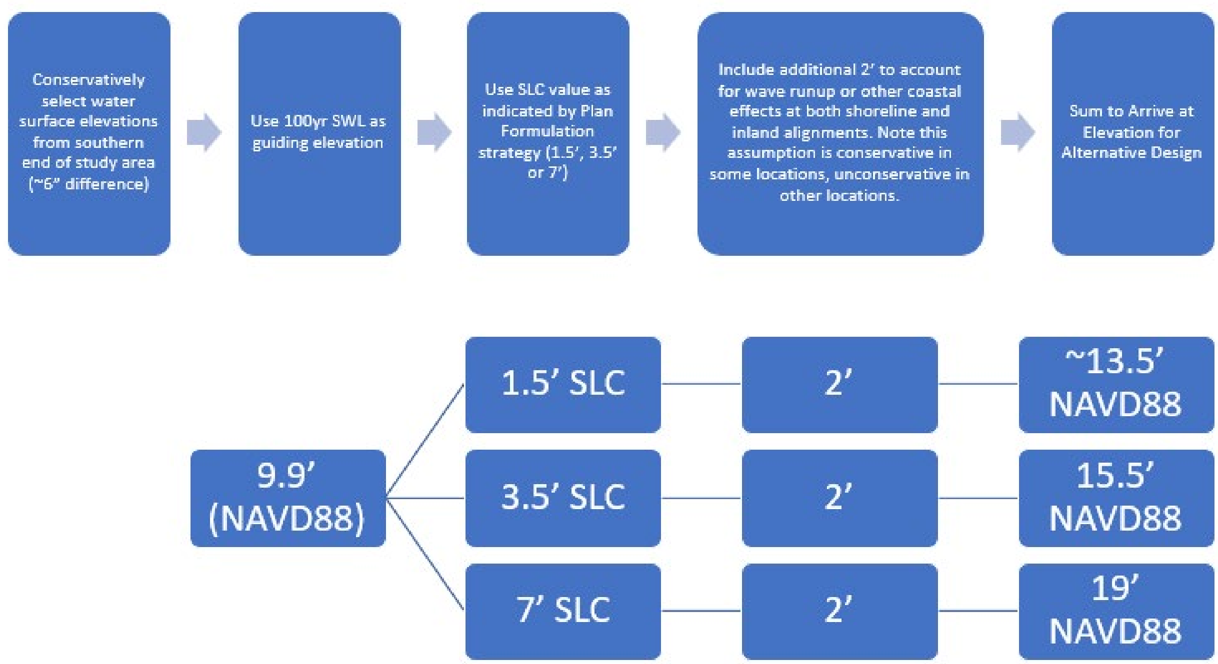


Figure B-41: Flowchart showing application of water level components and assumption to set crest elevation

3.4 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. The following sections detail the measures utilized to build and evaluate each of the alternatives. 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 design phase. The geotechnical and structural aspects of the various feasibility study measures are discussed below.

3.4.1 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 (**Table B-5**). During the design phase, all levees should be designed in general accordance with Engineering Manual (EM) 1110-2-1913 *Design and Construction of Levees* (USACE 2000 or current version at time of PED). A typical levee is shown on **Figure B-42**.

Table B-5: 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

Levee Height (feet)	10-Foot Crest 3H:1V Total Width (feet)	20-Foot Crest 3H:1V Total Width (feet)
7	52	62
8	58	68
9	64	74
10	70	80

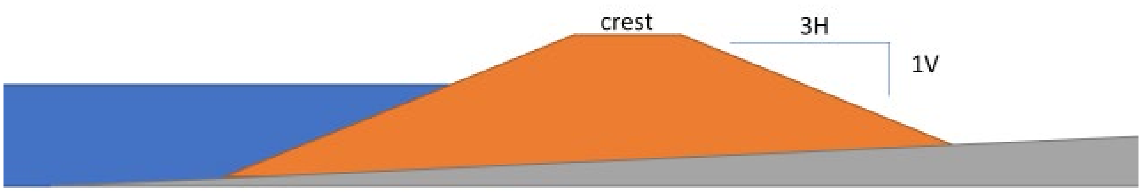


Figure B-42: Typical Cross Section of Levee

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 design phase geotechnical site investigations will need 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 critical seepage issue. The seepage barrier was assumed to be 20 feet long vinyl sheet piles extending several feet up into the levee section and driven along the length of the levees. The need to overbuild levees for potential consolidation settlements should be considered in future designs when determining the final levee sections for construction.

3.4.2 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-43**). In the isolated areas where real estate is not restrictive, levees (ecotone levees) were included in the Recommended Plan with these typical dimensions of flood side slopes and vegetation (**Table B-6**). Future designs will need to reconcile the fact that the water surfaces in this project are driven by RSLC and as such the assumed marsh conditions will not exist throughout decades of this project while the anticipated sea level 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*.

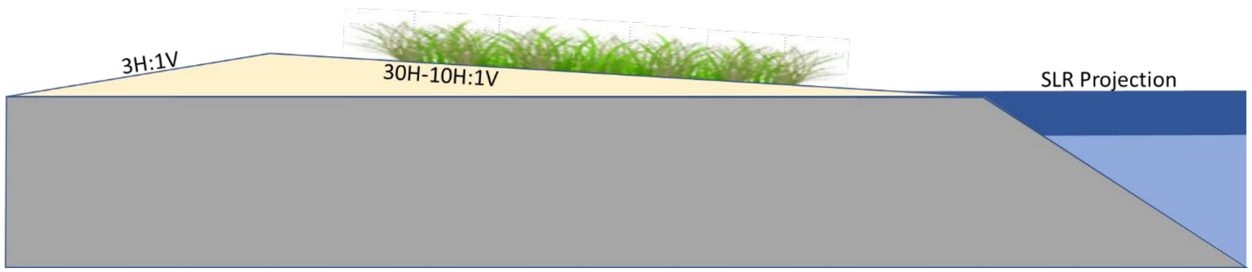


Figure B-43: Typical Cross Section of Ecotone Levee

Table B-6: 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

3.4.3 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 RSLC 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 RSLC 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 the design phase, 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 adaptation phase would be mandated as RSLC 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 the functioning of the overflow interior drainage system.

The water management structures are assumed to be supported on deep foundations where the structures' bases (or top of deep foundations) are at approximately elevation –25 to –30 feet, based on existing creek bottom elevations. Under seepage mitigation measures under the structures will be provided as determined in the design 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.

3.4.4 Curb Walls (Curb Extensions)

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 mitigate risk to these structures is to add approximately 2 feet of elevation with a perimeter curb wall (curb extension) around the piers. Gate structures were also included with these measures to allow access by vehicles or pedestrians during non-flood periods. Depending on actual RSLC over the project life, this flood protection measure will complement the main line of flood protection and temporarily extend the life of the existing infrastructure before more extensive measures (raising or abandoning) are required for the piers. Given their height and the design water levels, these curb walls are assumed to predominately only provide protection from wave action (2-foot design wave height) acting on the piers. Because these curb walls are not the main line flood protection for this project, we have not assumed they will require the same seismic performance requirements, including ground improvements to the existing pier foundations. It should be noted that not all existing pier structures will be suitable for installation of the curb wall and alternative risk reduction measures, such as dry floodproofing of the pier shed buildings, will be evaluated on a site-by-site basis in PED. This uncertainty in design scope has been included in the CSRA.

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. In addition, upliftof the pier deck structures as a result of wave action should be included in analyses. Water from precipitation or overtopping may be trapped temporarily behind curb walls, such that the use of pumps or redirection of water will be explored during PED. The potential cost of this additional scope has been included as a risk in the CSRA. 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-44**.

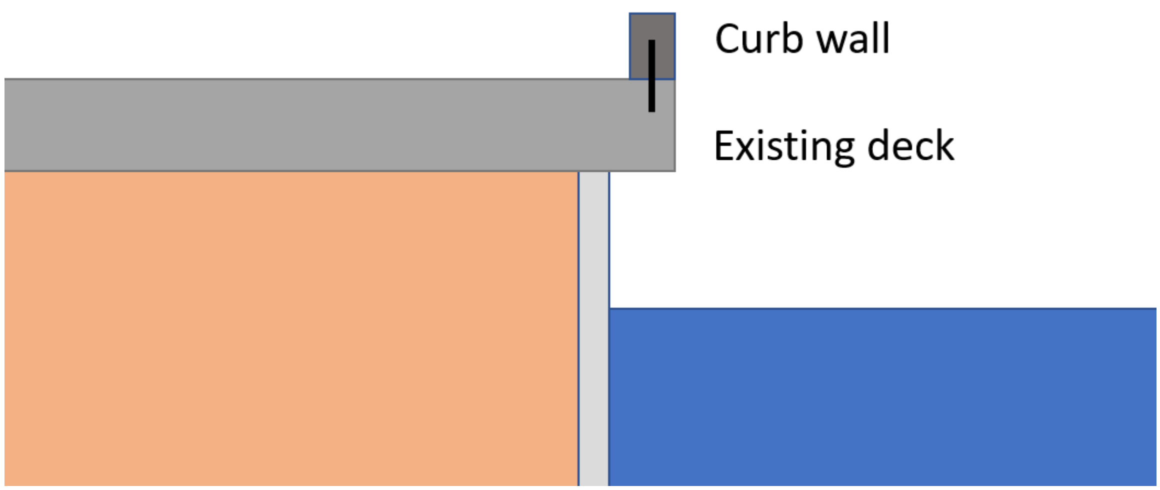


Figure B-44: Example of Curb Wall

3.4.5 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 2022 or current version at time of PED).

3.4.6 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-45.** 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 under seepage 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 Design phase. T-walls and L-walls should be designed in accordance with EM 1110-2-2502 *Retaining and Flood Walls* (USACE 2022 or current version at time of PED).

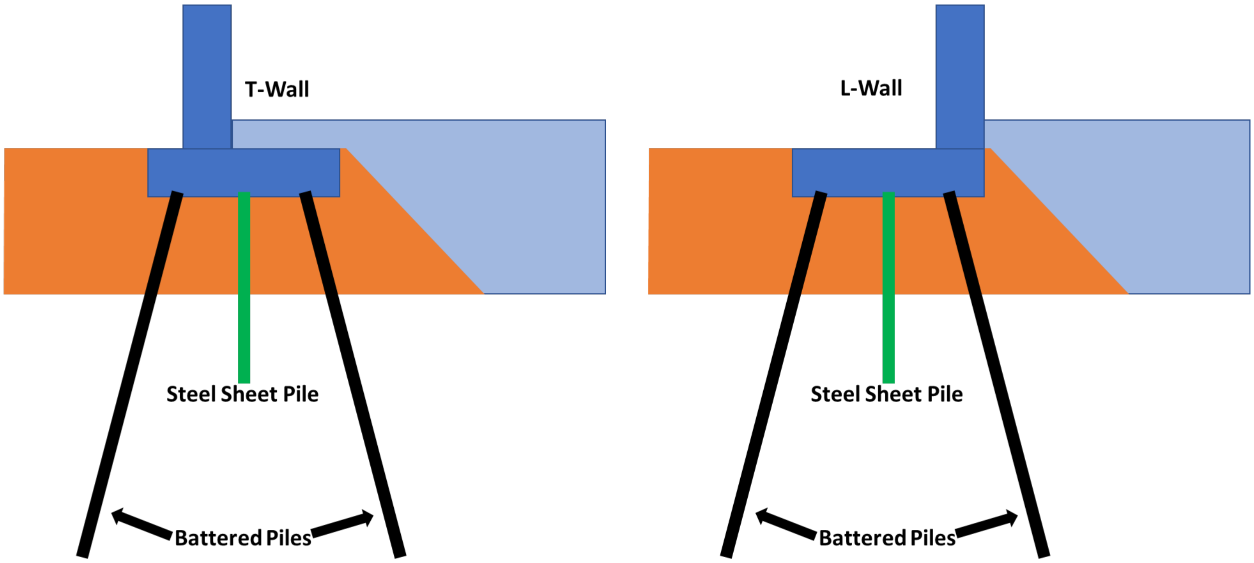


Figure B-45: L and T-Walls

3.4.7 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 the 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 can be refined in the Design phase if this method of construction is selected; although, it is noted that other flood control concepts will likely be reconsidered in future designs, such as cellular coffer cells. A general concept of this measure is shown on **Figure B-46**.

