Conceptual Engineering Report

Ocean Beach Long-Term Improvements Project

PREPARED FOR:

San Francisco Water Power Sewer
Services of the San Francisco Public Utilities Commission

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PREPARED BY:

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Contents

Figures

Tables

Glossary

AAR	Alternatives Analysis Report
CCC	California Coastal Commission
CEQA	California Environmental Quality Act
CER	Conceptual Engineering Report
CSD	Combined Sewer Discharge
CY	Cubic Yards
EQR	Emergency Revetment
LMO	Lake Merced Overflow
LMT	Lake Merced Transport and Storage Tunnel
M&N	Moffatt & Nichol
MG	Million Gallons
MGD	Million Gallons per Day
NGS	National Geodetic Survey
NOAA	National Oceanographic and Atmospheric Administration
NTS	Not To Scale
OBMP	Ocean Beach Master Plan
OSP	Oceanside Water Pollution Control Plant
PTI	Post-Tensioning Institute
SFPUC	San Francisco Public Utilities Commission
SPT	Standard Penetration Test
STA	Station
SWOO	Southwest Ocean Outfall
VLM	Vertical Land Motion
WPS	Westside Pump Station
WST	Westside Transport/Storage Box
WWE	{SFPUC} Wastewater Enterprise

Executive Summary

The San Francisco Public Utilities Commission's (SFPUC) South Ocean Beach Wastewater Systems provide treatment for the Oceanside watershed. The Oceanside watershed drains towards the Pacific Ocean and occupies over 11,000 acres. It represents roughly 35 percent of the total area of San Francisco and is divided into three sub drainage basins: Richmond, Sunset, and Lake Merced (see Figure ES-1).

The SFPUC wastewater infrastructure at South Ocean Beach (see Figures ES-2 and ES-3) includes: the Westside Transport/Storage Box (WST); the Westside Pump Station (WPS); the Lake Merced Transport and Storage Tunnel (LMT); the Oceanside Water Pollution Control Plant (OSP); and the Southwest Ocean Outfall (SWOO), and buried utilities that connect and support the listed facilities. These facilities were constructed as a result of the Clean Water Act, leading to the 1974 San Francisco Wastewater Public Works Plan to improve stormwater drainage and alleviate sewer overflows.

The City and County of San Francisco, through the Clean Water Program, constructed a major complex of sewer and stormwater infrastructure within the Oceanside watershed at Ocean Beach from about 1972 until 1997. The major components are located at South Ocean Beach (SOB). This elaborate system, some of which is located underneath the Great Highway, reduced coastal water pollution events by a factor of 10. Currently, this area is in need of coastal protection due to the narrowing of SOB as a result of coastal dynamics and sediment transport. As a result, components of the system face risk of exposure and damage due to current and future erosion in the face of sea level rise and extreme storm events. For the purposes of this report, SOB is broken up into five reaches, as shown in Figure ES-, as a means of evaluating the effects and rates of erosion in specific areas.

Historic efforts by the City and County of San Francisco (CCSF) to protect infrastructure along SOB have generally consisted of ad-hoc responses to extreme storm events, including sand berms and sandbag walls, and construction of rock revetments following El Nino storm seasons in 1999 and 2010. Recognizing the need for an integrated long-term management strategy for SOB, in 2009, the SFPUC partially funded efforts to begin the planning process for development of the Ocean Beach Master Plan (OBMP). The OBMP was a multi-agency effort to develop a sustainable long-term vision for Ocean Beach, addressing public access, environmental protection, and infrastructure needs in the context of erosion and climate-related sea level rise.

While the OBMP planning efforts were underway, the CCSF sought from the California Coastal Commission (CCC) a coastal development permit (CDP) authorizing the yet unpermitted 1997/1999 and 2010 revetments, as well as additional armoring. In the summer of 2011, the CCC denied the

CDP application. In its denial, the CCC made clear that it would no longer accept ad-hoc responses at SOB, and that any future proposals should consider the OBMP recommendations.

Through its participation in the OBMP planning process (completed in 2012), collaboration with regulators, and drawing upon new and better information related to climate change, sea level rise, and coastal dynamics, the CCSF has embraced a new approach. This updated approach is compatible with the OBMP and seeks to protect critical wastewater infrastructure at SOB in a manner that emphasizes the use of low impact techniques, and provides opportunities for integrated management (e.g., structural protection, improved public access, minimal environmental impact).

The 2018 Alternatives Analysis Report (AAR) documented the alternatives development and evaluation phase of the Coastal Adaptation Strategies for SOB Wastewater Systems. The evaluations conducted during the AAR phase used a consistent decision methodology, supported by engineering analysis, and were informed by the Coastal Protection Measures & Management Strategy for SOB (SPUR et al. 2015). A summary of the AAR's planning criteria are presented below.

The goal for the project is to:

- Maintain function and operational capacity of Oceanside Wastewater Infrastructure in a manner that incorporates the guiding principles of the OBMP and complies with regulatory requirements.

The objectives for the project are:

- Maintain current operational capacity
- Increase resilience to sea level rise
- Comply with applicable laws and regulations
- Improve beach access, recreation and habitat
- Remove shoreline armoring and rubble

The AAR considered 10 options to address structural protection, including no action and various project options involving onshore, offshore, structural, and non-structural interventions. Elements common to all of the alternatives and thus not analyzed in the AAR included:

- Removing shoreline armoring and rubble
- Improving beach access, recreation, and habitat

- Rerouting the Great Highway between Sloat and Skyline Boulevard
- Recontouring and revegetating the bluff
- Continued sand nourishment
- Improving stormwater management

These components are now being considered and are part of the CER. Refinements of these elements will continue into design.

As the Lake Merced Tunnel (LMT) is the seaward-most component of the existing wastewater system, it featured prominently in the options considered. The project options were screened based upon the Project Goal and Objectives. Four alternatives were carried forward for detailed analysis:

- Alternative A. Protect LMT with exterior low-profile wall
- Alternative B. Protect LMT with interior reinforcement + new storage
- Alternative C. Remove LMT + new tunnel alignment
- Alternative D. Remove LMT + new pump station, pipeline & storage

Each alternative was evaluated against eight criteria concerning cost, environmental impact, resilience to sea level rise, and operational complexity. The criteria were drawn, in part, from the list of suggested investigation topics presented in SFPUC's Procedures Manual, and from additional project- and site-specific considerations. The alternatives were scored and ranked based upon their relative performance. Alternative A ranked highest among the alternatives and therefore was carried on to the conceptual engineering phase.

This document represents the Conceptual Engineering Report (CER) for the chosen alternative that focused on structural protection and the common elements that were not analyzed. The purpose of the CER is to provide a clear basis for the design and construction of the project which aims to address all of the OBMP guiding principles of managed retreat, beach nourishment, structural protection and access and recreation. This document includes a 10% design of the preferred structural protection alternative, as shown in Figure ES-4, as well as conceptual designs for other elements of the project including traffic, landscaping, modified access to the zoo, modified access to the OSP and WSP facilities for SFPUC employees, and public recreational access to the beach and proposed relocated parking lot, bathroom, multi-use trail and beach. Some of these components are still in flux and will require further modification in the upcoming design documents.

This project follows the OBMP guidance and focuses on a solution in the form of managed retreat of the Ocean Beach shoreline in response to chronic erosion and future sea-level rise. However, the following criteria need to be met in order to maintain functionality of the LMT and the remaining wastewater infrastructure:

- Preserve the structural integrity of the LMT by protecting the tunnel against wave-, and erosion-related hazards. This is achieved by incorporation of a low-profile wall.
- Prevent uplift of the LMT due to buoyancy effected by high groundwater levels. This is achieved by incorporation of a soil cover (weight) over the LMT.
- Protect the LMT against seismic hazards, including liquefaction and lateral spreading. This is achieved by soil improvements around the LMT, anchored by the low-profile wall.
- Permit groundwater flow through the low-profile wall. This is achieved by limiting the tip elevation of every other pile of the (secant pile) low-profile wall.
- Permit wave runup on the beach and wave overtopping during extreme storm conditions. This is achieved by incorporation of a durable soil cover over the LMT.
- Protect existing wastewater infrastructure access and provide a public recreational trail with beach access as part of the project design. This is achieved via incorporation of an access road and trail along the coast and replenishment of sand on the beach via periodic beach nourishment.
- Protect the LMT during construction. This is achieved in several ways, but primarily by preventing construction-related dead and live loads atop the LMT, and maintaining a minimum clear distance to the tunnel during installation of the low-profile wall and re-grading the coastal bluffs.

This CER summarizes existing conditions at SOB area in terms of the beach and bluff topography, geology and stratigraphy, and natural hazards including erosion and coastal related hazards.

The proposed scope of work for the South Ocean Beach project includes:

1. Removing the Great Highway between Sloat and Skyline Boulevards and completing intersection/zoo access improvements to accommodate changed flow of traffic.
2. Installing a low-profile wall seaward of the LMT and re-grading the dune bluffs to restore the beach/dune habitat.

3. Removing the existing shoreline protection revetment and accumulated rubble.
4. Providing a wastewater infrastructure access road and public multi-use recreational trail with beach access in place of the Great Highway.
5. Establishing a program to ensure maintenance of the beach and dune system based on periodic sand nourishment.

These elements, with the aforementioned criteria, form the basis for the conceptual engineering design for the Wastewater Infrastructure Protection Project, which is shown in plan on Figure ES-4, and in representative Section on Figure ES-5.

This CER is structured around the main engineering disciplines involved in the development of the conceptual design, which include Coastal, Geotechnical, Civil, and Structural Engineering. The CER additionally considers aspects of constructability, operations and maintenance, right-of-way, and environmental review.

The overarching purpose of the project is to implement a long-term coastal management strategy for South Ocean Beach that addresses shoreline erosion and climate-related sea level rise. The specific project objectives are to:

- Preserve and enhance coastal public access, recreation, habitat, and scenic quality at South Ocean Beach
- Maintain current operational capacity of wastewater infrastructure to meet continued compliance with regulatory permits
- Protect the Lake Merced Tunnel, Westside Transport Box, and Westside Pump Station and associated facilities from damage due to shoreline erosion and storm and wave hazards
- Increase resilience to sea level rise
- Maintain emergency vehicle access
- Maintain dedicated service vehicle access to the Oceanside Treatment Plant, Westside Pump Station, and associated facilities
- Maintain visitor access to the San Francisco Zoo.