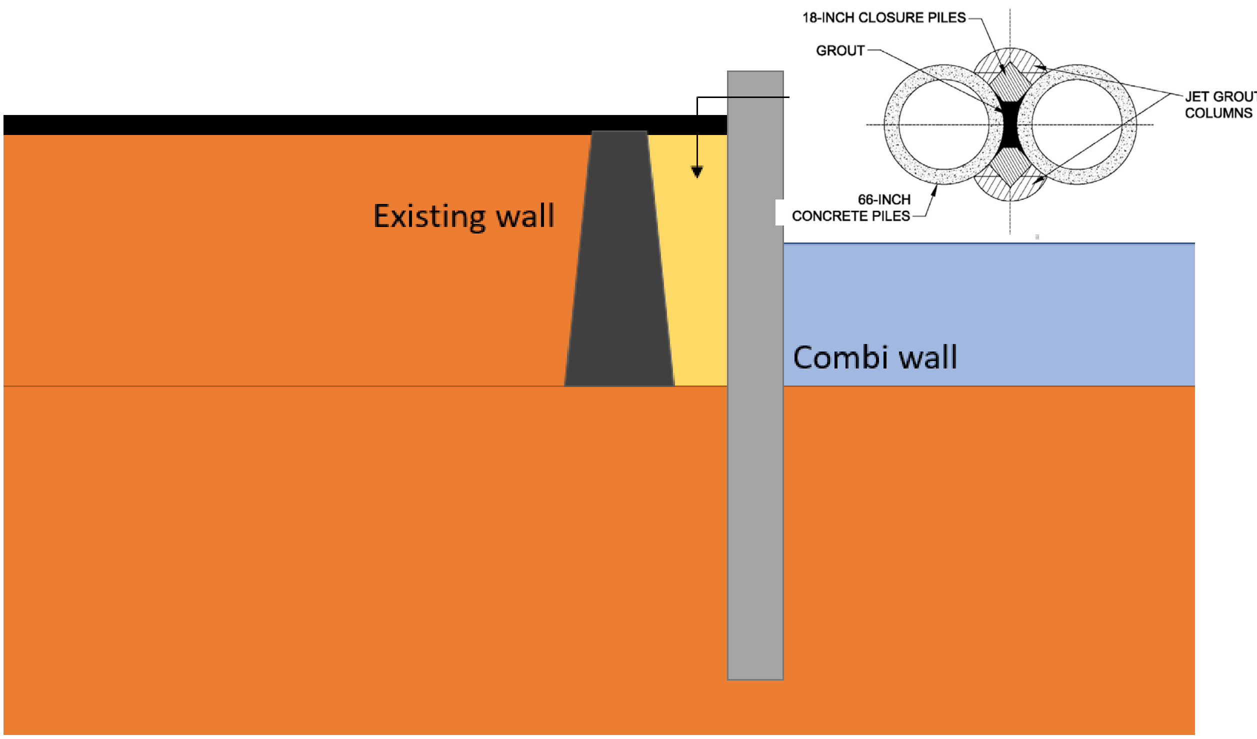


Figure B-46: Example of Combi-Wall

3.4.8 Tieback Retaining Wall

Existing tieback retaining walls exist along the south berth of Pier 96 to allow for berthing of vessels adjacent to the existing pier and warehouse structures. These existing structures are steel sheet pile walls supported by one row of tiebacks with a waler beam which are anchored to a deadman block. In these locations the PDT has recommended that a new tieback wall be constructed waterward of the existing wall using similar construction materials and wall type. The new sheet pile wall will be constructed approximately 5-10 feet waterward of the existing wall and can be tied back to the existing deadman block since the height of retained soil will not increase even though the wall height will increase by several feet to accommodate the new flood height protection elevation. The area between the existing wall and the new wall will be backfilled with a granular material. It is assumed that ground improvements will be needed to minimize consolidation settlements associated with this new fill.

3.5 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 the 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 the passage of pedestrians and vehicles.

3.5.1 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.

3.5.1.1 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-47**.

Source: courtesy of Flood Control International

Figure B-47: Typical Swing Flood Gate

3.5.1.2 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-48**)

Source: courtesy of Flood Control International

Figure B-48: Typical Roller Gate

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

3.6 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 relative sea level change) 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.

3.6.1 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-49**.

Figure B-49: Bayou Bienvenue and Bayou Dupre Sector Gates

3.6.2 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-50**.

Source: courtesy of Jacobs Engineering Group

Figure B-50: Queensborough Barrier, UK at Low Tide

3.7 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 the design phase.

3.7.1 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.

3.7.2 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-51**.

Source: courtesy of USACE Norfolk District

Figure B-51: Flap Gate

3.7.3 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-52**, 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-52: Pair of Sluice Gates

3.8 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 relative sea level change Vulnerability Zone. The relative sea level change 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 relative sea level change occurs in the future.

The 12 pump stations located within the study area are listed in **Table B-7** and shown on **Figure B-53**, 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-7: 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

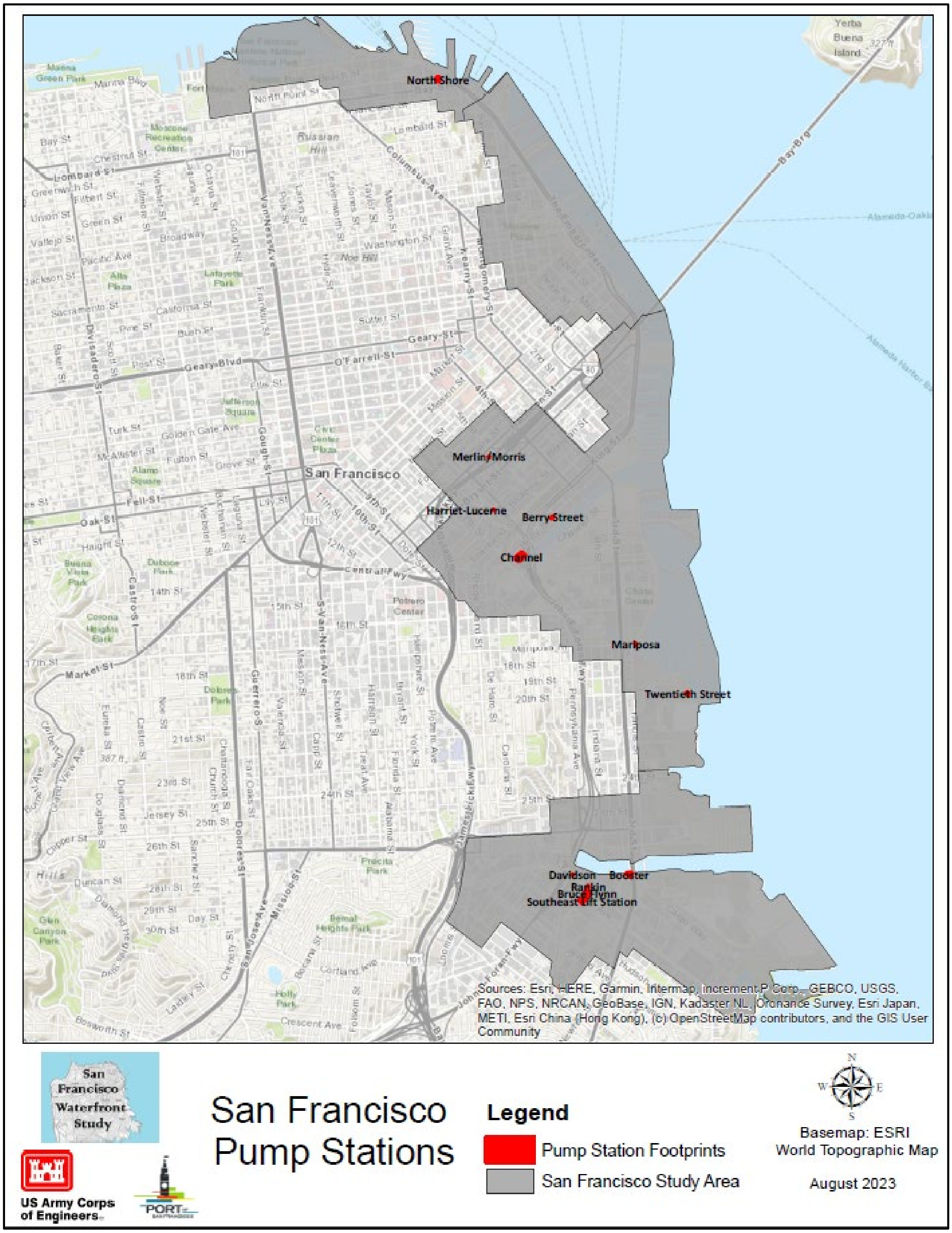


Figure B-53: Location and Names of Pumps in the Study Area

For this study the coastal flood protection system is assumed to be implemented in two phases over the 100-year project life. A combination of pumps, culverts, and isolation structures in Reach 1 for the combined sewer system were identified as part of the Recommended Plan. Pump sizing was evaluated using a HEC-RAS 2D overland flow model. Pumps were placed in areas that naturally ponded to validate sizing for the minimum facilities and stormwater runoff elements. Storm inlets, storm pipes and storage box manifolds were used to covey the runoff from the ground surface to the pumps which evacuated flows directly to the bay. This is a standalone system that does not interact directly with the combined sewer. For additional detail on the interior drainage analysis and pumping requirement for the future with project alternatives see Sub-Appendix B.1.4.

4.0 Design Considerations and Constraints

There are a host of design considerations and constraints that influence the way in which the alternatives were formulated, measures designed, and costs estimated. The following subsections outline several of these based on either discipline or major topic.

4.1 Civil Design

Civil Design efforts will be extensive for this project during the Design phase to properly lay out 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.

4.2 Mechanical and Electrical Requirements

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

4.3 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.

4.4 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. We currently have no plans to incorporate the use of dredged material from the bay into any proposed NNBF or EWN project features. 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*.

4.5 Corrosion Mitigation

This project is being constructed along the shoreline in a heavily corrosive environment. Therefore, during the design phase, 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.

4.6 Resiliency and Adaptability

Due to relative sea level change and the harsh marine environment along the waterfront, measures will be taken to ensure the line of protection can adapt to the changing hydrological and meteorological 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 Design 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

4.7 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 the design phase, 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.

5.0 Plan Selection Process

A total of 7 alternatives were developed for the San Francisco waterfront which consisted of a no-action alternative, a nonstructural alternative, and 5 structural alternatives (formulated to address various RSLC 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 RSLC associated with the low USACE relative sea level change 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 Recommended Plan.

5.1 Tentatively Selected Plan

The Tentatively Selected Plan (TSP) presented in the Draft Integrated Feasibility Report and Environmental Impact Statement was released to the public for review in January 2024. The plan that maximizes total net benefits, the Total Net Benefits Plan (TNBP) was selected as the TSP. It 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 TSP 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 plan formulation appendix of the Integrated Feasibility Report and Environmental Impact Statement.

5.2 Agency Decision Milestone & Recommended Plan

Following public and agency reviews (Agency Technical Review, Policy Review, Independent External Peer Review, etc.) the PDT identified refinements to the TSP that would mitigate deficiencies identified by these reviews. These refinements were described at the Agency Decision Milestone along with the proposed course of action to complete the feasibility study. Vertical team decision makers accepted the proposed refinements and therefore actualized the milestone enabling the PDT engineering team to continue efforts to improve the design maturity of the TSP with proposed refinements to produce the Recommended Plan.

5.3 Recommended Plan Development

The Recommended Plan builds upon the work completed for the Draft Integrated Feasibility Report, with the objective of clearly identifying the scope of construction activities such that cost, schedule and risk matrices can be developed to a level of maturity compliant with Guidance on Cost Engineering Products update for Civil Works

Projects in accordance with Engineering Regulation 1110-2-1301 dated 05 June 2023. To develop the scope of the recommended plan, the engineering team:

- identified specific geotechnical ground improvement solutions based on geotechnical zones as described in Section 1.3 Ground Improvement Footprint.
- divided the waterfront into 17 discrete cross sections the capture the full range of shoreline measures and existing conditions.
- at each section, defined the existing conditions and major scope of work items to build out the Recommended Plan.
- spatially locate the typical cross sections along the project alignment and extrapolate to plan view representation of project measures.
- develop quantities at the section level based on target and existing elevations, adjacent infrastructure constraints, and assumed design features for each type of measure (i.e. slopes, crest width, etc.).
- extrapolate quantities along the project alignment using the plan view representation provided in Appendix B.5 and understanding of cross-sectional values.