Figure ES-1: Wastewater Treatment Facilities Operated by SFPUC

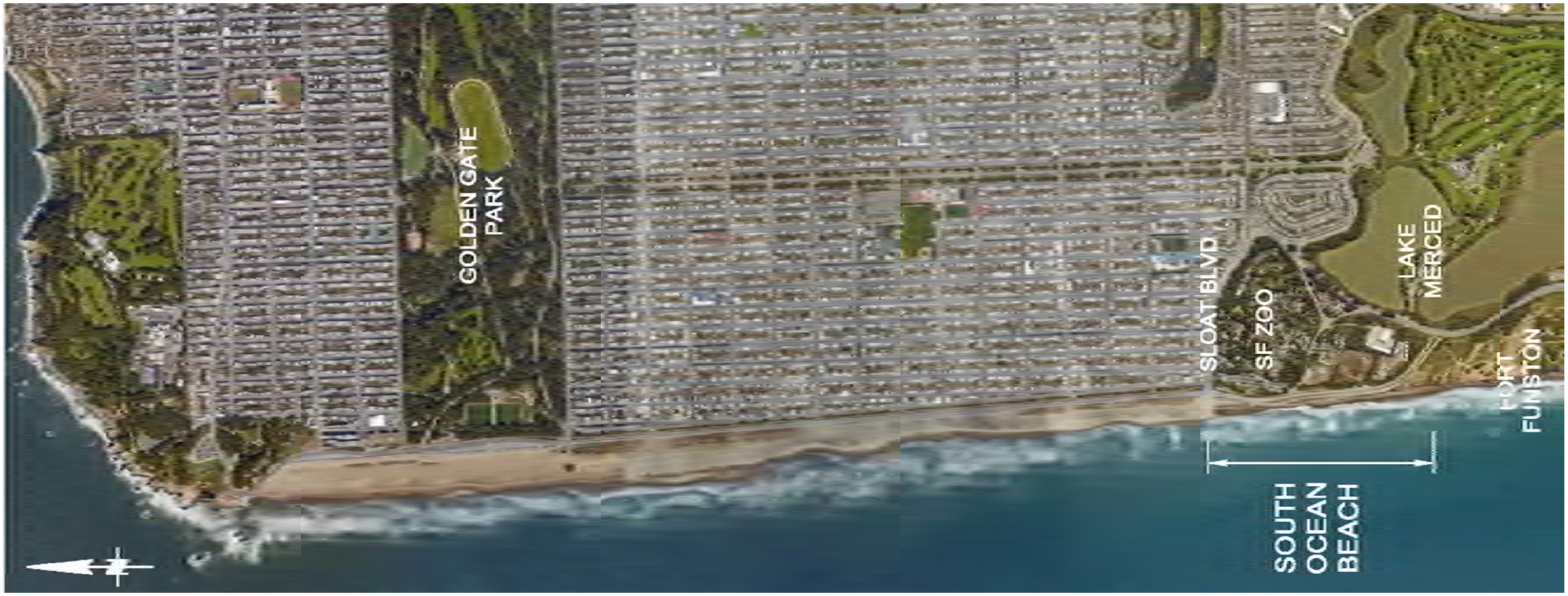


Figure ES-2: South Ocean Beach Location

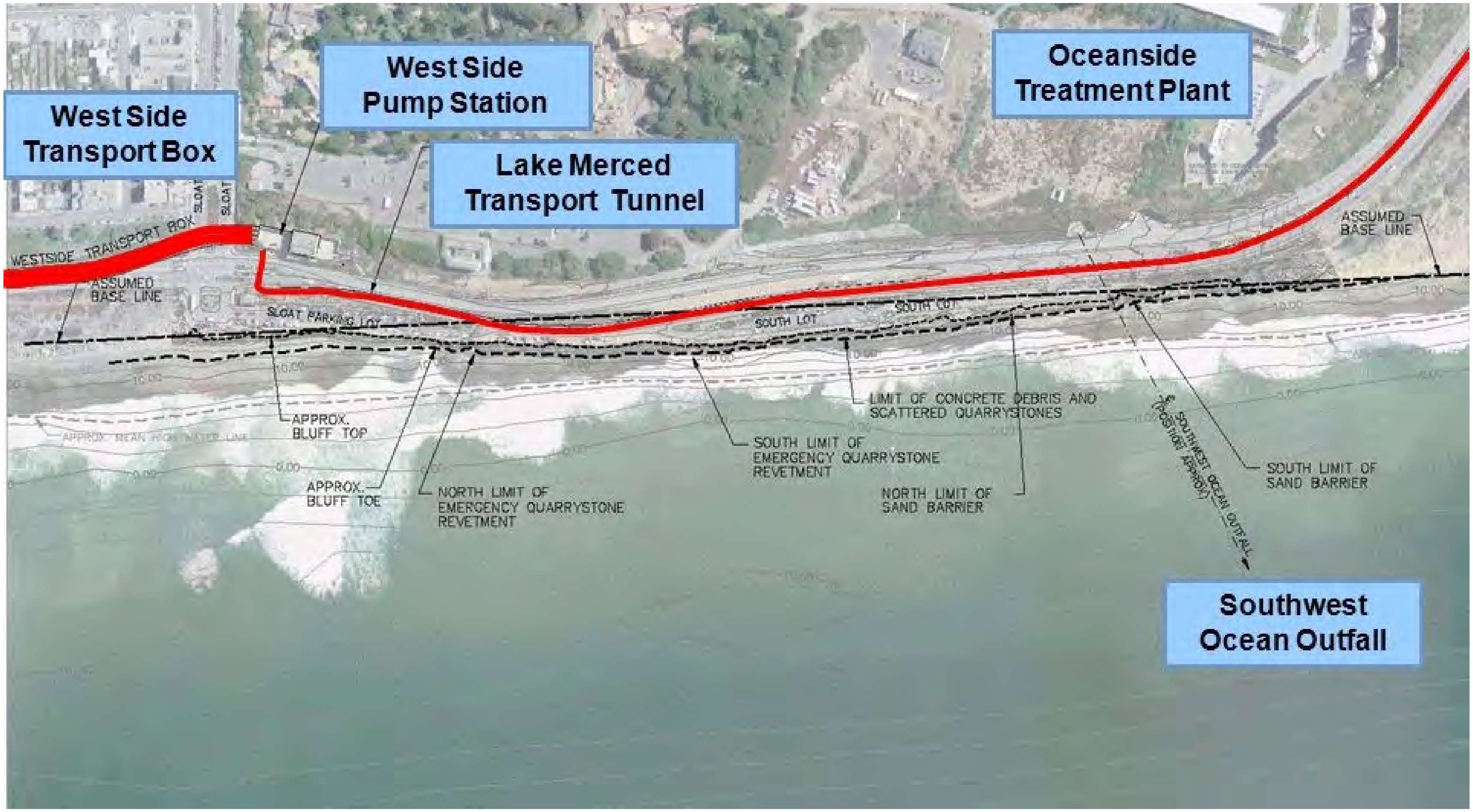


Figure ES-3: San Francisco West Wastewater Facilities

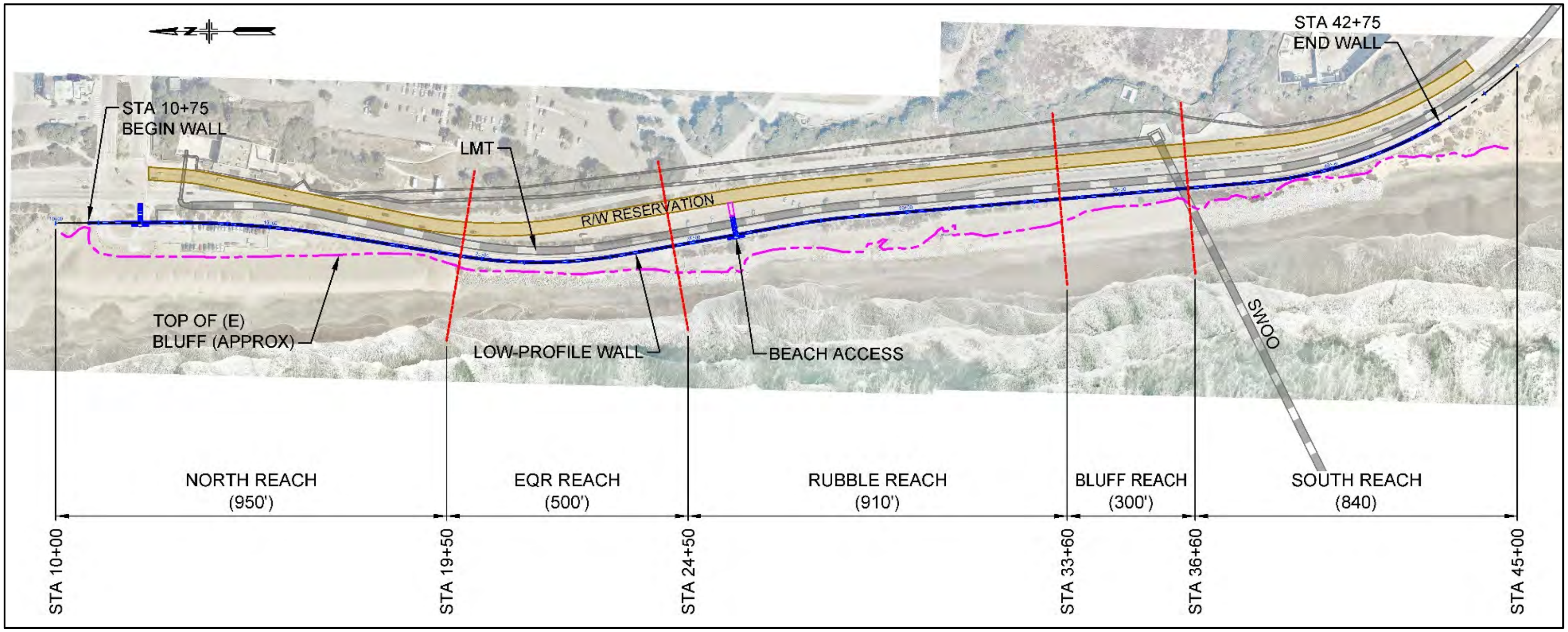


Figure ES-4: Low Profile Wall-Plan

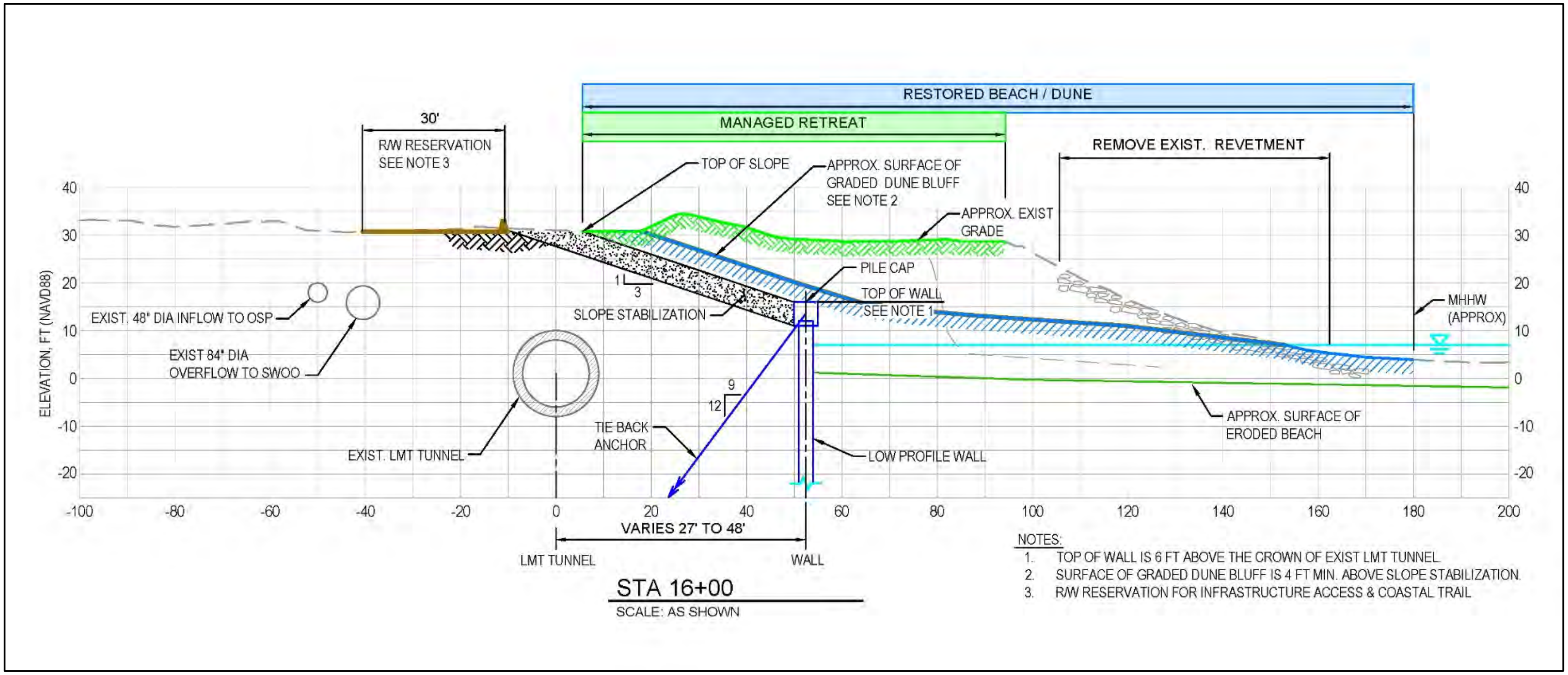


Figure ES-5: Low Profile Wall-Representative Section

1. Introduction

1.1. Purpose and Need

Currently, the existing wastewater infrastructure within the South Ocean Beach project area (see Figure 1-1 and Figure 1-2) is threatened by chronic coastal erosion of the beach and bluffs, caused by wave action and episodic bluff failures. Critical infrastructure, such as the Lake Merced Transport and Storage Tunnel (LMT), has the most immediate need for protection, as it is located immediately behind the bluff, and is in jeopardy of structural instability and eventual structural failure without some form of engineered protection. Failure of the LMT or parts thereof would cripple the functionality of the Oceanside Wastewater Infrastructure.

Over the years, federal, state, and local agencies have adopted erosion mitigation measures, aimed at protecting the existing shoreline and beach. These efforts have included depositing sand along the bluffs and/or offshore areas and the construction of engineered rock revetment (under emergency permit order).

Efforts in recent years have focused on the development of the Ocean Beach Master Plan (OBMP), which outlines coastal protection strategies along Ocean Beach through mid-century. The OBMP recommends management and protection measures for the existing essential wastewater infrastructure at Ocean Beach (including the LMT) in conjunction with increasing local access to the beach, improving aesthetics, and improving the beach's ecological functions. This project follows the OBMP guidance and focuses on a solution in the form of managed retreat of the Ocean Beach shoreline in response to chronic erosion and future sea-level rise.

In 2018, the SFPUC produced an Alternatives Analysis Report (AAR), entitled: "Alternative Analysis Report for Coastal Adaptation Strategies for South Ocean Beach Wastewater System." The AAR analyzed ten (10) options to address the threat of chronic erosion to the LMT and associated Oceanside facilities. The goal of the Alternatives Analysis phase of planning, and the subsequent report, was to analyze engineered solutions that would maintain the operational capacity of the Oceanside facilities, incorporate the guiding principles of the OBMP and comply with regulatory requirements. Through the Alternatives Analysis process the following goals and objectives were established for this project:

1.2. Project Goals and Objectives:

The goal of the project is to maintain function and operational capacity of the Oceanside wastewater infrastructure in a manner that incorporates the guiding principles of the OBMP and complies with regulatory requirements.

Primary objectives that the proposed project intends to achieve are:

- Maintain current operational capacity
- Increase resilience to sea level rise
- Comply with applicable laws and regulations
- Improve beach access, recreation and habitat
- Remove shoreline revetment and rubble

Four of the alternatives that were analyzed in the AAR, based on the above described project goals and objectives, were carried forward for detailed analysis. They include the following:

- Alternative A: Protect LMT with exterior low-profile wall
- Alternative B: Protect LMT with interior reinforcement + new storage
- Alternative C: Remove LMT + new tunnel alignment
- Alternative D: Remove LMT + new pump station, pipeline & storage

Each alternative was evaluated against eight criteria concerning cost, environmental impact, resilience to sea level rise, and operational complexity and all alternatives included ongoing beach nourishment. Alternative A, an exterior low-profile wall, ranked highest among the alternatives.

This Conceptual Engineering Report (CER) develops the chosen alternative from the Alternatives Analysis Report into a 10% design level and also presents conceptual designs for other elements of the project including traffic, landscaping, modified access to the zoo, modified access to the OSP and WSP facilities for SFPUC employees, and public recreational access to the beach and proposed relocated parking lot, bathroom, multi-use trail and beach.

Similar to the Alternatives Analysis Phase and subsequent report, the CER follows the OBMP guidance and focuses on a solution in the form of managed retreat of the Ocean Beach shoreline in response to chronic erosion and future sea-level rise. However, the report establishes the following additional criteria to be applied:

- Preserve the structural integrity of the LMT by protecting the tunnel against wave-, and erosion-related hazards. This is achieved by incorporation of a low-profile wall.
- Prevent uplift of the LMT due to buoyancy effected by high groundwater levels. This is achieved by incorporation of a soil cover (weight) over the LMT.
- Protect the LMT against seismic hazards, including liquefaction and lateral spreading. This is achieved by soil improvements around the LMT, anchored by the low-profile wall.
- Permit groundwater flow through the low-profile wall. This is achieved by limiting the tip elevation of every other pile of the (secant pile) low-profile wall.
- Permit wave runup on the beach and wave overtopping during extreme storm conditions. This is achieved by incorporation of a durable soil cover over the LMT.
- Protect existing wastewater infrastructure access and provide a public recreational trail with beach access as part of the infrastructure protection design. This is achieved via incorporation of an access road and trail along the coast and replenishment of sand on the beach via periodic beach nourishment.
- Protect the LMT during construction. This is achieved in several ways, but primarily by preventing construction-related dead and live loads atop the LMT and maintaining a minimum clear distance to the tunnel during installation of the low-profile wall and re-grading the coastal bluffs.

This criteria forms the basis for the conceptual engineering design for a low-profile wall design described in this CER (see Figure 1-3 and Figure 1-4).

1.3. Scope of Proposed Project

The CER summarizes existing conditions at SOB in terms of the beach and bluff topography, geology and stratigraphy, and natural hazards including erosion and coastal related hazards, and addresses the main engineering disciplines involved in the development of the conceptual design, which include Coastal, Geotechnical, Civil, and Structural Engineering. The CER additionally considers aspects of constructability, operations and maintenance, right-of-way, and environmental review.

The proposed scope of work presented in this report includes:

1) Installing a low-profile secant pile wall seaward of the LMT
2) Re-contouring the bluff at SOB and providing ongoing sand nourishment for the beach on an as-needed basis for increased recreational access
3) Removing the Great Highway between Sloat and Skyline Boulevards and completing intersection improvements at Sloat and the Great Highway and Skyline and the Great Highway to accommodate changed traffic flows.
4) Relocating the existing parking lot and restroom, currently located along the Great Highway, south of Sloat Boulevard.
5) Creating a multiuse recreational trail and access road for the SFPUC in place of the existing north bound lanes of the Great Highway
6) Providing access points to the beach for the public
7) Modifying the entrance to the zoo to accommodate changed traffic flows.
8) Modifying MTA bus turn-around at Sloat and Great Highway to account for changed traffic flows.
9) Providing landscaping and sand management strategies for the re-contoured bluff and the beach.
10) Removing the Existing Shoreline Revetments and Rubble

1.4. Approach to Developing Conceptual Design

To prepare the CER, the project team reviewed the OBMP, the Coastal Protection Measures and Management Strategy, the AAR as well as additional reports, conducted field investigations and communicated with the Wastewater Enterprise (WWE) operations personnel to further define the needs as well as the design criteria for the project. The CER provides an evaluation of coastal conditions and information on the geology and stratigraphy of the South Ocean Beach area, which forms part of the basis for the conceptual engineering design of the wastewater infrastructure protection/low-profile wall preferred alternative shown in Plan on Figure 1-3. The wall extends 3200 lin. ft. (over a half mile) and has 5 distinct reaches defined by similarity of beach and bluff conditions within each reach as delineated on the Plan.

The CER describes engineering aspects of the proposed project, which includes all the elements described in Section 1.3, with consideration to geotechnical-, civil-, structural-, and coastal engineering design. A summary of guidance with respect to constructability, operations and maintenance, and right-of-way is also provided along with a status on the project environmental review. The planned timeline for project execution and construction is provided in a project schedule and an estimate of project costs for construction, a list of project specifications, and concept-level drawings are also provided.

Figure 1-1: South Ocean Beach Location

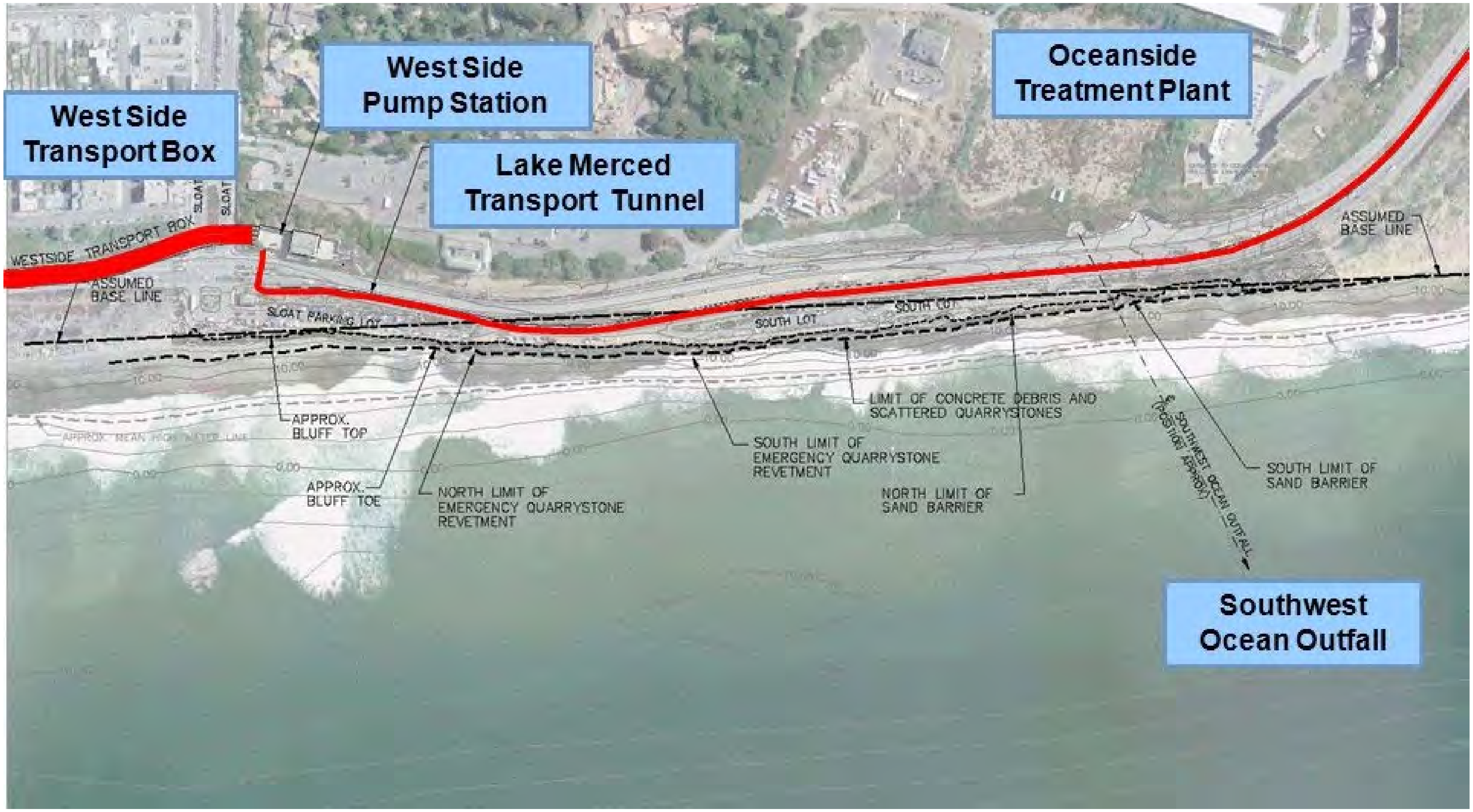


Figure 1-2: San Francisco West Wastewater Facilities

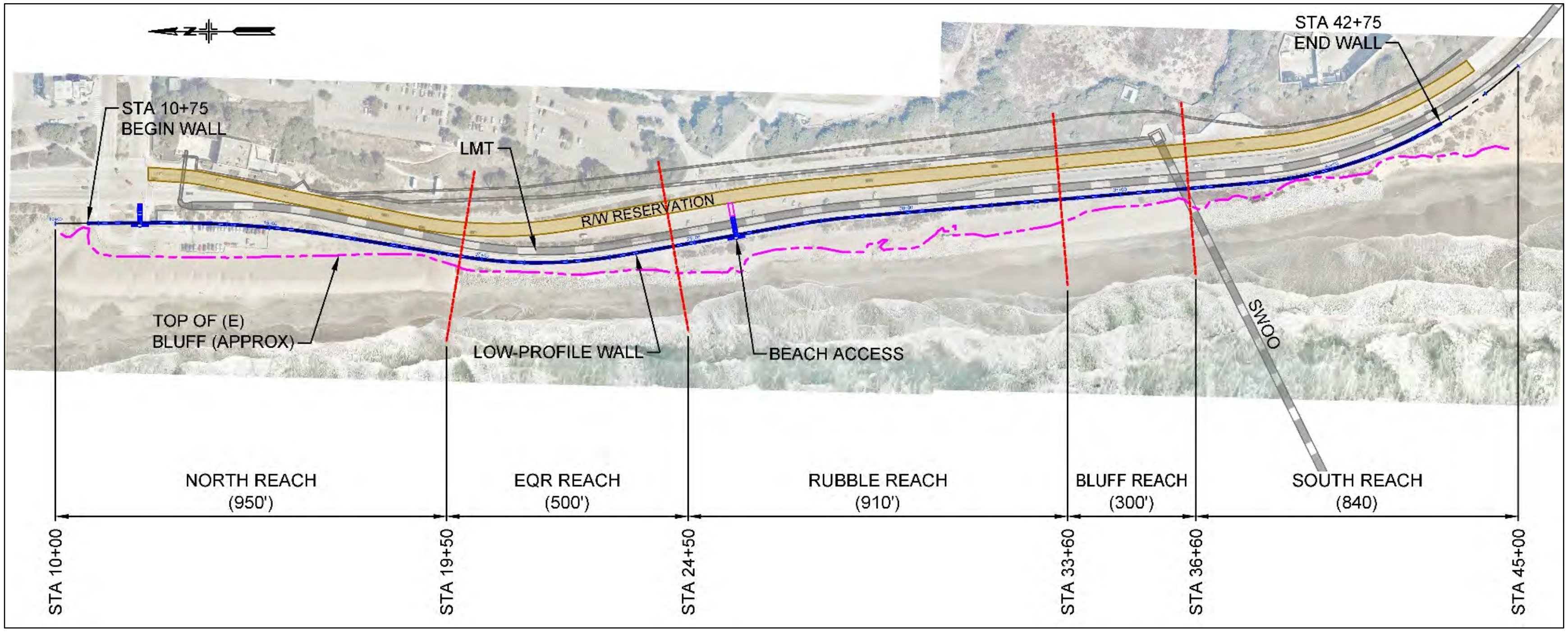


Figure 1-3: Low Profile Wall-Plan

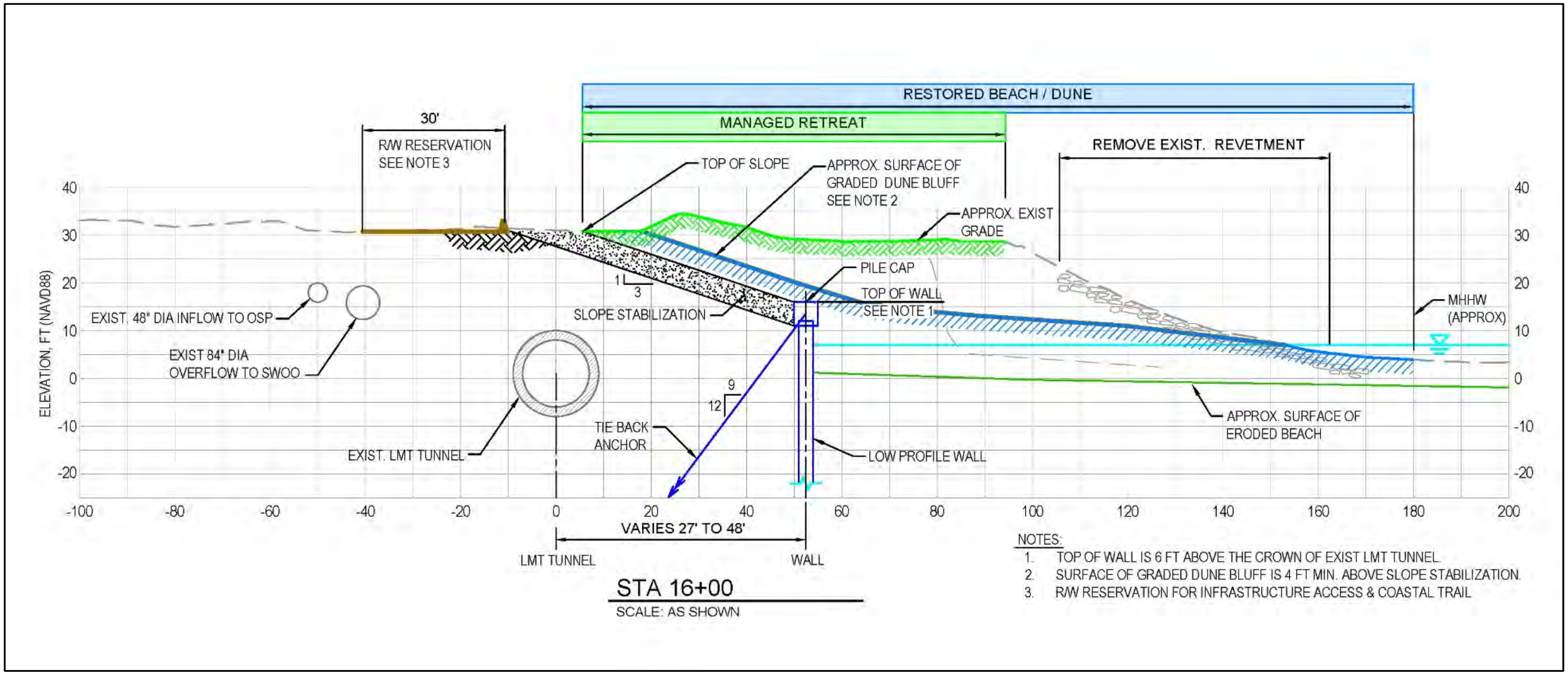


Figure 1-4: Low Profile Wall-Representative Section

2. Background

To comply with Clean Water Act regulations for improving water quality, the City of San Francisco adopted the San Francisco Wastewater Public Works Plan in 1979 to alleviate the impact of combined sewer overflows, which the California Coastal Commission subsequently approved. This led to construction of the following Oceanside Wastewater Infrastructure facilities (see Figure 1-2) to reduce combined sewer overflows:

- Westside Transport/Storage Box (WST)
- Westside Pump Station (WPS)
- Lake Merced Transport and Storage Tunnel (LMT)
- Oceanside Water Pollution Control Plant (OSP); and
- Southwest Ocean Outfall (SWOO)
- Buried utilities that connect and support the listed facilities

2.1. Existing System

The City is naturally divided by a ridgeline running roughly north-south into two main drainage watersheds: Bayside and Oceanside (see Figure 2-1). The Oceanside watershed drains towards the Pacific Ocean and occupies over 11,000 acres. It represents roughly 35 percent of the total City area and is divided into three sub drainage basins: Richmond, Sunset, and Lake Merced (from north to south). The Sewer System Master Plan was issued in 1974, which called for upgrading sewer infrastructure citywide to reduce pollution caused by combined sewer-stormwater overflows and to bring the city into compliance with the 1972 Clean Water Act. The City and County of San Francisco, through the Clean Water Program, constructed a major complex of sewer and stormwater infrastructure within the Oceanside Drainage Basin at Ocean Beach. This elaborate system, some of which is located underneath the Great Highway, reduced coastal water pollution events by a factor of 10. Its construction included the redesign of the Great Highway, the installation of existing dune-like sand embankments and considerable restoration of vegetation and amenities.

The Lake Merced Transport and Storage Tunnel (LMT) is an essential asset in the Westside wastewater collection system. The Park Merced, Stonestown, Ingleside, Oceanview, and Balboa Terrace neighborhoods are the primary sources of flow from the Lake Merced Watershed that converges at a three-compartment structure located near Lake Merced Blvd. Dry-weather flows are conveyed by the LMT to the OSP. When wet-weather flows exceed the capacity of the system (infrequent events), the combined sewage and stormwater discharges under a baffle and over a weir.

2.2. Current Operation

The primary function of the LMT is to transport collected combined wastewater flow from the Lake Merced Watershed to wastewater facilities for further treatment and to store peak flow during intense rain events to minimize local combined sewer discharges near Ocean Beach via the LMO. The LMT has a wet-weather storage capacity of 9.5 MG within the tunnel and 10.0 MG including connected sewers.

In dry weather operation, collected raw wastewater travels through a network of gravity sewer pipes (including the LMT) to the WST, a rectangular concrete structure under the Great Highway between Lincoln and Sloat Boulevards (as shown in Figure 2-2). Collected raw wastewater flows to the Westside Pump Station (WSS) at Sloat Boulevard, where flows are pre-treated (coarse solids removed) and pumped to the Oceanside Water Pollution Control Plant (OSP) for further treatment. The OSP receives 20% of the City’s total flows and treats 15 MGD and up to 175 MGD during rain events. The secondary-treated effluent is discharged approximately 4.5 miles out to the ocean through the 80-feet deep Southwest Ocean Outfall (SWOO).

During wet weather operation (intense rain events), the OSP attains maximum flow capacity (flows greater than 65 MGD), and the LMT storage capacity is utilized to reduce local Combined Sewer Discharges (CSD’s) events and discharge volume. CSD flows are decanted through a second chamber in the WST box and a second set of wet-weather pumps are permitted to discharge the decanted overflows to the SWOO directly. When Westside system’s capacity is exceeded (flows greater than 175 MGD), CSDs occur through seven (7) outfall structures, which are located at Ocean Beach, Mile Rock and China and Baker beaches.

2.3. Summary of Needs

The existing wastewater infrastructure within the South Ocean Beach project area is threatened by chronic coastal erosion of the beach and bluffs, caused by wave action and episodic bluff failures. Infrastructure, such as the LMT that is closest to the beach is in jeopardy of structural instability and eventual structural failure without engineered protection.

In addition to threatened wastewater infrastructures protection at South Ocean Beach is needed to address projected sea level rise impacts on:

- beach access and recreation
- beach and dune habitat

Over the years, federal, state, and local agencies have adopted erosion mitigation measures, aimed at protecting the existing shoreline and beach. These efforts have included depositing sand and sandbags along the bluff Toe, and the construction of engineered rock revetments (under emergency permit order).

Recent efforts have focused on the development of the Ocean Beach Master Plan (OBMP), SPUR (2012); and the South Ocean beach Coastal Protection Measures and Management Strategy, SPUR (2015), which outline coastal protection strategies along Ocean Beach through mid-century. The OBMP recommends management and protection measures for the existing essential wastewater infrastructure at Ocean Beach (including the LMT) in conjunction with improving access to the beach, shoreline aesthetics, and the beach's ecological functions. The objective will be achieved through the below scope of work that is described further in the following sections

1) Installing a low-profile secant pile wall seaward of the LMT
2) Re-contouring the bluff at SOB and providing ongoing sand nourishment for the beach on an as-needed basis for increased recreational access
3) Removing the Great Highway between Sloat and Skyline Boulevards and completing intersection improvements at Sloat and the Great Highway and Skyline and the Great Highway to accommodate changed traffic flows.
4) Relocating the existing parking lot and restroom, currently located along the Great Highway, south of Sloat Boulevard.
5) Creating a multiuse recreational trail and access road for the SFPUC in place of the existing north bound lanes of the Great Highway
6) Providing access points to the beach for the public
7) Modifying the entrance to the zoo to accommodate changed traffic flows.
8) Modifying MTA bus turn-around at Sloat and Great Highway to account for changed traffic flows.
9) Providing landscaping and sand management strategies for the re-contoured bluff and the beach.
10) Removing the Existing Shoreline Revetments and Rubble

2.4. Ocean Beach Master Plan

Key recommendations of the 2012 Ocean Beach Master plan for the South Reach (present South Ocean Beach project area) were to:

1. Reroute the Great Highway inland behind the San Francisco Zoo via Sloat Blvd. and Skyline Blvd., including:

    a) Closure of the Great Highway south of Sloat Blvd. and incorporation of a coastal trail,

    b) Reconfiguration of Sloat Blvd. and key intersections to create a safer, more efficient traffic flow,

    c) Consolidation of street parking, the L Taraval terminus and bicycle access along the south side of Sloat Blvd.; and

    d) Reconfiguration of the zoo's parking lot to permit access via Skyline Blvd. and Zoo Road.

2. Introduce a multipurpose coastal protection/restoration/access system, including:

    a) Managed retreat and phased removal of the Great Highway and adjoining parking lots, allowing erosion to proceed inland,

    b) Protection of the existing wastewater infrastructure including the LMT in place with a low-profile wall, a cobble berm, and beach nourishment with placed sand,

    c) Allowing storm surges to wash over the low-profile wall and dissipate toward higher ground; and

    d) Restore and revegetate the higher ground to enhance recreational and ecological functions.

The 2012 Ocean Beach Master Plan recommendations for South Ocean Beach stemmed from long-term progressive erosion along the Ocean Beach shoreline requiring active management and maintenance of the shoreline and adjoining Great Highway and parking facilities. Major coastal erosion events occurred in 2009, requiring placement of the 2010 Emergency Quarrystone Revetment (EQR), and in 2012 emergency repair with placement of a sandbag revetment to combat localized erosion. The Master Plan highlighted the vulnerability of the LMT to erosion, but opined that hard armoring of the bluff would increase the erosion potential due to lowering of the toe elevation permitting larger waves to attack the bluff and thereby increasing wave runup and overwash hazards. The recommended alternative to hard armoring was beach nourishment backed by a cobble berm, which would enable the beach profile to maintain the toe elevation of the back beach and relegate wave action and runup from the toe of the bluff to the much flatter beach profile. Figure 2-3 to Figure 2-5 shows the coastal profiles envisioned for the South Ocean Beach area in the 2012 Master Plan.

2.5. South Ocean Beach Coastal Protection Measures & Management Strategy

Under contract to SPUR, in a joint effort between SPUR, and engineering firms ESA PWA, Moffatt & Nichol, McMillen Jacobs Associates, and AGS, Inc., SFPUC promulgated the vision of the 2012 Ocean Beach Master Plan for South Ocean Beach into a coastal protection measures and management strategy, *SPUR (2015)*, for South Ocean Beach and the LMT critical infrastructure.

The work in *SPUR (2015)* further investigated the potential vulnerability of the LMT and concluded that:

- The South Ocean Beach area has been subject to chronic erosion at least since the 1850's.
- The varying geology, stratigraphy, and armoring relative to the LMT alignment results in a range of vulnerability along the Ocean Beach shore.
- Although the LMT is located below the beach level and inland of the existing bluff, it is located too far seaward to be sustained without adaptive measures to protect it from damage.
- The emergency protective measures, EQR and sandbag revetment, have been successful in protecting the LMT.
- Beach nourishment is expected to be adequate to mitigate risks over the next few years.
- Any damage to the LMT could impact coastal water quality, resulting in impacts to the environment and violation of regulatory permits.
- Relocation of the LMT is not feasible, and protecting the LMT in place is therefore the recommended solution.

The *SPUR (2015)* recommended solution was to incorporate a reinforced concrete low-profile vertical secant pile wall[1] to provide protection for the LMT, combined with a cap and/or sufficient overburden to resist buoyancy and provide vertical restraint of the LMT. This solution (Figure 2-6), in combination

[1] Various pile wall solutions, and cementitious grout soil mix wall were considered.

with beach nourishment was recommended to protect the LMT and the remaining wastewater infrastructure against anticipated coastal hazards.

Figure 2-1: Wastewater Treatment Facilities Operated by SFPUC

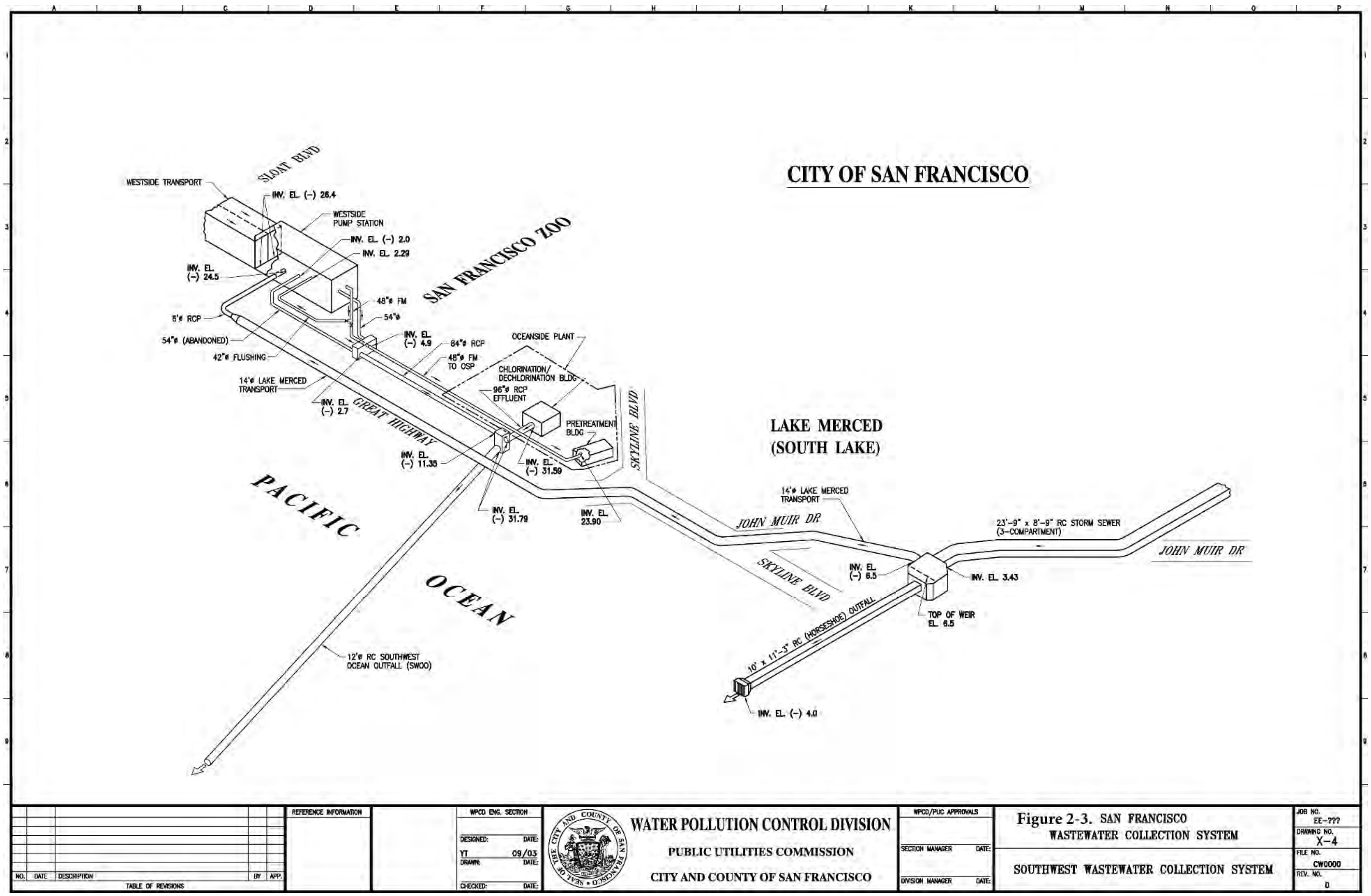


Figure 2-2: SFPUC Southwest Wastewater Collection System Schematic.