Scope of work items and sizing of measures for the Recommended Plan are based on relevant project designs from across USACE portfolio, recent project examples completed by the Port of San Francisco and other stakeholders along the San Francisco Waterfront, and engineering judgment based on prior experience. Key assumptions relevant to each cross section are listed in the following subsections and depicted in **Figure B-54** through **Figure B-70**. The PDT engineering team expects these assumptions, and proposed techniques will be further evaluated, analyzed and designed through the design phase while intent of the plan remains consistent with what has been provided in this feasibility study. A plan view representation that shows the approximate footprint and measures of the Recommended Plan can be found in Sub-Appendix B.5. The location of the sections described as follows are shown in Sub-Appendix B.5.

5.3.1 Section 1 – Typical Northern Embarcadero Seawall, Wharf and Bulkhead Building with Zone 1 Soils

- **Embarcadero Right-of-Way:** Gradual shoreline elevation across full width of Embarcadero right-of-way to ensure elevation gained at seawall alignment meets requirements for accessibility along and across the coastal flood defenses, while mitigating negative environmental impacts. Utilize engineered fill to make up grade between existing and finished elevation. This includes replacing roadway in-kind, elevating the light rail corridor located between north and south travel lanes as well as landscaping corridor to replicate existing.
- **Utilities:** Existing utilities to be relocated within the Embarcadero right-of-way to permit installation of ground improvement and over-burden compensating measures as required. Assume 5 typical utility systems, not including stormwater, run parallel to the alignment and will require relocation. Assume each system will

require a connection across the alignment (perpendicular) for each operational facility waterside of the alignment. Perpendicular utilities are to be connected utilizing flexible joints.

- **Ground Improvement:** Ground improvement utilizing buttress size and panel layout as identified in geotechnical engineering section of this report. For Zone 1, ground improvement assumed to remain waterside of the existing combined sewer storage box, with nominal amount of ground improvement bayward of new concrete seawall but below the water surface to avoid permanent bayfill. This will require improving soils below the existing rock dike, which is expected to require pre-drilling to access soils to be improved with deep soil mix or jet grout equipment.
- **Light Weight Fill:** Over-burden compensation such as lightweight fill, ground improvement or over-build to be included as required where engineered fill is added to elevate the shoreline. This compensating mechanism is required due to high consolidation settlement expected when weight of fill is added to existing column of soft soils outside of ground improvement buttress footprint. Urban environment and hardscape surfaces utilized in the plan complicate post-construction mitigation of settled soils. Therefore, assume balance fill approach, wherein a depth of existing normal weight soil is replaced by light weight fill to balance the total weight of soil to match in-situ stress on soft soils.
- **Seawall:** New concrete seawall assumed to be constructed just bayward of existing wall utilizing a combination of driven reinforced concrete piles and in-situ grouting to fill annular space. Concept based on the design of the Lake Borgne surge barrier. Existing seawall to remain in place or be demolished after completion of new seawall. Assume temporary sheetpile cofferdam is required during construction of the seawall to contain potential grout blowout from entering the bay.
- **Wharf:** Bulkhead wharf, typically 50 feet wide running parallel to the alignment, to be replaced with new elevated reinforced concrete wharf structure with top of deck at specified flood defense crest elevation. New wharf assumed to have 12 inches thick reinforced concrete flat slab, with 24 -inch octagonal reinforced concrete piles 100 feet long spaced on a 15feet by 15 feet grid. Alternatively, steel pipe piles could be utilized for this design.
- **Building Move:** Historic bulkhead buildings are located on top of the existing bulkhead wharf. The plan calls for these buildings to be preserved and elevated, resting atop the new wharf structure. It is assumed that these buildings will be either braced and moved to a temporary location (on land or to floating barge) or carefully dismantled and stored until the new wharf is constructed. Once new wharf is constructed the historic bulkhead building will be moved back or reassembled. Assume building restored in-kind as cold shell, not including seismic retrofit. Further evaluation of building code triggers may identify a requirement for seismic retrofit as part of the project.

  Piers: Finger pier and historic shed building that stretch out into the bay will remain at existing elevation. Short pier floodwall assumed to be 2 feet tall and no more than 1-foot-thick reinforced concrete to be attached to perimeter of existing pier deck. This wall will act as asset level floodproofing rather than part of the coastal flood defense system that protects multiple assets and infrastructure systems. Reinforced concrete ramps to allow pedestrian and vehicular access from the elevated wharf to the finger piers also assumed in the scope of the project.

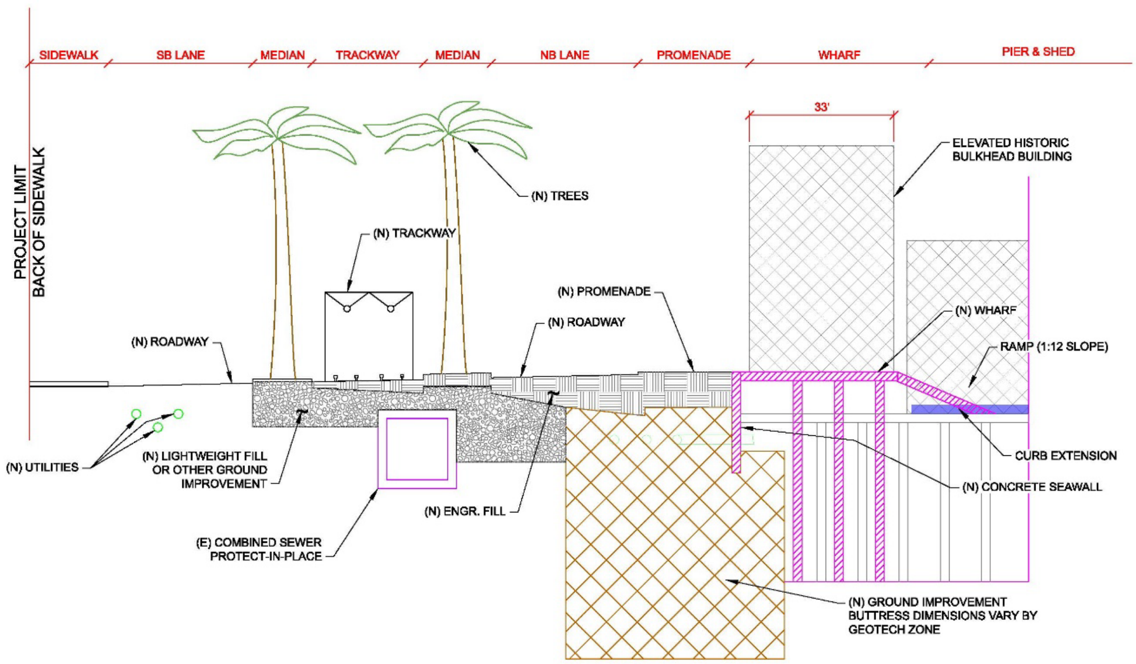


Figure B-54: Section 1 Recommended Plan (Reach 2, Pier 19)

5.3.2 Section 2 – Typical Central Embarcadero Seawall, Wharf and Bulkhead Building with Zone 2 Soils

- **General:** Scope for Section 2 like Section 1, including the means to gain elevation across the Embarcadero right-of-way, relocation of utilities, use of light weight fill, construction of new seawall and installation of pier floodwalls and ramps.
- **Ground Improvement:** The ground improvement techniques for this section are like those assumed for Section 1, but due to geotechnical conditions this section fits within geotechnical Zone 2 where competent soils are deeper which requires a deeper and wider ground improvement buttress. As a result, the buttress may require further bayward extension below the mudline to avoid impacts to the existing combined sewer storage box located within the Embarcadero right-of-way.

- **Wharf & Building Move:** In this section, the width of the wharf replaced increases due to the characteristics of the historic bulkhead buildings that are to be moved and restored to preserve historic integrity.

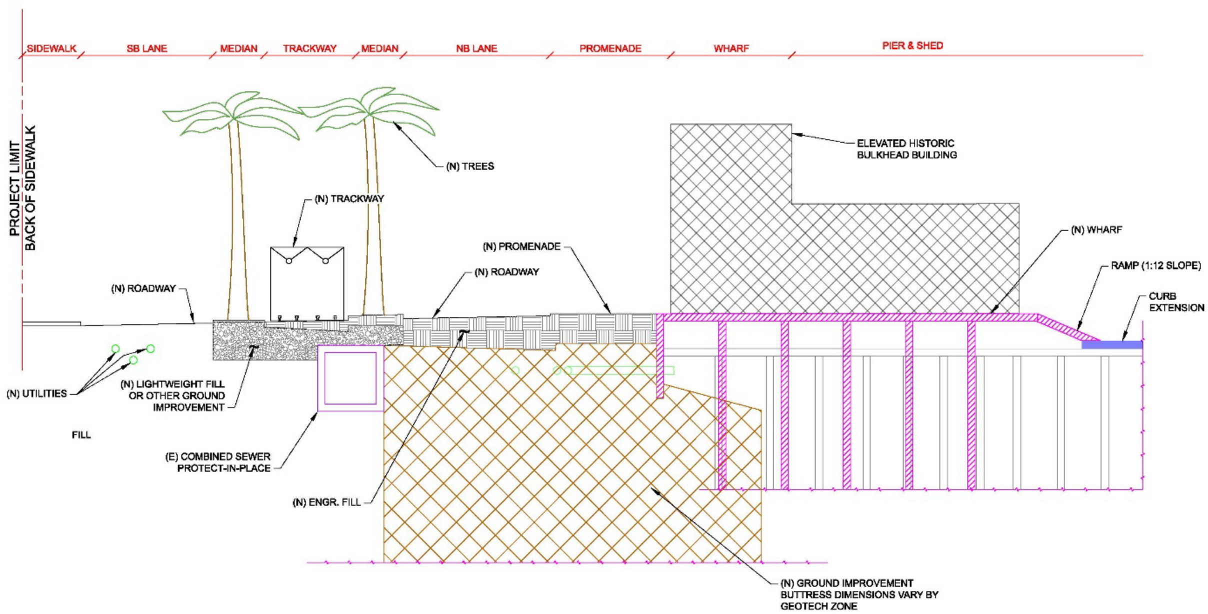


Figure B-55: Section 2 Recommended Plan (Reach 2, Pier 3)

5.3.3 Section 3 – Typical Ferry Building Seawall with Zone 2 Soils

- **General:** Scope for Section 3 shares several similarities with Section 2 landside of the existing seawall including elevation gain across the Embarcadero right-of-way, relocation of utilities, use of light weight fill, and ground improvement. However, due to the unique nature of the historic ferry building the following scope differs from Section 2. Underground transit tunnel and existing 36-inch diameter force main to be protected in place during ground improvement and pile driving operations.
- **Seawall:** New concrete bulkhead wall around the perimeter of the ferry building, assume similar seawall details identified for Section 1.
- **Ferry Building Raise:** The Ferry Building is bayward of the existing seawall supported by a forest of timber piles, concrete caissons and an arched concrete slab. The recommended plan calls for this building to be elevated in place which is assumed to require:
  - dewatering of the building substructure
  - construction of a 4' thick concrete slab at the base on the existing concrete caissons

  - installation of 8-inch diameter by 100-foot long micropiles on 10 feet by 10 feet grid to support additional surcharge without significant settlement
  - relocate tenants and selectively demolish interior improvements to allow for temporary bracing and reinforcement to minimize damage during lifting operation
  - saw cutting of the existing concrete caissons to create plane for jacking
  - use of synchronized jacks to lift building to final elevation
  - re-establish structural connection between upper and lower section of the concrete caisson
  - complete repair and restoration of substructure and superstructure to allow building to resume operations

- **Wharf:** Replace the pile supported structure bayward of the ferry building at the same elevation as the coastal flood defenses, thereby providing emergency egress from critical transit infrastructure. New pile supported structure assumed to be reinforced concrete flat slab on piles like details identified for Section 1.
- **Floodproofing:** Critical transit facility located is assumed to be dry floodproofed utilizing deployable elements across 3 openings in the building´s perimeter. Remainder of the structure is reinforced concrete not considered to be vulnerable to flood damage. Elevation of adjacent pile supported structure may require modifications to openings or ramps to allow continued access.

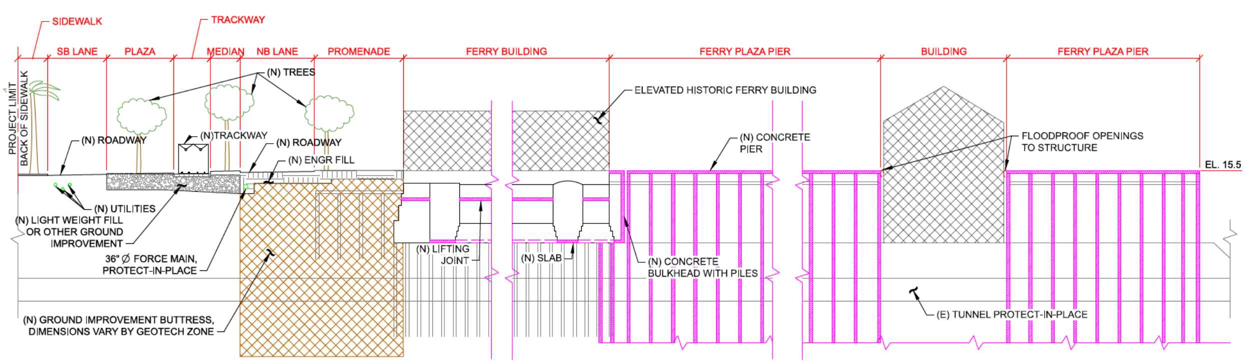


Figure B-56: Section 3 Recommended Plan (Reach 2, Ferry Building)

5.3.4 Section 4 – Typical Rincon Park Bayward Seawall with Zone 2 Soils

- **General:** Utilize similar assumptions as Section 2 for utilities, ground improvement, and lightweight fill. Protect existing subsurface transit tunnel, combined sewer storage box and force main in place during ground improvement and excavation activities.
- **Embarcadero Right-of-Way:** Gradual shoreline elevation across partial width of Embarcadero right-of-way and existing park space to ensure elevation gained at

seawall alignment meets requirements for accessibility along and across the coastal flood defenses, while mitigating negative environmental impacts. Utilize engineered fill to make up grade between existing and finished elevation. This includes replacing roadway and park space in-kind but does not include elevating the light rail corridor located between north and south travel lanes.

- **Seawall:** new concrete seawall utilizing details like those specified for Section 1 to be located approximately 60 feet bayward of the existing seawall to mitigate disruption to below grade transit infrastructure and surface transit corridor.
- **Bayfill:** utilize engineering fill to backfill the area between the new and existing seawall, elevating the surface to the final coastal flood defense elevation.
- **Natural & Nature Based Feature:** construct a rocky intertidal nature-based feature along new seawall, which is assumed to require bayfill up to 100 feet bayward of the new seawall varying in depth and thickness. Ground improvement below nature-based feature or overbuild of fill will be utilized to account for expected settlements when fill is added to the compressible bay mud. See discussion of rocky intertidal features in Appendix I: Engineering with Nature for additional technical information.

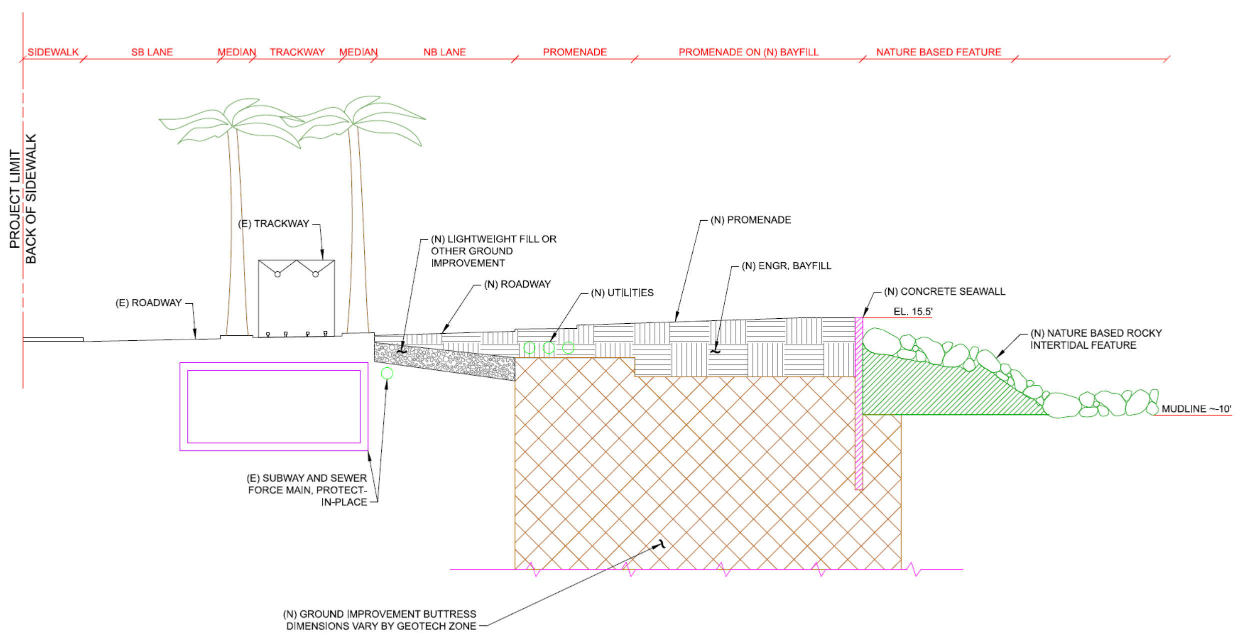


Figure B-57: Section 4 Recommended Plan (Reach 2, Rincon Park)

5.3.5 Section 5 – Typical Southern Embarcadero Seawall and Wharf with Zone 1 Soils

- **General:** Scope for Section 5 like Section 1, except as identified below.

- **Embarcadero Right-of-Way:** elevation gained across partial width of the Embarcadero right-of-way, including only the northbound travel lanes and promenade while avoiding construction within the light rail corridor.
- **Seawall:** new seawall assumed to be constructed landward of the existing seawall utilizing a drilled secant pile wall design composed of 3-foot diameter reinforced concrete shafts with a depth of 60 feet.

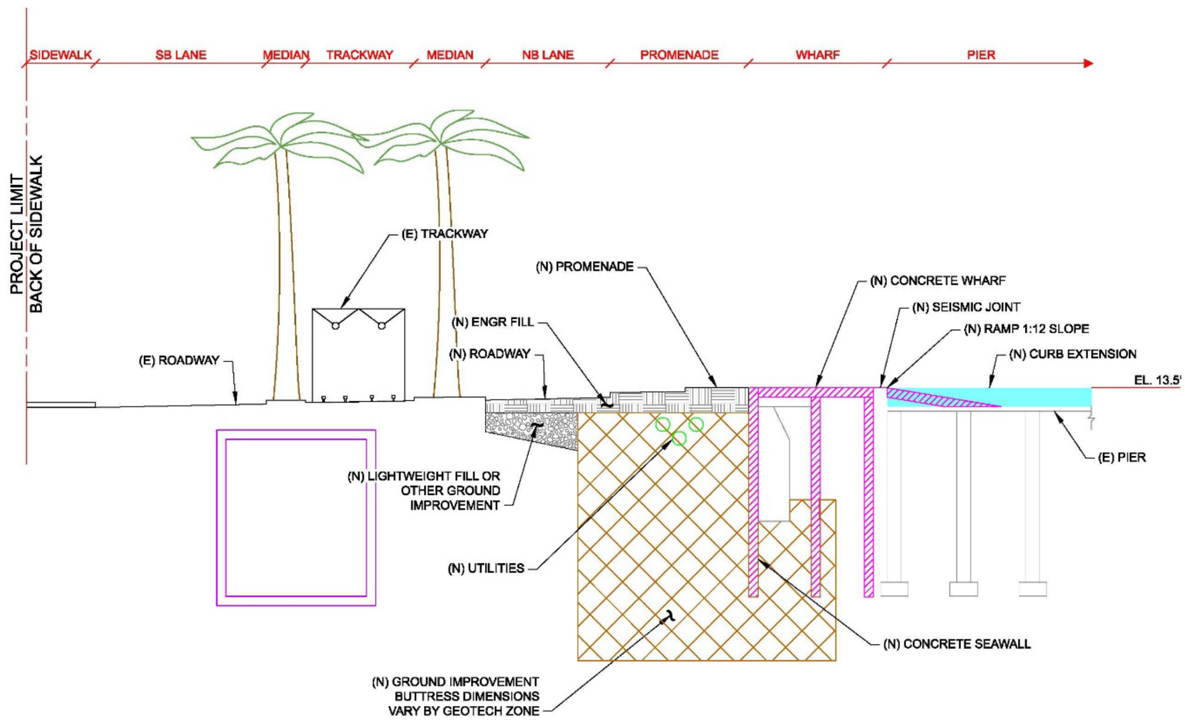


Figure B-58: Section 5 Recommended Plan (Reach 3, Pier 28)

5.3.6 Section 6 – Typical South Beach Harbor Seawall and Wharf with Zone 1 Soils

- **General:** Section 6 is located away from the Embarcadero right-of-way, therefore is not assumed to have any impact to the traffic or light rail corridors. Scope for utilities is expected to be like Section 1.
- **Wharf:** replace existing bulkhead wharf bayward of the existing seawall utilizing detail like that specified for Section 1. Note that existing and planned wharf dimensions extend further bayward in this section than other sections of the northern waterfront.
- **Ground Improvement:** assume that ground improvement technique is like those identified in Section 1, however due to adjacent properties a significant portion of

this work may need to be performed bayward of the existing seawall while remaining below the mudline.

- **Seawall:** utilize drilled secant pile seawall design like Section 5 located landward of the existing seawall.

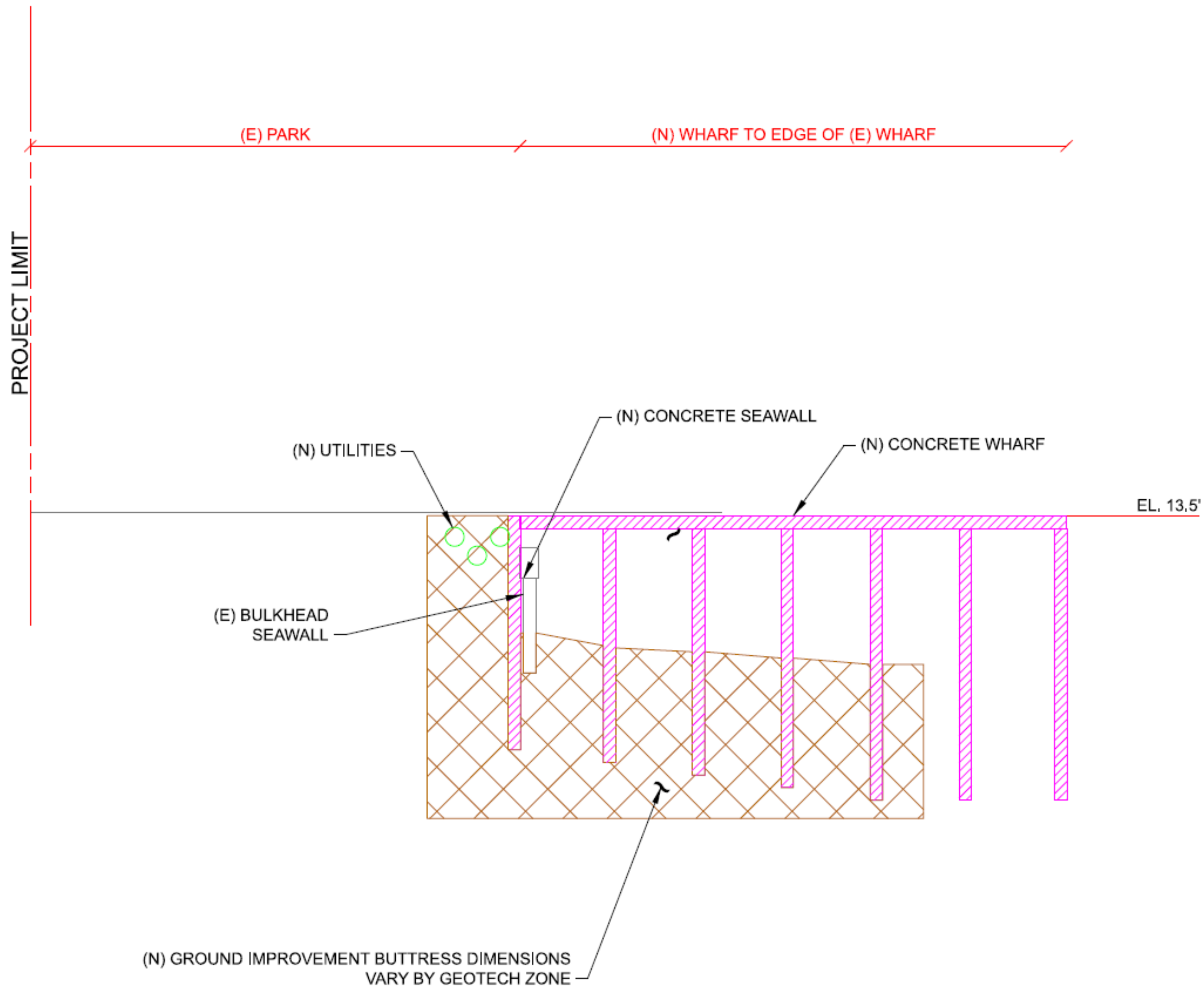


Figure B-59: Section 6 Recommended Plan (Reach 3, South Beach Wharf)

5.3.7 Section 7 – Typical Mission Creek North Levee and Constrained EWN with Zone 2 Soils

- **Levee:** Section 7 represents the levee design for a typical condition along a creek where space for elevation gain is constrained. The levee is assumed to be constructed of engineered fill with a hardscape slope or steps on the landside at approximately a 3Horizontal (H):1 Vertical (V) slope between existing and crest elevation. Crest assumed to be 12-foot-wide paved promenade matching the existing functionality. Creekside slope assumed to be 5H:1V planted with native grasses and vegetation complying with EP 1110-2-18 between crest and sheetpile terraces. Landside hardscaping intended to mitigate risk of erosion in the event of overtopping.