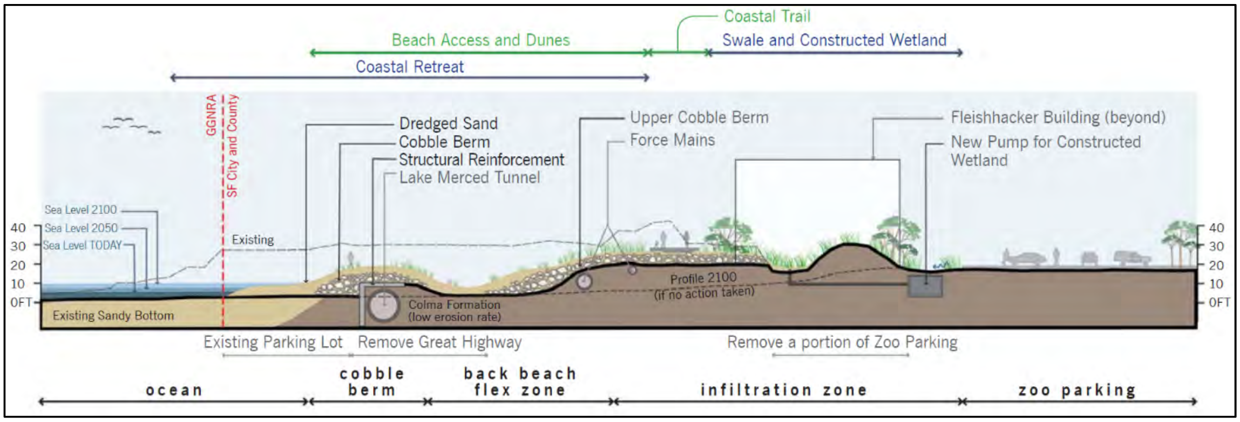


Figure 2-3: Summary of 2012 Master Plan Recommendations near Sloat Blvd/Great Highway Intersection

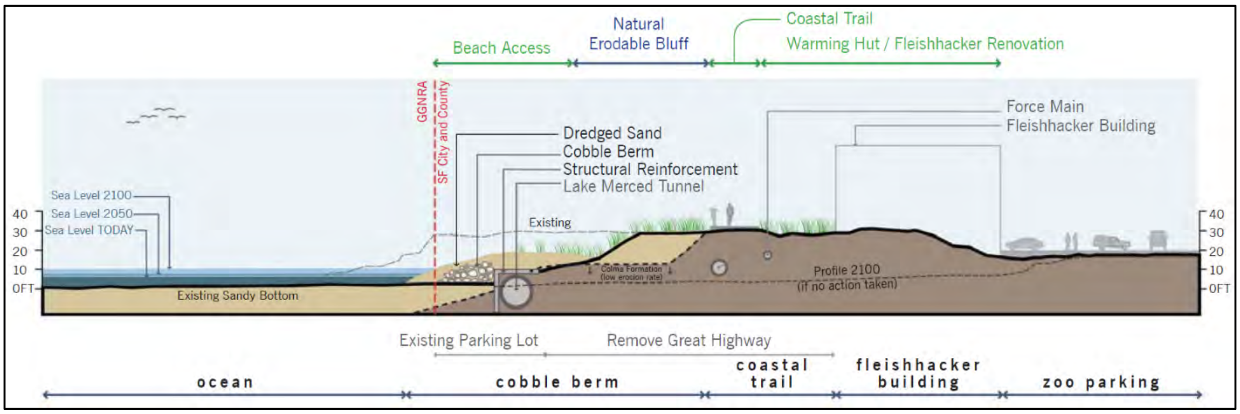


Figure 2-4: Summary of 2012 Master Plan Recommendations at Zoo Parking Lot

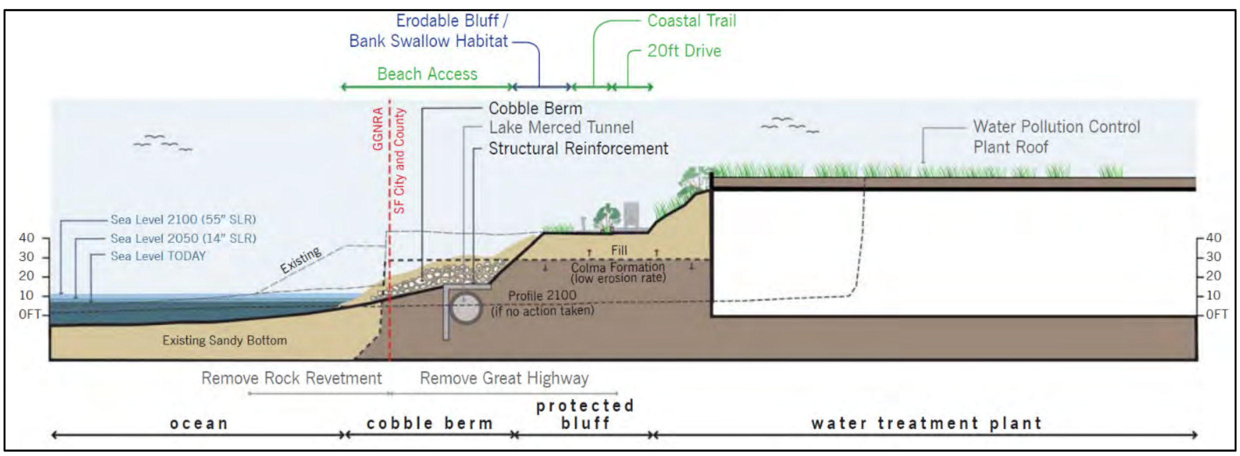


Figure 2-5: Summary of 2012 Master Plan Recommendations at Oceanside Water Pollution Control Plant.

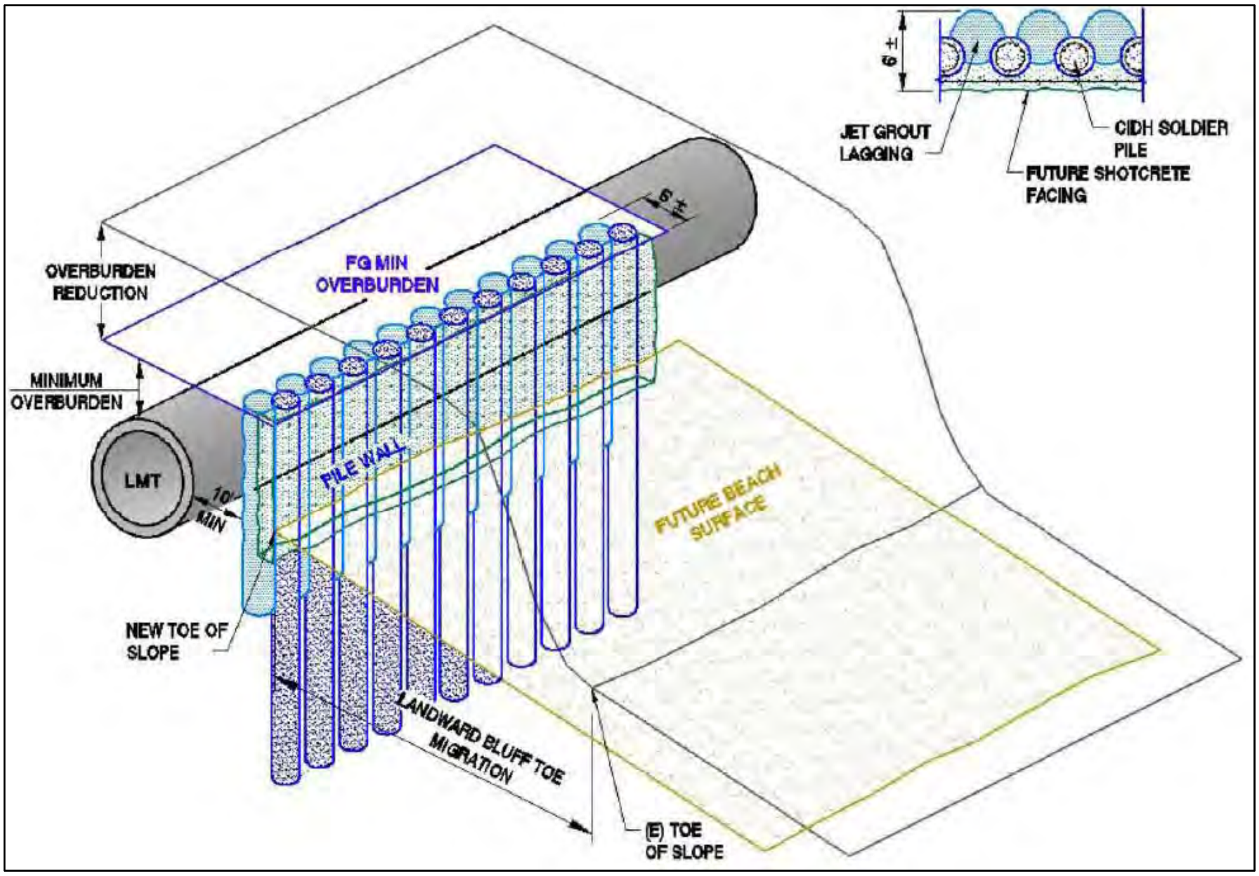


Figure 2-6: Isometric View of Reinforced Secant Pile Wall, reproduced from *SPUR (2015).*

3. Selected Alternative

The plan elements and alternative solutions to address the South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection Project needs for structural protection were evaluated in the Alternatives Analysis Report (AAR), SFPUC *(2018)*. The findings and recommendations of these are summarized in the following

3.1. Alternatives Analysis

As part of the realization of the *SPUR (2012)* master plan elements for South Ocean Beach, and implementation of the South Ocean Beach coastal protection measures & management strategy, *SPUR (2015)*, SFPUC consulted with the California Coastal Commission (CCC) to review the basis for a coastal development permit (CDP). The permit application process requires an alternatives analysis, which was conducted and presented in the 2018 Alternatives Analysis Report (AAR), *SFPUC (2018)*.

An Alternatives Analysis defines: 1) the purpose and need for the project; 2) Possible alternatives; 3) an analysis of the practicability of the alternatives; 4) identification of (beneficial or adverse) environmental impacts, leading to 5) identification of the least environmentally damaging alternative.

The alternatives analysis was conducted for the structural protection aspect of the project and a recognition that the other OBMP elements would be included during the CER phase. The AAR was developed with an emphasis on the following objectives:

- Maintaining the function and operational capacity of wastewater infrastructure facilities in a manner that incorporates the guiding principles of the Ocean Beach Master Plan and complies with regulatory requirements.
- Increasing resilience to sea level rise.
- Compliance with applicable laws and regulations.
- Improved beach access, recreation and habitat; and
- Removal of existing shoreline armoring.

The alternatives analysis considered ten options for structural protection, including a No Action alternative and various options involving onshore, offshore, structural, and non-structural elements. The alternatives were screened with respect to the above-mentioned objectives.

Out of the ten alternatives considered, the initial Fatal Flaw analysis screened out 6 alternatives, including the No Action Alterative. Four alternatives emerged as potentially feasible, which included:

- Alternative A – Protection of the LMT with an exterior low-profile wall.
- Alternative B – Protection of the LMT with internal reinforcement, including addition of storage capacity.
- Alternative C – Removal of the existing LMT and construction of a new tunnel alignment.
- Alternative D – Removal of the existing LMT and construction of a new pump station, pipeline and storage.

These alternatives were evaluated in terms of the criteria and weighting Factors presented in Table 3-1.

Table 3-1: Summary of Evaluation Criteria Weighting

Category	Criterion	Weight
Cost	Construction	20%
	Operations & Maintenance	5%
Environmental Impact	Construction	5%
	Post-Construction (beach width)	20%
Implementation/ Operational Complexity	Construction Risks	10%
	Operational Functionality	10%
	Right-of-Way Access	10%
	Resilience to Sea Level Rise	20%

The alternatives were then scored and ranked based upon their relative performance with the result presented in Table 3-2.

Alternative A, protection of the LMT with an exterior low-profile wall, emerged as the highest ranking alternative and was therefore carried forward as the chosen alternative.

Table 3-2: Alternatives Scoring and Ranking

Alt.	Cost		Environmental Impact		Implementation/Operational Complexity				Score/Rank		
	Construction (20%)	O&M (5%)	Construction (5%)	Post-Construction (20%)	Construction Risks (10%)	Operational Functionality (10%)	ROW Access (10%)	Resilience to Sea Level Rise (20%)	Raw Score	Weighted Score	Rank[1]
A	4	4	4	3	4	4	4	4	3.88	3.80	4
B	3	3	3	3	1	3	3	3	2.75	2.80	3
C	1	2	1	4	2	3	2	2	2.13	2.25	1
D	2	1	2	4	3	2	1	2	2.13	2.35	2

[1] Higher number indicates superior rank

3.2. Proposed Project

SFPUC review and approval of the AAR findings enabled the project to enter the Conceptual Engineering Phase including development of a Conceptual Engineering Report (CER). The conceptual engineering phase will develop the South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection project components (see Table 3-3) to a level sufficient to support the California Environmental Quality Act (CEQA) review and California Coastal Commission (CCC) Coastal Development Permit (CDP) application. The scope of work proposed for conceptual engineering includes:

- Summarize Background Information
- Characterize Existing Conditions, including SFPUC infrastructure facilities, easements and right-of-way, buried utilities and infrastructure, topographic data, and geologic profile and soil characteristics.
- Analyze Coastal Processes and Assess LMT Vulnerability
- Prepare Design Objectives and Preliminary Design Criteria, including Applicable Federal, State, and Local building codes; Geotechnical Design Criteria; Tunnel Stability Criteria; Structural Code requirements for static/dynamic loading and buoyancy; Drainage and Groundwater Control; Coastal Design Criteria; Project Life; and Maintenance assumption for beach nourishment over the project life.
- Develop Design Concepts for LMT Protection and Coastal Management
- Preparation of Concept Level Design Drawings
- Preparation of a CER CEQA Checklist and Environmental Review Technical Memorandum.