- **Natural & Nature Base Feature (Sheetpile Terraces):** A natural and nature-based feature has been identified along the creek banks; therefore assumption is that sheetpile terraces are constructed at varying elevation between MLLW and 10-foot NAVD88 elevation. Utilize 3 rows of PZ27 sheetpile with a length of 15 feet to construct the terraces. Annular space between the sheet piles to be planted as identified in Appendix I: Engineering With Nature. Alternative terracing structures such as gabions to be explored during design phase.
- **Vinyl Sheetpile:** to mitigate seepage through existing fill material below the levee assume a single row of 20 feet long cmi SG-625 vinyl sheetpile installed to act as seepage barrier.
- **Utilities:** Like Section 1, relocation of existing utilities within the levee alignment assumed to occur. This includes the installation piping as required to manage local drainage.
- **Ground Improvement:** utilizing buttress size and panel layout as identified in geotechnical engineering section of this report. This section occurs within geotechnical Zone 2 with deeper and wider buttress. To avoid disruption to landside buildings some portion of ground improvement may need to be constructed within the existing creek below the mudline. Assume cofferdam and temporary fill are installed to complete the ground improvement operation utilizing land-based equipment, then fill and cofferdam removed to restore the creek channel with specified CSRM improvements once ground improvement is complete. No rock dike in this section of the waterfront therefore assumes more cost-efficient deep soil mixing is utilized.
- **Armoring:** armor stone installed at the toe of sheetpile terraces to mitigate scour. Stone quantity assumed to be 5 cubic yard (cy) per linear foot (lf)_ covering shoreline slope between elevation -2 feet and +6 feet NAVD88 with a layer thickness of approximately 5 feet for sufficient interlock and stability.

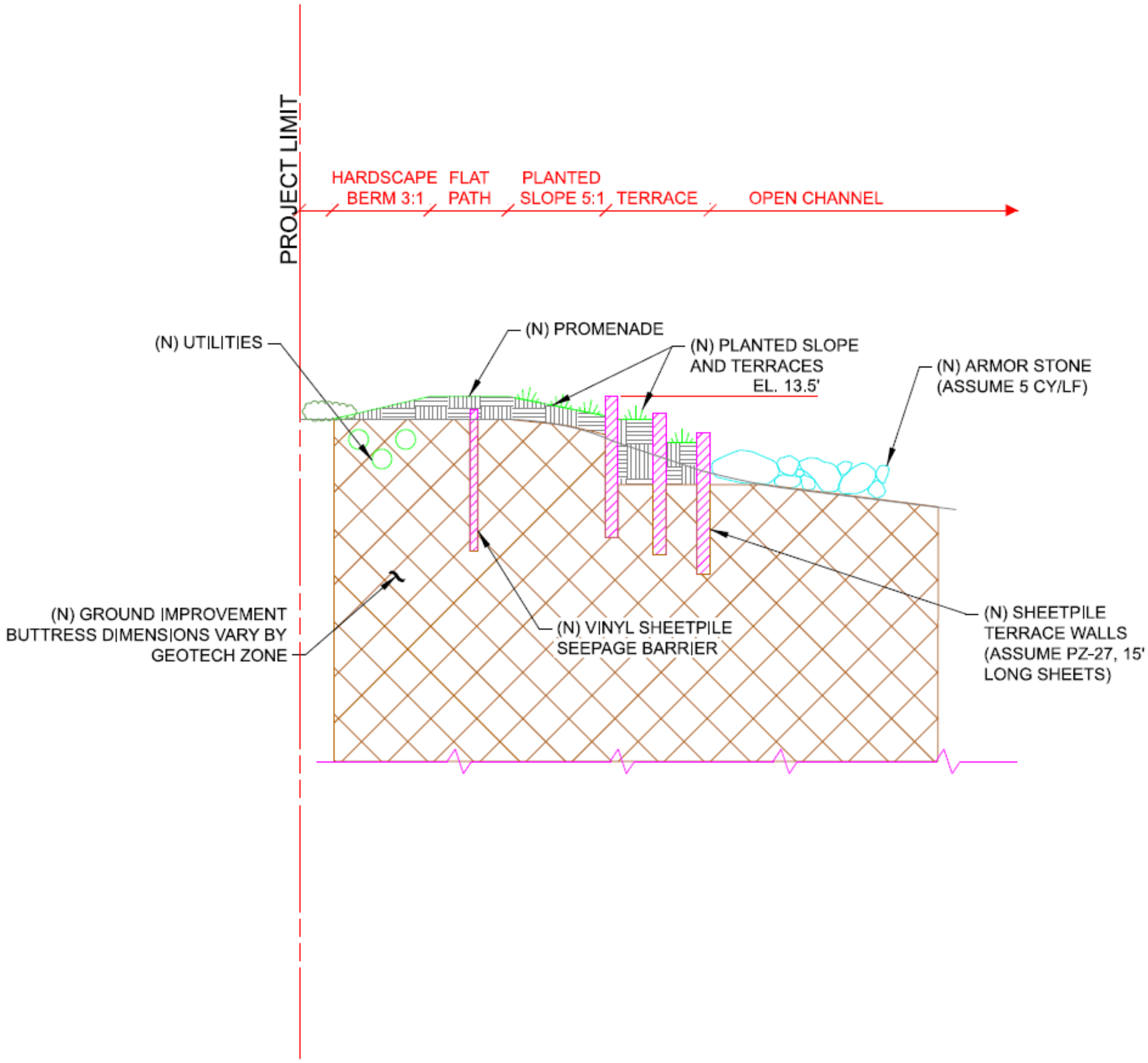


Figure B-60: Section 7 Recommended Plan (Reach 3, Mission Creek North)

5.3.8 Section 8 – Typical Mission Creek South Levee and Wide EWN with Zone 2 Soils

- **General:** Utilize similar assumptions as Section 7 for utilities, vinyl sheetpile and armoring.
- **Ground Improvement:** buttress size is like Section 7, however, the location of the existing combined sewer storage box parallel with the levee alignment complicates the ground improvement design. Assume two sets of ground improvement panels on either side of the combined sewer box, ensuring that the existing box is protected in place at all times. The design phase shall analyze the effectiveness of a split ground improvement buttress to resist settlement, liquefaction and lateral spreading loads.

- **Levee:** utilize a varying slope no steeper than 5H:1V on the landward side to meet existing park topographic features. Crest to be a 20-foot-wide hardscape promenade replacing the existing paved pathways. Assume Creekside slope is 4H:1V planted with native grasses and vegetation aligned with Appendix I: Engineering With Nature description of creek enhancements. No sheetpile terraces along this section which is not as constrained as Section 7.

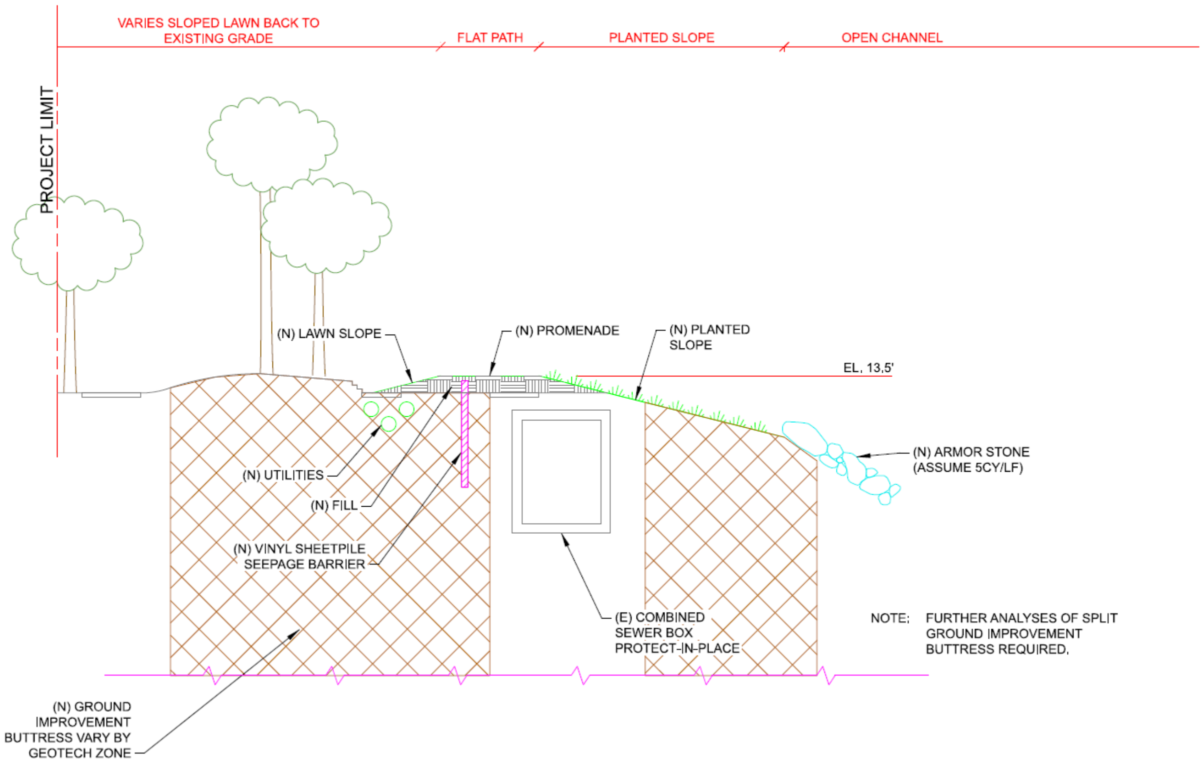


Figure B-61: Section 8 Recommended Plan (Reach 3, Mission Creek South)

5.3.9 Section 9 – Typical Mission Bay Levee and EWN with Zone 2 Soils

- **Levee:** Section 9 represents the levee design for the typical bayfront condition. The levee is assumed to be constructed of engineered fill with a hardscape slope or steps on the landside between existing and crest elevation. Crest assumed to be 16' wide paved promenade matching the existing functionality. Bayside slope assumed to be a nature-based feature described in subsequent bullet. Landside hardscaping intended to mitigate risk of erosion in the event of overtopping.
- **Vinyl Sheetpile**: Like Section 7, vinyl sheetpile wall installed as seepage barrier due to porous nature of existing soils.
- **Ground Improvement:** buttress size is like Section 7 however no temporary fill within the water column will be required due to sufficient footprint within the right-of-way to the low water line.

- **Terry Francois Right-of Way:** Gradual shoreline elevation across partial width of Terry Francois boulevard right-of-way and shoreline promenade ensure elevation gained at seawall alignment meets requirements for accessibility along and across the coastal flood defenses, while mitigating negative environmental impacts. Utilize engineered fill to make up grade between existing and finished elevation. This includes replacing roadway and promenade in-kind, where promenade is assumed to be located at crest of levee.
- **Utilities:** Like Section 1, relocation of existing utilities within the levee alignment assumed to occur. This includes the installation piping as required to manage local drainage.
- **Lightweight Fill:** utilize light weight fill for overburden compensation like Section 1 where elevation is gained outside of the ground improvement footprint.
- **Armoring:** Like Section 7, armor stone to be installed at the toe of the bayside slope to mitigate excessive erosion and bayward movement of the Nature Based Feature.
- **Natural & Nature Based Feature:** bayside slope of the levee assumed to be a coarse beach considered to be a nature-based feature as identified in Appendix I: Engineering With Nature. The stone size assumed to be 4 inches minus gravel and cobble as locally available with a thickness of approximately 4 feet over the length of the 4H:1V slope.

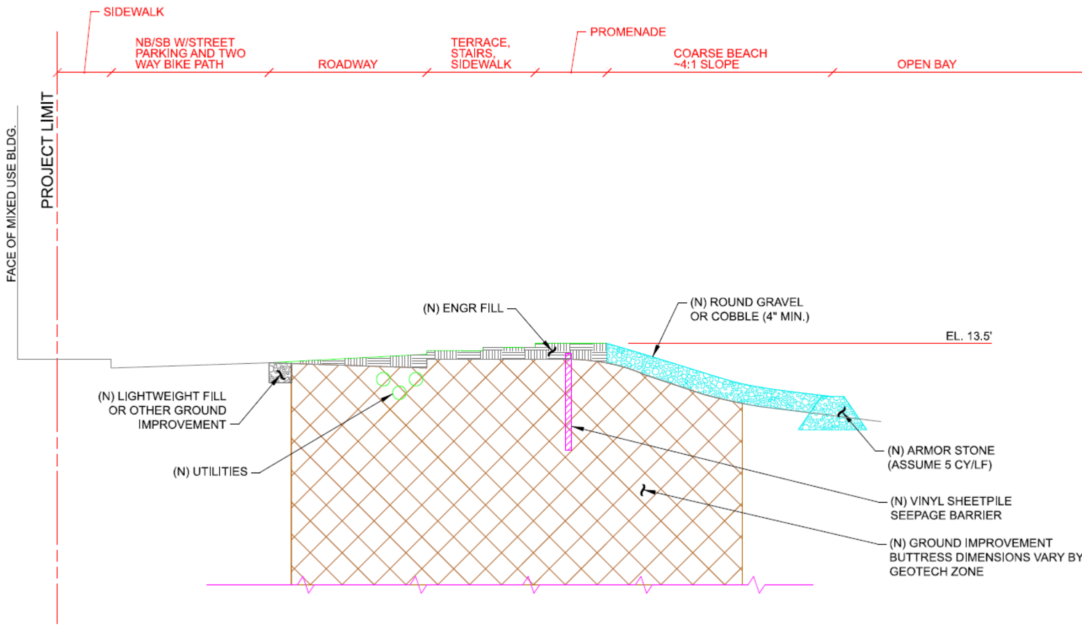


Figure B-62: Section 9 Recommended Plan (Reach 3, Mission Bay)

5.3.10 Section 10 – Typical Shipyard Levee with Zone 1 Soils

- **General:** Utilize similar assumptions as Section 7 for utilities and vinyl sheetpile seepage barrier within the levee core. Adjacent buildings are assumed to be protected in place during construction.
- **Levee:** Section 10 represents the levee design within an active parking lot. Due to the relatively small elevation gain required, the levee design in this area is assumed to have shallow side slopes covered with asphalt paving to allow for vehicular traffic to pass over the levee. Side slopes are assumed to be 20H:1V with a vertical curve at the crest rather than a typical flat crest. Hardscape surface expected to significantly decrease erosion risk in the event of overtopping.
- **Ground Improvement:** Buttress size for Zone 1 soils is assumed to be 50 feet by 50 feet along the levee alignment. At Section 10, the buttress is assumed to be centered on the levee and expected to mitigate liquefaction, lateral spreading and settlement risk to ensure the CSRM system meets the design requirements.

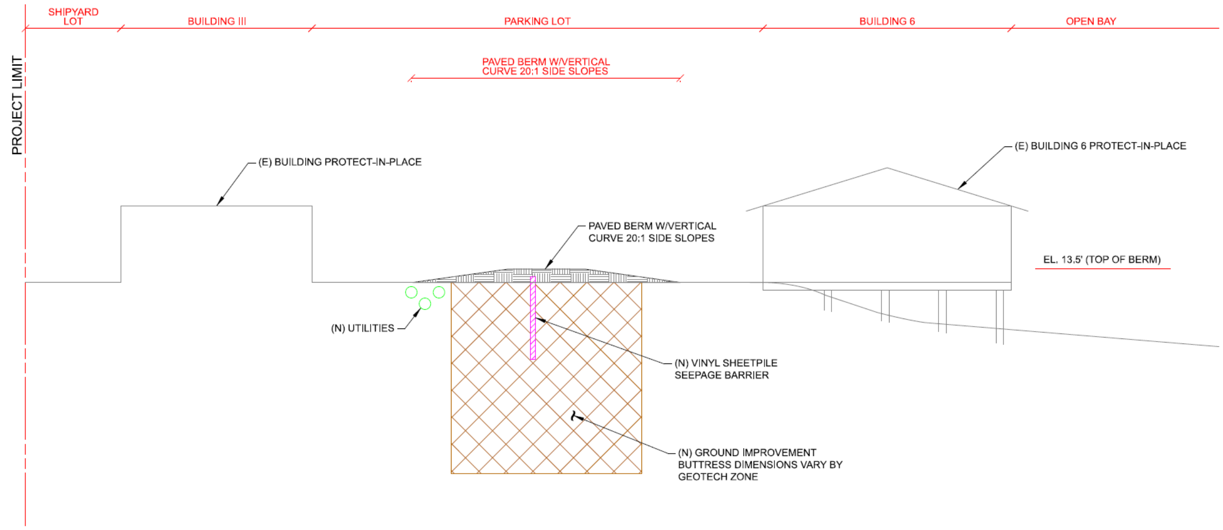


Figure B-63: Section 10 Recommended Plan (Reach 3, Shipyard)

5.3.11 Section 11 – Typical Warm Water Cove Levee and EWN with Zone 1 Soils

- **General:** Utilize similar assumptions as Section 9 except for ground improvement and levee design.
- **Ground Improvement:** Buttress size for Zone 1 soils is assumed to be 50 feet by 50 feet assumed to constructed on land, such that in-water construction is not required.
- **Levee:** utilize assumptions like Section 9, except landside slope to be 3H:1V planted slope and crest width assumed to be 10-foot wide with paved trail along the crest. Natural and nature-based feature assumed to be utilized on the bayside slope.

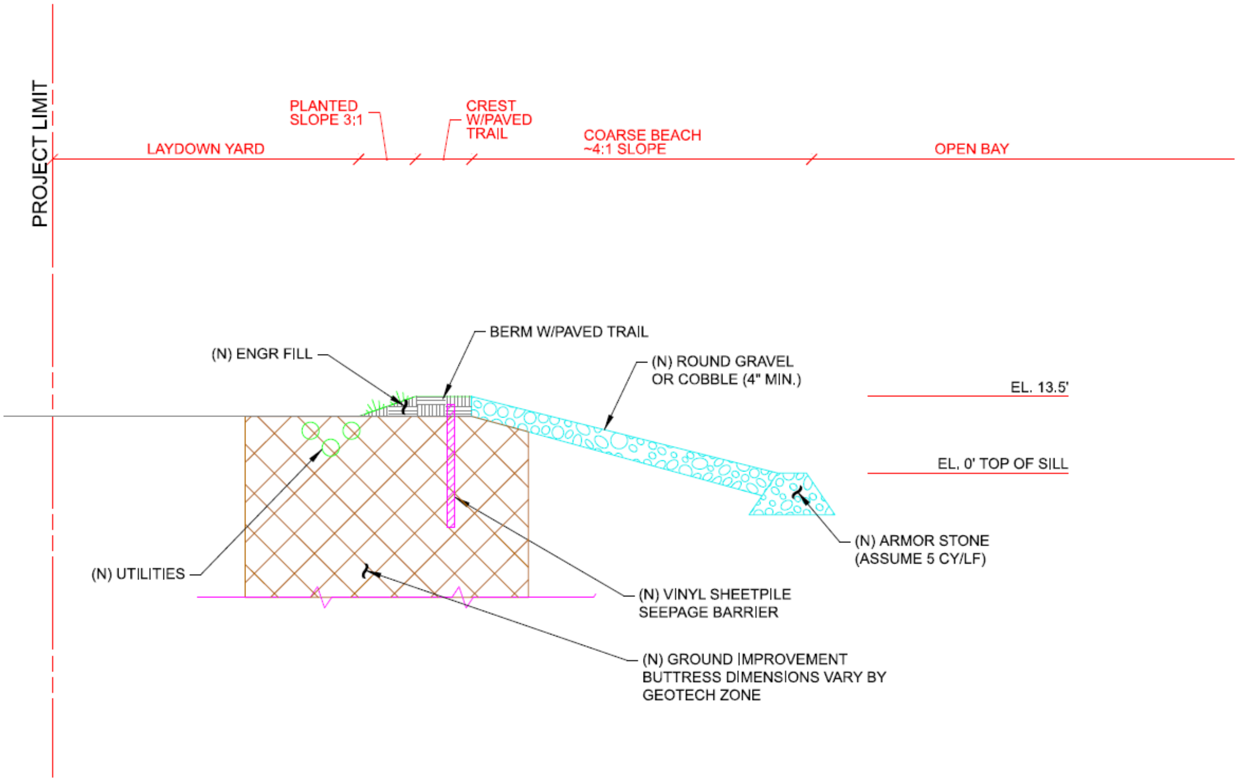


Figure B-64: Section 11 Recommended Plan (Reach 4, Warm Water Cove)

5.3.12 Section 12 – Pier 80 Seawall and Building with Zone 2 Soils (Typical Pier 80 similar)

- **Seawall:** Assume secant pile wall landward of existing bulkhead wall like Section 5. However, due to increased depth to competent soils, assume increased pile diameter to 4 feet and length to 80 feet.
- **Building Move:** Existing warehouse buildings are located within the ground improvement footprint. To mitigate constructability challenges, the assumption is that these warehouse buildings are moved, through either demo and replacement or lifting and moving the building to a new location within the project site. Future constructability analysis may indicate that ground improvement operations can occur within the building footprint in which case the building move would not be required.
- **Ground Improvement:** utilize ground improvement buttress size and construction approach like Section 2 for Zone 2 soils assumed to require 100 feet by 100 feet buttress. Due to the nature of liquefiable sand dike along the seawall alignment, it is likely that some portion of ground improvement will need to occur waterside of the seawall, but below the mudline such that it does not constitute “bay fill”. Relieving platform for existing force main may complicate ground improvement operations.

- **Utilities:** assume utilities within the ground improvement footprint are permanently relocated prior to buttress construction. New utilities should support landside drainage and connect with warehouse building as required.
- **Force Main:** an existing 60-inch wastewater force main along the southern and southeastern berth of pier 80 bisects the CSRM alignment and significantly complicates construction operations. To mitigate this, it is assumed that the force main is temporarily or permanently relocated outside of the CSRM alignment prior to construction of the seawall and ground improvement buttress. Force main is critical infrastructure and must always remain operational.