Table 3-3: Proposed Project Components

1. Low Profile Secant Pile Wall
2. Recontoured Bluff with Sand Management program
3. Existing shoreline revetment and rubble removal
4. Great Highway removal between Sloat Blvd and Skyline Drive
5. Intersection Improvement at Sloat/Great Highway & Skyline/Great Highway for Changed Traffic Flow
6. Relocated Sloat Restroom & Parking
7. Multi-Use Public Recreational Trail and wastewater Infrastructure Access Road
8. Multi-point Public Beach Access
9. Zoo Entrance Modifications for Changed traffic Flow
10. MTA transit Sloat Turnaround Modifications
11. Habitat restoration for recontoured bluff

4. Coastal Evaluation

Coastal engineering design information is summarized in the following. Refer to the Coastal Engineering Appendix for further details on background data and analyses.

4.1. Historical Background

Beach and dune fill activities started as early as the 1870's when dune stabilization and road improvements affected the shoreline position and shape, *M&N (1995)*. Significant beach and dune fill occurred in the period from 1900 to 1929 when the O'Shaughnessy Seawall was constructed (not within the project area). Between the years 1900 and 1956, a total volume of 2.35 million cubic yards (CY) of sand was placed as beach and dune fill. Since 1956, over one million cubic yards of sand was placed, primarily south of Lincoln Way. Additional sand may have been dumped on the beach and dunes in the late 1940's and early 1950's when nearby residential development peaked, requiring removal of sand dunes from lots. About 100,000 CY of sand was mined between 1963 and 1967 (mining started in 1953). Since completion of the Great Highway in 1929, significant beach and dune nourishment has taken place, while sand mining rates remained relatively low. The net volume increase to the beach and dunes by man since 1929 is estimated to be about 1.3 million cubic yards.

4.2. Area Geology and Morphology

The portion of the LMT alignment located within the South Ocean Beach project area passes through dune sands, Colma Formation, and artificial fill. Bluffs along the project area are in the Colma Formation, interspersed with artificial fill, riprap shore protection and rubble.

Sand on Ocean Beach originates from several different sources, including sediment from bluff erosion, sand that migrates to the beach from the San Francisco Bar, and sand from other sources imported for beach nourishment.

4.2.1. Bluff Material

The bluff material along the project area is defined as the Colma Formation, which consists of moderately cemented to uncemented sand deposits with varying amounts of clay and silt. The Colma Formation varies in thickness from about 25 feet to 40 feet and is overlain by a few feet of recent dune sand and artificial fill.

4.2.2. Beach Material

Median grain sizes for Ocean Beach are summarized in *M&N (1995)*. The majority of samples are representative of *Medium Sand*, with a few samples of *fine* and *coarse* to *very coarse sand*.

4.3. Bluff Retreat

4.3.1. Short-Term Bluff Recession Rates

The USGS conducted a comprehensive coastal processes study at Ocean Beach from 2004 to 2006, *USGS (2007)*, which concluded the following:

- Single storm events can cause shoreline retreat of over 30 feet.
- Very strong El Niño conditions such as the winter of 1997-98 can double the average shoreline retreat.

In connection with emergency repairs along the Great Highway in response to erosion during the 2009-10 winter, the recurrence and magnitude of episodic bluff failures was studied. Figure 4-1 summarizes findings from *M&N (2010)* compared with findings from earlier studies. The results indicate that bluff failures on the order of 10 feet can occur every 5-8 years on average; 20 feet of bluff erosion every 8-17 years on average; and 40 feet of bluff erosion every 25-33 years on average.

In addition, Prof. Sitar of University of California (Berkeley) together with USGS conducted a detailed study on recession of bluffs composed of weakly cemented and moderately cemented material (Merced Formation), *JOG (2008)*. The study utilized LiDAR surveys to identify episodic bluff failures due to wave action and precipitation runoff. The findings are also summarized in Figure 4-1. As seen in the figure, bluff retreat rates associated with failures in the moderately cemented bluffs are generally consistent with the findings in *M&N (2010)*. Dr. Sitar's data is situated at the lower end of the curve because the data spanned a shorter duration, between 2002 and 2006. Bluff retreat in weakly cemented material (provided for comparison) exhibits higher recession rates as this material is more erodible.

4.3.2. Long-Term Bluff Recession Rates

Shoreline mapping was conducted in *M&N (1994)*. The analysis determined the location of the toe of the bluff for years: 1938, 1948, 1959, 1970, 1971, 1978, 1980, 1985, 1986, 1992, and 1993.

An updated aerial photo analysis was performed for the present study; the bluff retreat rates are summarized in Figure 4-2. These rates are determined based on a linear trend of data for the location of the bluff edge over the years from 1938 to 2019. Along the central and northern part of the project area where the shoreline has been maintained since 1938, the rate of retreat is near zero or slightly positive (blue bars) due to armoring and accumulation of debris. This indicates a stable shoreline enabled by manmade shore protective structures.

Transitioning to the southern part of the project area where the bluff is unprotected, the rate of retreat increases progressively. The colored bars indicate the rate of retreat ranging from 0.5 feet per year (light yellow) to 2.4 feet per year (purple).

These findings are consistent with the shoreline change rates determined in *M&N (2005)*, which established the following trends:

- 0.5 to 2.6 feet per year of recession for the unprotected bluffs reach, south of the project area
- 1.2 feet per year of recession to 0.7 feet per year of advance within the South Ocean Beach project area
- 0.9 to 1.8 feet per year of advance for reaches north of the project area.

4.4. Erosion Patterns

Wave action brings beach material into suspension and is active across the shore and along the shore. Only a limited fraction of sandy material eroded from bluffs contributes to beach nourishment. The majority of the bluff material is fine and swiftly removed by wave-driven longshore sediment transport.

The presence of shore armoring such as vertical walls and rock revetment tends to result in lowering of the beach level and narrowing of the beach. This effect has been noted in front of the EQR structure (see Figure 1-3).

4.5. Sea-Level Rise

Current guidance for California recommends evaluation of SLR impacts using a scenario-based analysis. This method is founded on the approach by the Intergovernmental Panel on Climate Change (IPCC) to understand how SLR and other drivers interact to threaten health, safety, and resources of coastal communities. Comprehensive SLR guidance for California was first developed by the National Research Council, *NRC (2012).* The guidance relied on the best available science at the time to identify a range of sea-level rise scenarios including high, low, and intermediate projections, taking into account regional factors such as El Niño and extreme storm events that affect ocean levels, precipitation, and storm surge. This approach allows planners to understand the full range of possible impacts that can be reasonably expected based on the best available science and build an understanding of the overall risk posed by potential future SLR.

The best available science and most recent guidance adopted by the California Coastal Commission is provided in *OPC (2018)* and has been adopted for this vulnerability assessment. Table 4-1 summarizes SLR scenarios adopted from *OPC (2018)* for time horizons out to 2150. The columns outlined in dark blue reflects the OPC guidance for risk levels, which include low risk aversion, medium to high risk aversion, and extreme risk aversion. The SLR scenario adopted for this analysis is the *Medium – High Risk Aversion* scenario, assuming high greenhouse gas (GHG) emissions.

Table 4-1: Sea-Level Rise Projections for San Francisco Bay Area, *OPC (2018)*.

		Probabilistic Projections (in feet) (based on Kopp et al. 2014)						H++ scenario (Sweet et al. 2017) *Single scenario
		MEDIAN	LIKELY RANGE			1-IN-20 CHANCE	1-IN-200 CHANCE	
		50% probability sea-level rise meets or exceeds…	66% probability sea-level rise is between…			5% probability sea-level rise meets or exceeds…	0.5% probability sea-level rise meets or exceeds…	
					Low Risk Aversion		Medium - High Risk Aversion	Extreme Risk Aversion
High emissions	2030	0.4	0.3	-	0.5	0.6	0.8	1.0
	2040	0.6	0.5	-	0.8	1.0	1.3	1.8
	2050	0.9	0.6	-	1.1	1.4	1.9	2.7
Low emissions	2060	1.0	0.6	-	1.3	1.6	2.4	
High emissions	2060	1.1	0.8	-	1.5	1.8	2.6	3.9
Low emissions	2070	1.1	0.8	-	1.5	1.9	3.1	
High emissions	2070	1.4	1.0	-	1.9	2.4	3.5	5.2
Low emissions	2080	1.3	0.9	-	1.8	2.3	3.9	
High emissions	2080	1.7	1.2	-	2.4	3.0	4.5	6.6
Low emissions	2090	1.4	1.0	-	2.1	2.8	4.7	
High emissions	2090	2.1	1.4	-	2.9	3.6	5.6	8.3
Low emissions	2100	1.6	1.0	-	2.4	3.2	5.7	
High emissions	2100	2.5	1.6	-	3.4	4.4	6.9	10.2
Low emissions	2110*	1.7	1.2	-	2.5	3.4	6.3	
High emissions	2110*	2.6	1.9	-	3.5	4.5	7.3	11.9
Low emissions	2120	1.9	1.2	-	2.8	3.9	7.4	
High emissions	2120	3	2.2	-	4.1	5.2	8.6	14.2
Low emissions	2130	2.1	1.3	-	3.1	4.4	8.5	
High emissions	2130	3.3	2.4	-	4.6	6.0	10.0	16.6
Low emissions	2140	2.2	1.3	-	3.4	4.9	9.7	
High emissions	2140	3.7	2.6	-	5.2	6.8	11.4	19.1
Low emissions	2150	2.4	1.3	-	3.8	5.5	11.0	
High emissions	2150	4.1	2.8	-	5.8	5.7	13.0	21.9

4.6. Sea-Level Rise Scenarios

Coastal erosion is projected to increase with sea-level rise. Additional factors that can exacerbate coastal erosion events include high tides, storm surge, El Niño effects, and elevated groundwater tables. These elements can increase the severity and frequency of coastal erosion and bluff recession.

- Tides occur regularly with about two high tides and two low tides each day. The highest tides (spring tides) occur twice a month during the full moon and the new moon. Around December and January when a new or full moon occurs at the same time as the moon is at its closest to the earth, the tides run higher. These higher perigean spring tides are commonly known as King Tides.

- Storm surge can occur as a combination of wind shear over the water and low atmospheric pressure.

- El Niño (and La Niña) are cycles of warming and cooling of the ocean, typically lasting 9 to 12 months. They often commence in June or August and reach their peak during December through April, and subsequently decay over May through July of the following year. Their periodicity is irregular, occurring every 3 to 5 years on average. The warming associated with El Niño produces a rise of the ocean level, which can be on the order of 6 to 13 inches. The period of elevated (or lowered) ocean levels can be on the order of months, while the peak highs and lows occur on a scale of days to weeks.

- Elevated Groundwater Tables. Sea-level rise can cause seawater intrusion into coastal aquifer systems and can raise shallow groundwater tables. These can short circuit levee systems and contribute to inland flooding and/or impacts to buried infrastructure.

The historically highest water levels recorded around the Bay Area occurred in January of 1983 and were due to a combination of King Tides and rise of the ocean level due to a pronounced El Niño episode. Based on the tide station at San Francisco Golden Gate (NOAA Station 9414290) the estimated water level at South Ocean Beach would have been around +8.82 feet MLLW in January 1983.

Table 4-2 provides a breakdown of tidal datums and extreme water levels for existing conditions, and estimated water levels with SLR projected for 2030, 2050, and 2100. The sea-level rise projection reflects the *Medium to High Risk Aversion* OPC Scenario, assuming *High Emissions*.

The CCSF Capital Planning Committee (CPC) sea-level rise guidance provided in *ONESF – Building Our Future* details sea-level rise scenario selection and design tide calculation.

The 2015 CPC Guidance recommended the NRC 2012 sea level rise projections for the likely and upper range scenarios for guiding design and adaptation decisions, respectively. To accommodate the updated science, and the 2018 State Guidance, the CPC Sea Level Rise Checklist has been updated to include the likely and 1-in-00 chance values for RCP 4.5 and RCP 8.5. For the likely values, *NRC (2012)* recommended using 36 inches at 2100. This compares well with the updated science, which ranges from 33 inches under RCP 4.5 to 41 inches under RCP 8.5. In the 2015 CPC Guidance, the likely value was recommended for most design decisions; therefore, little to no change it needed for compliance with the updated science. For the upper range values which are most often used for adaptation planning, *NRC (2012)* recommended using 66 inches of sea level rise by 2100.

The 1-in-200 chance values for RCP 4.5 and RCP 8.5 both exceed this value, with 71 inches and 83 inches of sea level rise by 2100, respectively. Although this change is minor, it does represent an increase in the amount sea level rise recommended for use in adaptation planning.

Table 4-2: Tidal and Extreme Water Level Datums, SLR Scenarios.

Condition	Existing	Sea Level Rise (feet) by [1]		
		2030	2050	2100
		0.8	1.9	6.9
	Water Level (feet NAVD88)			
1% Annual Chance Storm	+8.7	+9.5	+10.6	+15.6
King Tides	+7.2	+8.0	+9.1	+14.1
MHHW	+5.9	+6.7	+7.8	+12.8
MHW (Shoreline)	+5.3	+6.1	+7.2	+12.2
MTL	+3.3	+4.1	+5.2	+10.2
MSL	+3.2	+4.0	+5.1	+10.1
MLW	+1.2	+2.0	+3.1	+8.1
MLLW	+0.1	+0.9	+2.0	+7.0
[1] State of California Sea-Level Rise Guidance, OPC (2018) Update.				

4.6.1. Trends in Local Relative Sea Level

Local relative sea-level rise reflects the chance in sea-level due to climate change and vertical movement of the landmass. Vertical land motion (VLM) can occur due to tectonic activity, isostatic rebound which is adjustment of the earth due to compression from the ice masses during the last ice age, and due to subsidence.

Estimates of vertical land motion (VLM) for California and Nevada from *JGR (2016)* indicate that the South Ocean Beach area is subsiding by 0.5 mm per year. At this rate the land will sink by 1.6 inches by 2100.

The vertical land motion in this case adds to the relative sea level rise at South Ocean Beach, but the effect is limited as the projected rise in ocean level is an order of magnitude larger than the VLM.

4.7. Coastal Engineering Design

4.7.1. Design High Water Level

The design high water level is the Still Water Elevation (SWEL) including adjustments for wave setdown, wave setup, and surf beat as indicated in Table 4-3.In addition sea-level rise should be added to the design high water level.

Table 4-3: Design High Water Level.

Water Level and Wave Effects	Contribution to Design Water Level
Still Water Elevation	+8.69 feet NAVD88
Wave setdown	-1.61 feet
Wave setup	3.00 feet
Surf Beat	1.69 feet
Total (without sea-level rise)	+11.77 feet NAVD88

4.7.2. Wave Action

Table 4-4 provides estimates of offshore significant wave height extremes based on extreme-value analysis (EVA) of the wave data from NDBC Station 46026.

Table 4-4: Significant wave height extremes, NDBC Station 46026

Return Period (years)	Offshore Significant Wave Height (feet)	90% Confidence Interval	
		Lower Bound	Upper Bound
5	22.9	20.9	24.8
10	25.4	22.9	28.0

Return Period (years)	Offshore Significant Wave Height (feet)	90% Confidence Interval	
		Lower Bound	Upper Bound
50	31.1	27.2	35.0
100	33.5	29.0	38.0

Wave transformation by refraction and shoaling occur over the complex bathymetry around the San Francisco Bar, but note that waves become depth-limited and will break and reform in the fairly wide surf zone at South Ocean Beach. The governing design wave for the low-profile wall alternative is therefore the maximum breaking wave supported by the design water depth at the wall.

4.7.3. Scour Elevations

Potential scour at the toe of the wall is assessed in the following. Using the method of Fowler (1992), the maximum scour depth can be estimated as:

$$S_{max} = H_0 \sqrt{\frac{22.72 \cdot d_s}{L_0} + 0.25}$$

Where H_0 is the zero moment wave height, L_0 is the deep water wave length, and d_s is the pre-scour water depth at the wall. This method estimates a toe scour elevation of approximately +1.4 feet NAVD88.

4.7.4. Wave Runup

Estimated elevations of wave runup on the slope above the crest of the wall are summarized in Table 4-5. The first column of elevations identifies the wall crest elevation, which transitions over the reaches between STA 12+20 and STA 42+60 (see Figure 1-3). The subsequent columns indicate the wave runup elevations for no sea-level rise followed by sea-level rise in increments of two feet.

4.7.5. Wave Loads

Wave loads on the low-profile wall were estimated based on *ASCE (2016).*

Figure 4-3 shows how ASCE 7-16 defines the breaking wave load on a wall as the sum of a hydrostatic pressure and a dynamic pressure component.

The maximum combined dynamic and static pressure, P_{max}, is given by:

$$P_{max} = C_p \gamma_w d_s + 1.2 \gamma_w d_s$$

Where: C_p is a dynamic pressure coefficient, γ_w is the unit weight of water, and d_s is the still water depth at the base of the wall. The estimated maximum pressure is: P_{max} = 10.5 psi.

The breaking wave force per unit length of wall, F_t, is given by:

$$F_t = 1.1C_p\gamma_w d_s^2 + 2.4\gamma_w d_s^2$$

The estimated breaking wave force is: F_t = 19,0 kip/ft.

Table 4-5: Wave Runup Elevations for Project Reach Segments.

<table>
<tr><th rowspan="3">Station</th><th rowspan="3">Segment</th><th colspan="5">Elevation (feet NAVD88)</th></tr>
<tr><th rowspan="2">Wall Crest</th><th colspan="4">Wave Runup at Crest</th></tr>
<tr><th>No SLR</th><th>0.8’ SLR</th><th>1.9’ SLR</th><th>6.9’ SLR</th></tr>
<tr><td>10+75</td><td rowspan="2">North Reach</td><td>+14.50</td><td>+21.5</td><td>+22.5</td><td>+24.0</td><td>+30.4</td></tr>
<tr><td rowspan="2">19+50</td><td rowspan="2">+15.50</td><td rowspan="2">+21.3</td><td rowspan="2">+22.2</td><td rowspan="2">+23.6</td><td rowspan="2">+30.2</td></tr>
<tr><td rowspan="2">EQR Reach</td></tr>
<tr><td rowspan="2">24+50</td><td rowspan="2">+16.10</td><td rowspan="2">+21.2</td><td rowspan="2">+22.1</td><td rowspan="2">+23.5</td><td rowspan="2">+30.0</td></tr>
<tr><td rowspan="2">Rubble Reach</td></tr>
<tr><td rowspan="2">33+60</td><td rowspan="2">+17.10</td><td rowspan="2">+21.0</td><td rowspan="2">+21.9</td><td rowspan="2">+23.2</td><td rowspan="2">+29.7</td></tr>
<tr><td rowspan="2">Bluff Reach</td></tr>
<tr><td rowspan="2">36+60</td><td rowspan="2">+17.75</td><td rowspan="2">+21.0</td><td rowspan="2">+21.9</td><td rowspan="2">+23.1</td><td rowspan="2">+29.5</td></tr>
<tr><td rowspan="2">South Reach</td></tr>
<tr><td>42+75</td><td>+18.50</td><td>+21.1</td><td>+21.8</td><td>+23.0</td><td>+29.4</td></tr>
</table>

4.7.6. Scour at Wall Crest

An analysis was conducted to assess the spatial extent of wave overtopping past the crest of the low-profile wall with respect to sea-level rise, and the potential for scouring behind the wall if the crest is not protected.

The results of the analysis are summarized in Figure 4-4, which shows that substantial scour behind the wall could develop if the slope at the crest is not protected. For the scenarios with 1.9’ to 6.9’ of sea-level rise (SLR), it is estimated that the ground level behind the wall could erode down to approximately El. 0.0 feet NAVD88 and expose the LMT. Progressive erosion would be noted from present day to 0.8’ of SLR. It is therefore imperative that the slope above the crest of the wall be protected to prevent loss of cover material over the LMT and potential undermining of the coastal trail at the crown of the slope. The estimated spatial extent of wave overtopping is about 15 feet for 0.8' of SLR, 30 feet for 1.9' of SLR, and 45 feet for 6.9' of SLR.

4.7.7. Beach Nourishment

A preliminary analysis of the longevity of beach nourishment is included in the Coastal Engineering Analysis, Appendix A.

Beach nourishment serves to protect upland structures and infrastructure from the effects of storms by building a beach, which acts as a buffer. While not mutually exclusive, three basic versions of beach nourishment can be identified:

- Placement of material (generally sand) offshore, attenuating wave energy and reducing wave impacts on the shoreline.
- Placement of material on the beach with a focus on the intertidal and dry-beach zones, thereby constructing a wider (and/or higher) beach to act as a buffer between waves and the upland infrastructure.
- Placement of material on dunes above the dry beach, again to provide a buffer between the waves and upland infrastructure.

Only the second of these provides a wider beach with significant recreational benefits. Other potential benefits can include habitat restoration. The low profile wall will act as a final line of defense in case of an extreme erosional event.

The level of storm protection provided by a nourishment project cannot be calculated absolutely because of uncertainties in the frequency and intensity of storms and the subsequent effects after sand is transported away from the nourished beach. The level of protection may be reduced in the aftermath of a major storm, and it may also be compromised if periodic nourishment is not performed when scheduled.

The preliminary analysis found that the longevity of beach nourishment can be managed by the volume of material placed and the frequency of placements. The analysis found that the longevity also depends on the median grain size of the material placed, i.e. larger diameter material will tend to be more stable. However, the analysis concluded that in the range of material from fine and medium sand to coarse sand, the mean diameter does not affect the longevity significantly as the wave climate at Ocean Beach is able to mobilize and transport material in the range from fine to coarse sand.

Table 4-6 summarizes recommended beach nourishment volumes and frequency to manage the beach width out to Year 2100, assuming a *Medium to High Risk Aversion* to sea-level rise based on the RCP 8.5 sea-level rise scenario (refer to Table 4-1). It is also recommended that the beach be nourished at the time the low profile wall is constructed to provide an initial dry beach width of at least 80 ft.

Table 4-6: Summary of Recommended Beach Nourishment Volumes and Frequency.

Beach Nourishment	Nourishment Frequency	
	Every 5 Years	Every 10 Years
Volume	125,000 – 165,000 CY	250,000 – 330,000 CY

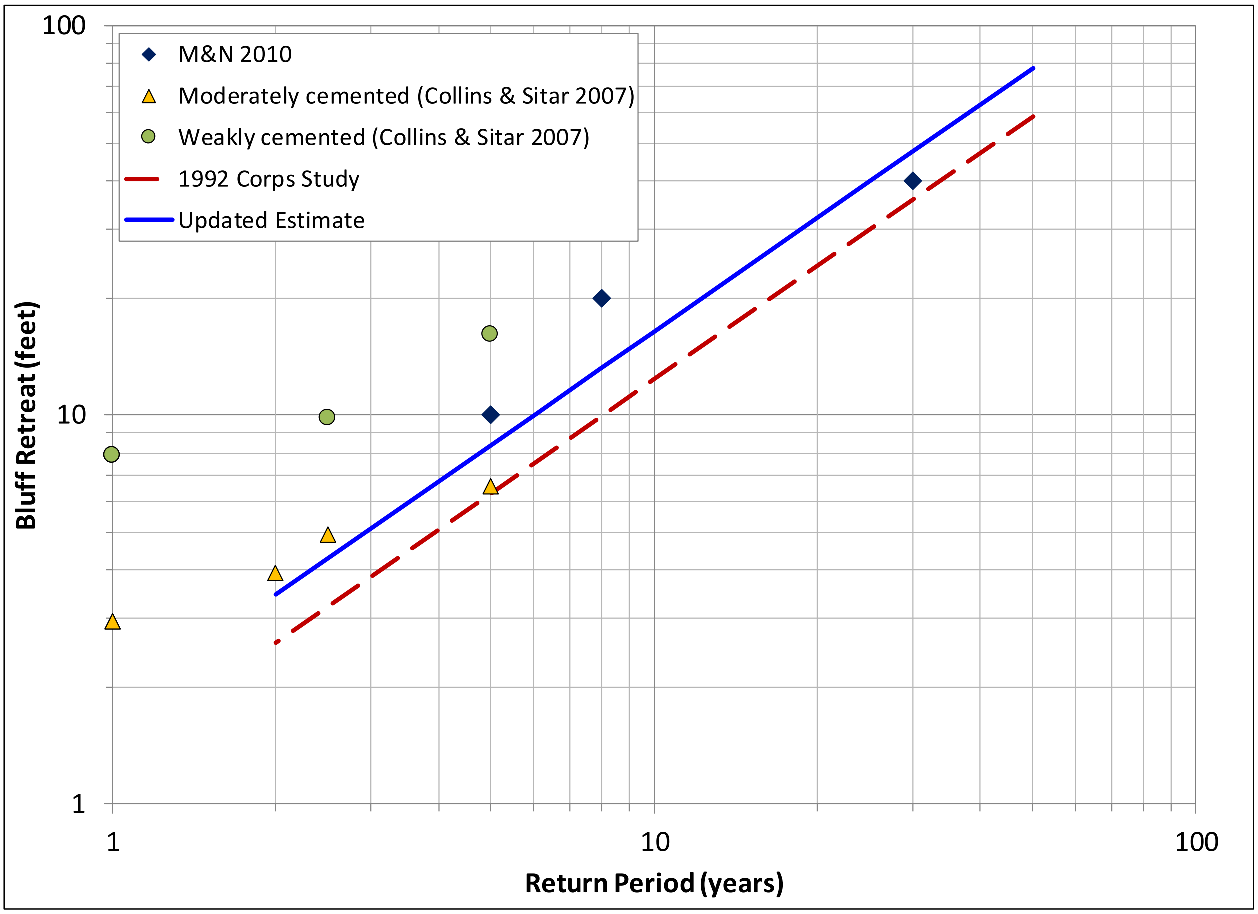


Figure 4-1: Estimated Return Period Bluff Retreat Values

Figure 4-2: South Ocean Beach Bluff Retreat Rates.