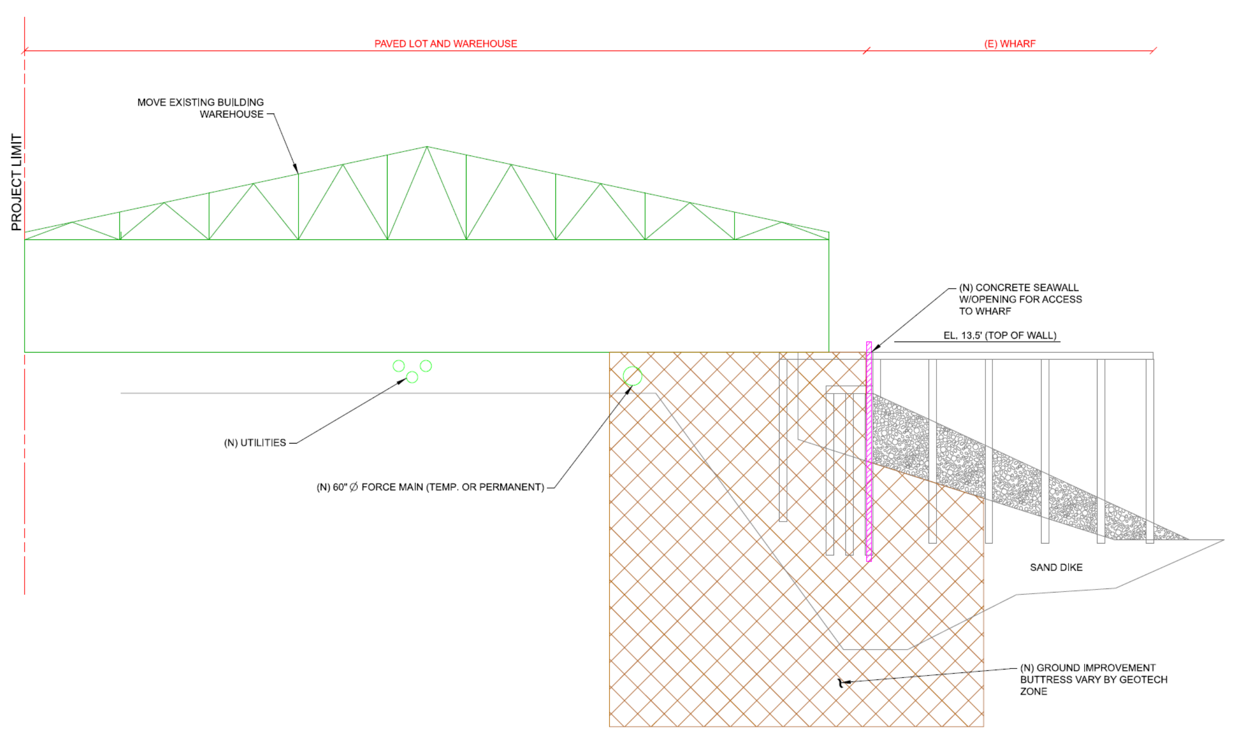


Figure B-65: Section 12 Recommended Plan (Reach 4, Pier 80)

5.3.13 Section 13 – Islais Creek North Floodwall at Existing Combined Sewer Outfall Structure

- **General:** Section 13 along the creek at the location of an existing combined sewer overflow structure which must remain fully operational during wet season. As such, the scope is limited to increasing the shoreline elevation along the edge of the existing structure utilizing short concrete floodwalls and utility work as needed. Assume no ground improvement required due to the design of the existing infrastructure as a critical facility. This assumption to be validated during the design phase.

- **Floodwall:** utilize a reinforced concrete floodwall with a height of 2 feet and no more than 1-foot thick mounted to the existing reinforced concrete structure. This wall to be similar to pier floodwalls identified in Section 1.
- **Utilities:** installation of a floodwall along the edge of the existing infrastructure is assumed to require new utilities to support local drainage and potentially lines (i.e. power, communication) for the flap gates identified in the interior drainage scope.

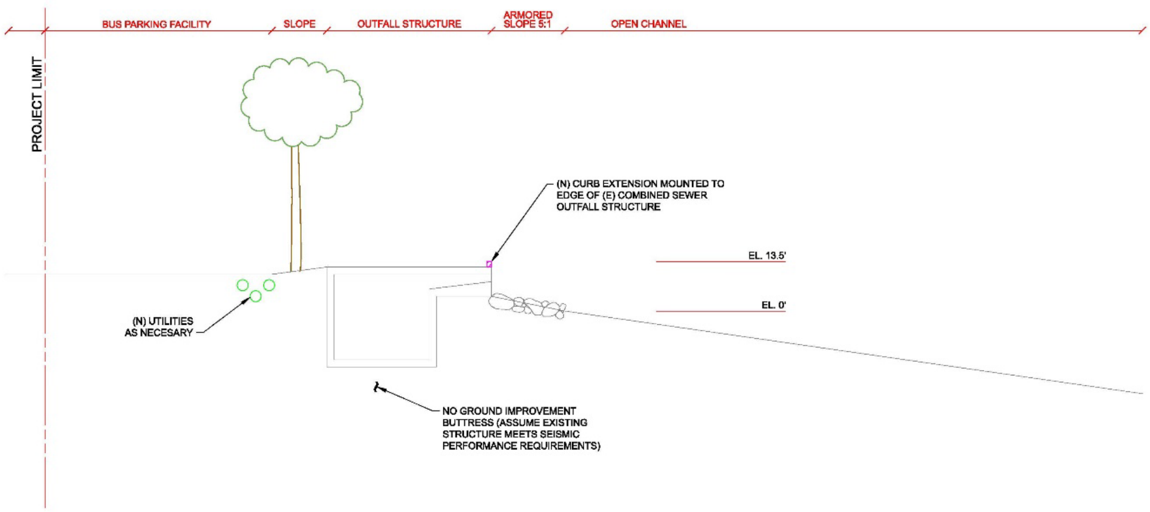


Figure B-66: Section 13 Recommended Plan (Reach 4, Islais Creek North)

5.3.14 Section 14 – Typical Islais Creek South Levee and Constrained EWN with Zone 2 Soils

- **General:** Section 14 is like Section 7, except requires partial demolition of several existing buildings within the project alignment.

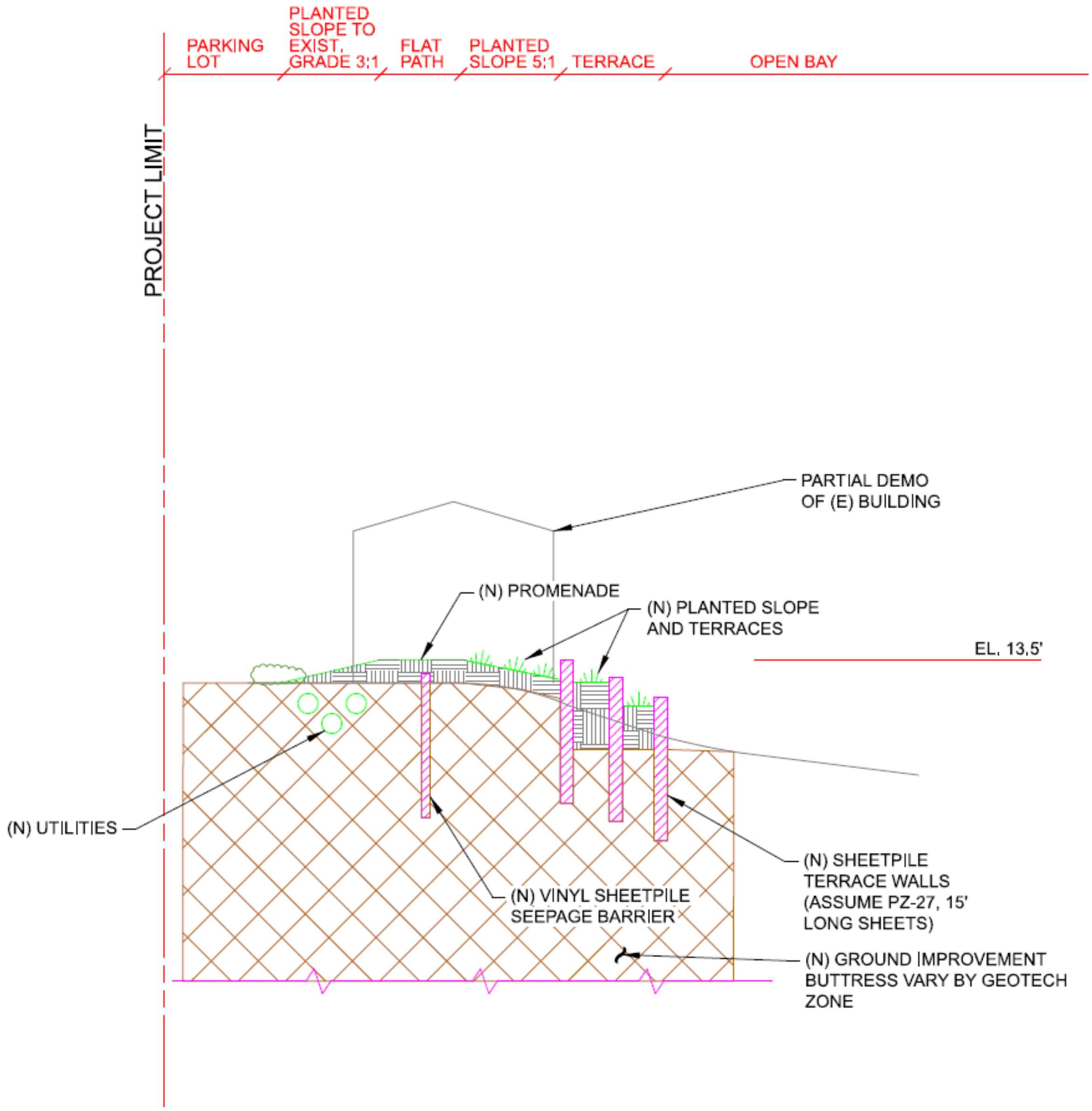


Figure B-67: Section 14 Recommended Plan (Reach 4, Islais Creek South)

5.3.15 Section 15 – Typical Pier 92 Seawall, Wharf and Buildings with Zone 2 Soils

- **General:** Section 15 represents a typical section along an active maritime berth with upland storage and processing operations. Like sections previously identified, utility relocation will be required to install the ground improvement buttress and manage local drainage. Additionally, the CSRM features require grade changes across this section such that new concrete ramps (20H:1V slope assumed) will be required to access elevated wharf from existing grade.
- **Seawall:** utilize secant pile seawall like Section 5.

- **Wharf:** new reinforced concrete wharf like Section 1 to replace footprint of existing wharf structure only in areas of active maritime operations. The wharf is wider, typically 80 feet wide, at this Section requiring more pile rows parallel to the shoreline compared to Section 1.
- **Ground Improvement:** buttress size, layout and landside installation assumed to be like Section 9. However, due to the large quantity of new piles installed for the wharf future, design phase analysis should consider the effects of pile pining on the shoreline slope and optimize ground improvement accordingly.
- **Buildings, Equipment and Goods:** to facilitate efficient construction operations, small buildings, equipment and goods shall be moved on site to a location outside of the project footprint. This may require demolition and rebuilding or lifting and moving, which will be determined in the design phase based on efficiency and value.

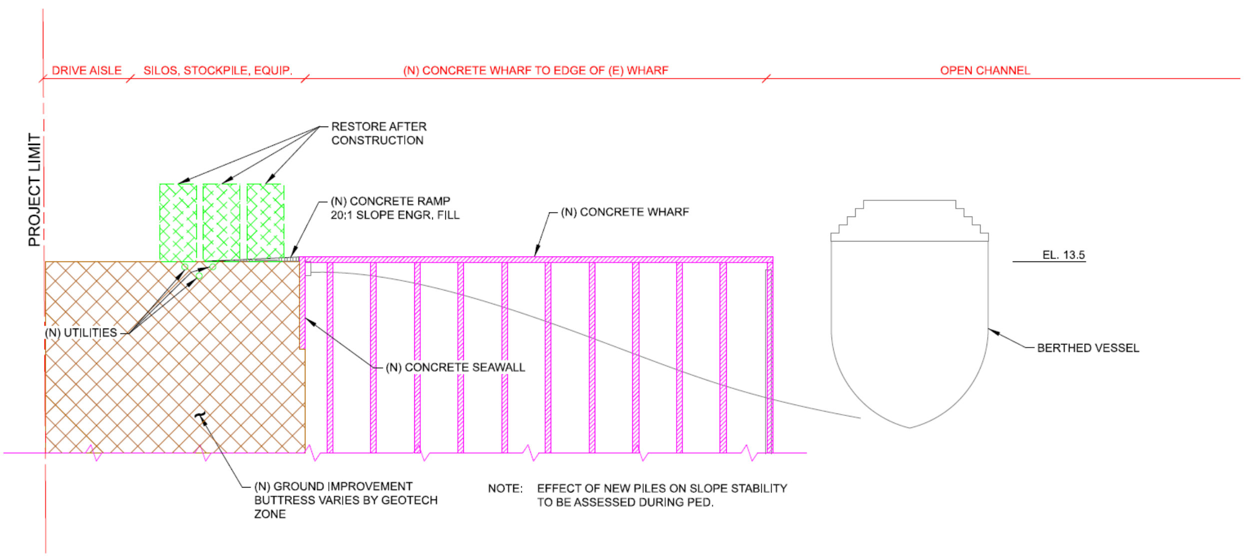


Figure B-68: Section 15 Recommended Plan (Reach 4, Pier 92)

5.3.16 Section 16 – Typical Pier 94 EWN

- **Natural & Nature Based Feature:** existing wetlands at Pier 94 are to be enhanced by construction of a wetland enhancement feature. Details of this feature are provided in Appendix I: Engineering With Nature. Note that no ground improvement is assumed for this Section due to the occurrence of high ground at this location.

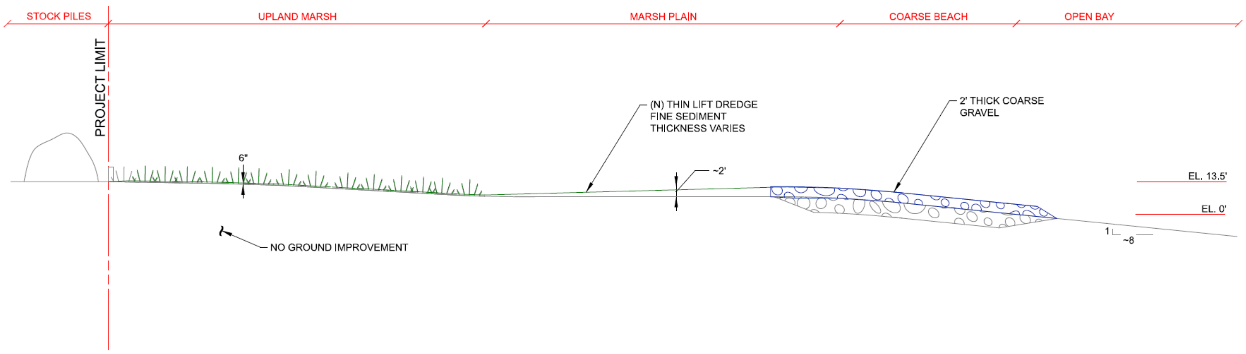


Figure B-69: Section 16 Recommended Plan (Reach 4, Pier 94 Wetlands)

5.3.17 Section 17 – Typical Pier 96 Combi Wall, Bayfill and Building with Zone 2 Soils

- **General:** utilize assumptions like Section 12 for building move, utilities and ground improvement.
- **Seawall:** due to the condition of the existing sheetpile wall and lack of a concrete wharf structure in this Section assume the new seawall is constructed approximately 10- feet bayward of the existing using a combi-sheet pile wall with tie backs. This wall is assumed to be combination of 52-inch diameter by 7/8-inch wall pipe pile on 9-foot centers with PZC26 sheets, with 60% sheet to pile depth ratio. Pile depth assumed to be 80 feet. Tieback assumed to be achieved with 3-inch diameter by 60-foot-long tie rods at 9-foot on center anchored to a continuous 4 feet by 4 feet reinforced concrete deadman located within the ground improvement zone.
- **Bayfill:** engineered fill is utilized to fill the annular space between the new and existing seawalls.

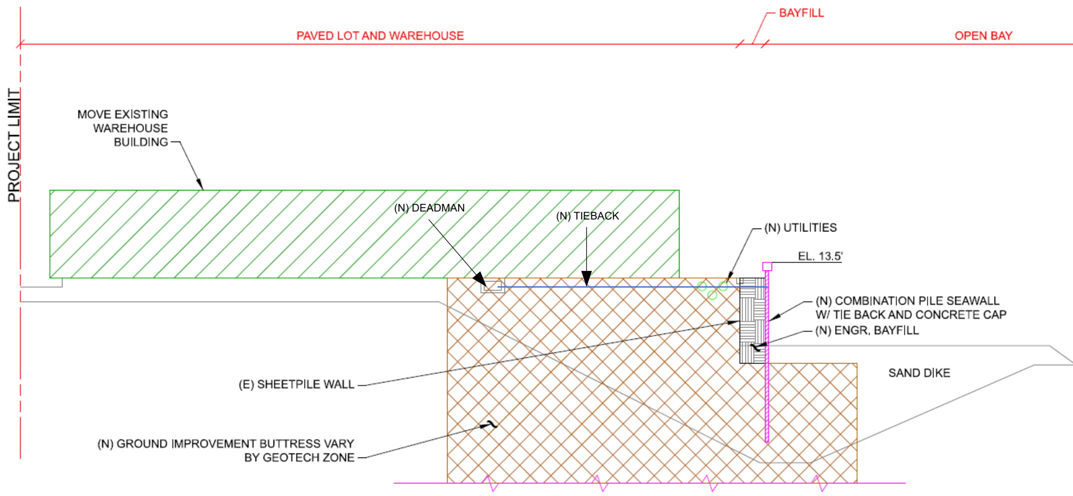


Figure B-70: Section 17 Recommended Plan (Reach 4, Pier 96 South)

6.0 Constructability

Construction, particularly in the NWF, will be along roadways and critical infrastructure corridors, 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 for pedestrians, bikes and access to key maritime facilities during construction. Additionally, a temporary or permanent relocation plan for utilities will be required to allow for effective and continuous ground improvement operations. These relocations and closures are included in the baseline construction schedule.

Construction will also occur near, adjacent or within structures, some of which are designated historic resources. This leads to smaller, congested work areas, typically taking longer and costing more to complete. Temporary relocation of historic buildings will be a challenging activity that requires careful preparation and protection of the buildings, large equipment for the move, sufficient staging areas and finally, when the building is returned to its current location, the works to reinstate its current condition. Similarly, the elevation of the historic Ferry Building involves several constructability challenges which require advanced construction planning and analysis during the Design phase, such as the lifting methods (above or below deck), tower elevation, and interface with the subterranean BART transit tunnel. The PDT has considered these factors in preparing the cost estimate and construction schedule as well as itemizing these topics in the CSRA.

Due to the high utilization of the existing waterfront space, it is expected that marine construction elements will be needed for some portion of the work. This may include the use of offshore equipment to mitigate onshore staging, material supply to limit disruption to landside transportation, or construction of plan measures such as the wharves and seawalls. Use of marine equipment will be impacted by sea conditions and weather, while landside construction operations will be influenced by the weather only.

Table B-8 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-8: 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. The PDT should evaluate various alternatives of pile installation in future designs including impact pile hammers, vibratory pile hammers, predrilling, advance while drilling, or drilled shafts with permanent casing for consideration of vibration levels. If pile driving is occurring near or adjacent to structures, monitoring will be required at these locations to ensure settlement/movement is not excessive. Additionally, the PDT has assumed the use of temporary sheetpile cofferdams to separate the bay water from the construction site where possible. It is expected that this construction feature will be required for environmental mitigation or as a best management practice when performing ground improvement near the shoreline or working in the water column outside of the in-water work window (June to November), therefore has been priced into the cost estimate.

Construction issues may also be encountered due to the highly variable subsurface conditions underlying the flood control measures. Foundation elements which will extend through the artificial fill 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. Ground improvement through the rock dike may require drilling operations to penetrate the dense cap to improve the underlying weak soils. Such ground improvement operations will produce a large volume of spoils that require onsite management and hauling to designated disposal sites. Additionally, the depth to groundwater will influence construction operations, especially in the nearshore area where groundwater table is directly correlated to the tidal water levels which vary by 6 feet on average every 6 hours. These constructability challenges have been accounted for by the PDT in the base estimate with additional risk factors added to the CSRA to account for unknowns and uncertainty.

7.0 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 TSP and Recommended Plan are included in *Appendix C: Cost Engineering*. The typical sections presented in Section 5.3 above are refined examples of figures used to estimate the quantities for this project.

8.0 Cost Estimates

The baseline cost estimate for the recommended plan was developed using MCACES in the Civil Works Work Breakdown Structure format. Quantities were calculated after plan sheets and sections were developed and provided to Cost Engineer by the PDT design engineers. The sections are described in Section 5.3 Recommended Plan Development and included with plan sheets in Appendix B.5. The total real estate costs were developed by real estate PDT members for permanent construction easements and acquisition. The costs were based on an appraisal conducted by the Sacramento District. The following costs were included in the summary of 02 Account Relocations; demolition, temporary detour, and final replacement of the traffic corridors and light rail track; existing utility demo, replacement, and/or relocation; and temporarily or permanently relocating, and demolition of, building structures. Costs for pier floodwalls and ground improvement were included in the 10 account and not in the 19 Account. Costs associated with the 19 Account are the nonstructural building dry proofing and raising the Ferry building. Cultural resources preservation costs were developed and provided by PDT cultural resource project staff. Cultural resources costs were applied as 1% of the construction costs. Utility relocations were based on available data. The PDT recognizes the need for more detailed utility data in the design phase and have accounted for the risk of unknowns in the Cost and Schedule Risk Assessment (CSRA). 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 Array of Alternatives (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) was performed, and results are present in Appendix C. 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*.

9.0 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, schedule and benefits of reducing risks when making decisions. A variety of variables and their associated uncertainties are incorporated into the risk assessment. Design conditions for major coastal and flood protection projects are often vague and design parameters contain large uncertainties.

A CRSA was conducted for this study. A Monte-Carlo based risk analysis was conducted by the PDT on remaining costs in compliance with Engineer Regulation (ER) 1110-2-1302, Civil Works Cost Engineering (for Civil Works projects). The purpose of this risk analysis study was to present the cost and schedule risks considered and respective project contingencies at a recommended 80% confidence level of successful execution to project completion. The draft CSRA is included in *Appendix C: Cost Engineering*. The final CSRA and cost certification is planned to be received after completion of the quality reviews.

9.1 Life Safety

An abbreviated Semi-Quantitative Risk Assessment (SQRA) for this planning study for the San Francisco Waterfront was conducted in the first quarter of 2024. This risk assessment was 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 consisted of a facilitated Potential Failure Mode Analysis (PFMA) and a risk assessment of the potential failure modes judged to be risk drivers.

Prior to the completion of the SQRA on the Recommended 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.

10.0 Operation and Maintenance

A draft O&M manual has been started. This preliminary O&M manual will need to be further refined during the design and construction phases 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.

10.1 Gate Closure Procedure

Gate closure procedure will be finalized during the design 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.

11.0 Preconstruction Engineering and Design Considerations

Due to the study area size, project complexity, schedule, and funding constraints, a significant effort will be required in the design phase to finalize all engineering aspects of this project. Finalization of the coastal hydraulic design methodology and results are vital such that elevations and configurations of flood protection measures can be determined for all future project design purposes. The determination of acceptable methodologies for development of seismic design parameters and acceptable performance criteria will also be important items to come to resolution early in the study. Historically, given the variety of opinions on these matters, USACE CSRM projects have had trouble in reaching consensus on final coastal hydraulic and seismic designs methodologies and parameters which has led to design delays and wasted design efforts. Therefore, we emphases the importance of the design phase PDT to prioritize finalizing these important basic design components early in the design phase to avoid delays. Further geotechnical and interior drainage analysis and designs will be required during the design phase. Some of this work, such as subsurface explorations and laboratory testing, will need to start at the beginning of the design phase to obtain the necessary information to complete geotechnical and structural analyses. The work required during the design phase is discussed in detail below.

11.1 Subsurface Investigation

Subsurface information is needed to provide the designers better data beyond the information compiled herein. 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 Recommended Plan 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; however, we have collected many available geotechnical explorations from this area and developed an initial 3D geologic model as described in a preceding section of this report. We recommend future subsurface investigations in the SWF (as also recommended for the NWF) to better characterize the foundation conditions early in the design 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. Both disturbed (SPT) and undisturbed samples should be collected, and appropriate laboratory testing performed to inform the design. The testing should consist of drained and undrained shear strength determination, consolidation, permeability, soil classification tests (Atterberg limits and grain size distribution), or other tests considered appropriate by the future PDT. A variety of geophysical test methods should also be considered for this project to better define general subsurface stratigraphy and other important soil properties such as in-situ shear wave velocities.

11.2 Verify Utility Locations

The NFS has coordinated with multiple local governmental and public utility entities for the purpose of obtaining information on existing utilities in the project area. As would be expected for a highly urbanized area, there are extensive amounts of underground utilities within the proposed project footprint that range in size from very large (many 10’s of feet) BART tunnels and stormwater storage boxes to small (inches) individual structure service lines. The PDT has reviewed this data; however, exact utility locations and types will need to be verified (as much as is possible) during design phase activities.

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.

The whole of 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 of the flood protection features.

11.3 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.

11.4 Final Interior Drainage Analysis

The interior drainage analysis in evaluation of the minimum facilities and stormwater runoff was 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 so only qualitative assessments for those impacts discussed. Initial estimates were developed to account for minimum facilities and stormwater runoff control measures (WRDA 2020 Section 203) based on local city regulations. During PED and design phase the interior hydrology should be more accurately modeled and refined 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.

11.5 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 design phase.

12.0 References

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