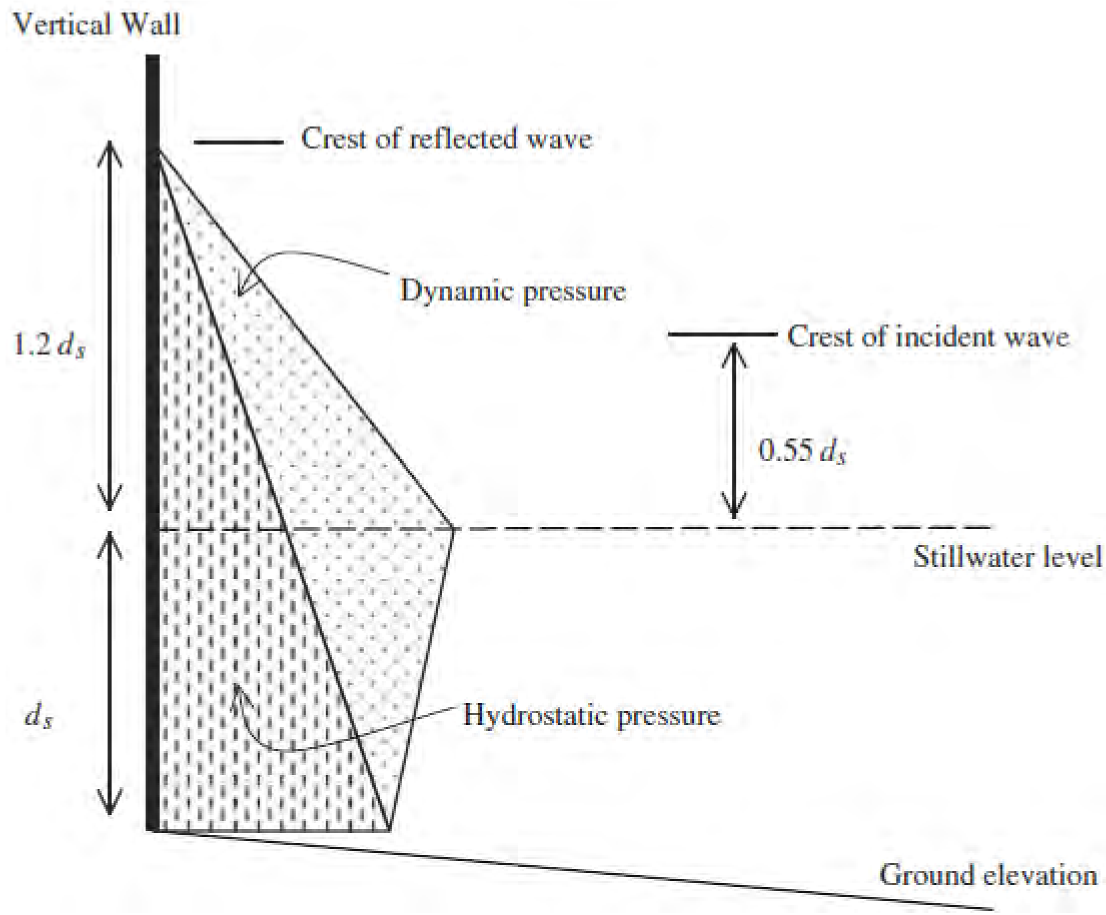


Figure 4-3: Normally Incident Wave Breaking Pressures, *ASCE (2016).*

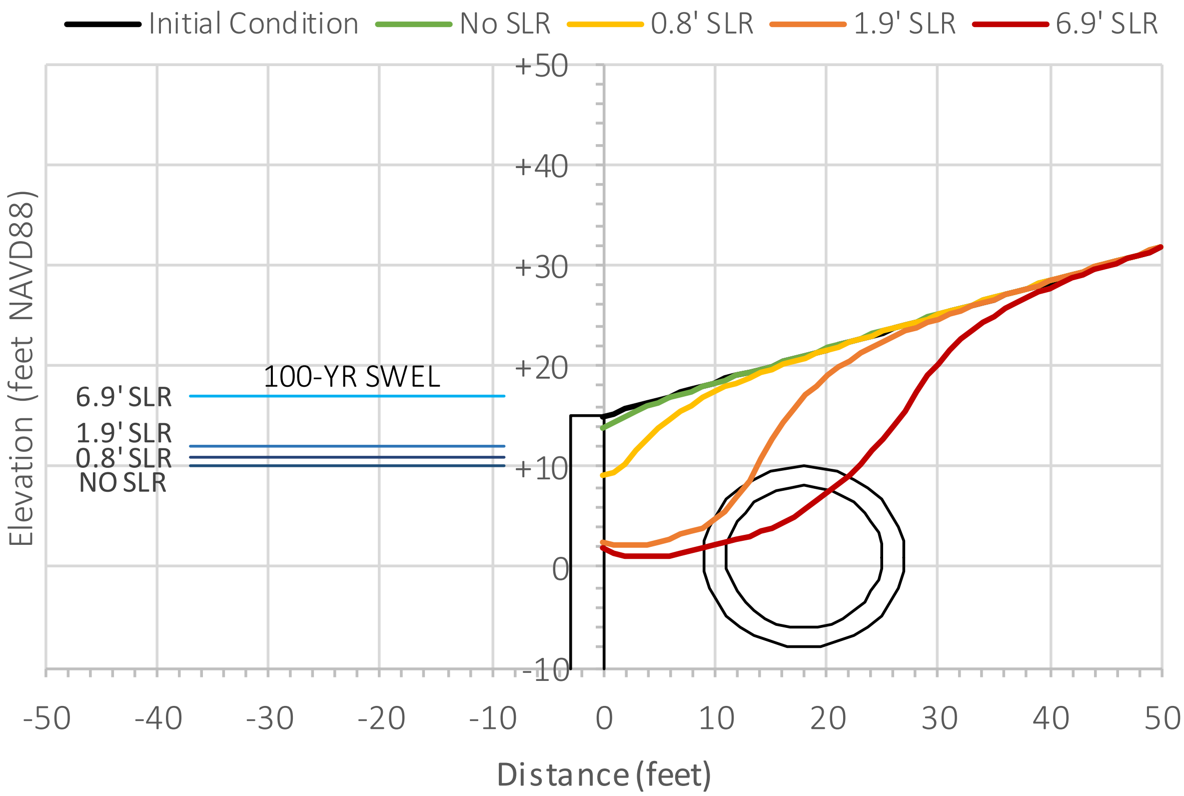


Figure 4-4: Assessment of Potential Scour of Slope above Wall Crest (No Slope Protection).

5. Geotechnical Evaluation

The Geotechnical Assessment Report is provided in Appendix B.

5.1. Introduction

5.1.1. General

A geotechnical investigation for this project is underway; results of the investigation will be presented in a Geotechnical Data Report (GDR) and a Geotechnical Interpretative Report (GIR). AGS' initial geotechnical findings and preliminary geotechnical recommendations for CER evaluations are summarized in this section. In general, the project as currently proposed is feasible from a geotechnical engineering standpoint, provided that the recommendations presented in AGS' geotechnical reports are incorporated in final design and construction.

5.1.2. Project Elements

The South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection project primarily includes the following two elements:

1. Structural protection of the LMT; and
2. Strategic management of the coastal conditions.

AGS' geotechnical investigation for this project is focused on the first element (structural protection of the LMT). The scope of AGS' geotechnical investigation on the second element (strategic management of the coastal conditions) is to characterize the ground conditions and soil properties at the beach and the bluff.

The Alternatives Analysis Report (AAR) by SFPUC (2018) has identified protection of the LMT with an exterior low-profile wall as the most feasible alternative. The low-profile wall would be constructed on the west (seaward) side of the LMT. The selected concept for the low-profile wall is a system of secant piles with tiebacks.

The secant pile wall would consist of overlapping unreinforced and reinforced drilled, cast-in-place concrete piles (called "primary" and "secondary" piles, respectively) installed at approximately 5-foot center-to-center spacing. Both the primary unreinforced and secondary reinforced piles would be approximately 3 feet in diameter. The primary unreinforced piles would be drilled first and filled with concrete, followed by the secondary reinforced piles drilled between and partially cutting into the primary unreinforced piles. The toe of the primary unreinforced piles would be set at approximately Elevation -10 feet (NAVD88). The secondary reinforced piles would be extended to greater depths as determined by structural analysis. An approximately 5-foot wide by 4-foot deep continuous grade beam would be constructed for the secant pile wall with the top set at an elevation approximately 6 feet above the crown of the LMT. It is anticipated that the tiebacks would be installed at a spacing of 10 to 15 feet along the grade beam and at an inclination of 9H:12V (approximately 53 degrees below the horizontal) to provide lateral restraint to the top of the wall.

Initially, the secant pile wall would be buried. However, over time, as beach recession occurs, the secant pile wall would be exposed (with the ground surface in front of the wall designed for a beach level of Elevation +2 feet). Ultimately, the landward side above the top of the secant pile wall would become a 3H:1V backslope except at the South Reach where the backslope gradient would gradually increase to 2H:1V. To provide resistance to wave run-up over the top of the wall, the upper 4 feet of soil cover for the ultimate backslope will be improved by in-situ soil-cement mixing.

The proposed wall alignment is divided into five reaches (each with a representative station for design) as shown below:

Table 5-1: Reach Descriptions

Name	Start STA	End STA	LMT Setback from Bluff (ft)	Depth of LMT Crown (Min/Max) (ft)	LMT Crown Elevation (Beginning / End) (NAVD88)	Representative Station
North Reach	10+75	19+50	40	20/20	9.47 / 10.31	16+00
EQR Reach	19+50	24+50	38	20/20	10.31 / 11.15	22+00
Rubble Reach	24+50	33+60	80	20/22	11.15 / 11.88	28+00
Bluff Reach	33+60	36+60	35	22/30	11.88 / 12.55	34+00
South Reach	36+60	42+75	28	30/50	12.55 / 13.33	40+00

5.1.3. Existing Data Review

Available data from previous geotechnical studies (as listed below) have been reviewed by AGS for this project:

- Geotechnical Report, Westside Pump Station Reliability Improvements, San Francisco, California, by GTC, Inc., 2016.
- Draft Report Geotechnical Study, Slope Stability Hazard Evaluation, Great Highway Stabilization, San Francisco, California, AGS. Inc., 2010.
- Preliminary Engineering Study, Lake Merced Tunnel, The Great Highway, San Francisco, California, Treadwell & Rollo, 2002.
- Lake Merced Transport Tunnel Geotechnical Design Summary Report, Parsons Brinckerhoff Quade & Douglas, Inc., 1990.
- Geotechnical Data Report, Lake Merced Transport, San Francisco, California, AGS, Inc., 1989.
- Preliminary Geotechnical Investigation, Lake Merced Transport Project, San Francisco, California, Harding-Lawson Associates, 1981.
- Geotechnical Engineering Evaluation, Southwest Ocean Outfall Project, Woodward-Clyde Consultants, 1977.
- Geologic Exploration Studies, Southwest Ocean Outfall Project, Woodward-Clyde Consultants, 1977.
- Review and Evaluation of Existing Data, Southwest Ocean Outfall Project, Woodward-Clyde Consultants, 1977.
- Preliminary Report, Offshore Geophysical Survey, Southwest Ocean Outfall Project, Woodward-Clyde Consultants, 1977.
- Onshore Seismic Refraction Survey, Southwest Ocean Outfall Project, Woodward-Clyde Consultants, 1977.
- West Side Transport Soil Investigation Phase I, Harding-Lawson Associates, 1976.

Relevant information from existing data review (including previous boring logs and locations) will be presented in AGS' geotechnical reports.

5.1.4. Field Exploration Program

AGS' field exploration program for this project was performed in February and March 2019, and consisted of:

- Seven geotechnical soil borings (B-1 through B-5, B-6A and B-6B);
- Fourteen cone penetration tests (CPT-1, CPT-2, CPT-3, SCPT-3, CPT-4 through CPT-13);
- Three monitoring wells (MW-1, MW-4 and MW-5 installed adjacent to B-1, B-4 and B-5, respectively);
- Six potholes (PH-1A, PH-1B, PH-2A, PH-3A, PH-3B and PH-4A);
- Geophysical survey subsurface profiles (ML-1A, ML-1B, and ML-2 through ML-4); and

- Six environmental borings (EB-1 through EB-6).

The results of AGS' field exploration program have been evaluated to develop geotechnical recommendations for this project. Details will be presented in AGS' geotechnical reports.

5.1.5. Geotechnical Laboratory Testing Program

Geotechnical laboratory testing was performed on selected soil samples from AGS' geotechnical soil borings. The geotechnical laboratory testing program included:

- Moisture content and density;
- Atterberg limits;
- Particle size analysis;
- Triaxial compressive strength (unconsolidated-undrained);
- Corrosivity;
- Petrographic analysis; and
- X-ray diffraction.

The results of AGS' geotechnical laboratory testing program have been evaluated to develop geotechnical recommendations for this project. Details will be presented in AGS' geotechnical reports.

5.1.6. Environmental Laboratory Testing Program

Samples collected from the six environmental borings drilled to a depth of approximately 5 feet adjacent to Borings B-1 through B-6 were sent to Enthalpy Analytical in Berkeley for the following tests:

- Total Petroleum Hydrocarbons – gasoline diesel and motor oil by EPA Method 8015B;
- California Title 22 Metals by EPA Methods 6010B and 7471A;
- Hexavalent Chromium by EPA Method 7196A;
- Volatile Organic Compounds (VOCs) by EPA Method 8260B;
- Semi-volatile Organic Compounds (SVOCs) by EPA Method 8270C; and
- Organochlorine Pesticides (OCPs) by EPA Method 8081A.

The results of AGS' environmental laboratory testing program will be presented in AGS' geotechnical reports.

5.1.7. Codes and Standards

The codes and standards applicable to AGS' geotechnical investigation for this project include the following:

- American Society of Civil Engineers Standard 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7-16);
- 2019 California Building Code (CBC); and
- San Francisco Public Utilities Commission General Seismic Requirements for Design of New Facilities and Upgrade of Existing Facilities, Revision 3, June 2014 (SFPUC GSR 2014).

5.2. Initial Geotechnical Findings

5.2.1. Site Geology

The major geologic units at the project site include the following:

Artificial Fill (Qaf)

The artificial fill consists mainly of reworked dune sand, with occasional gravel and construction debris, and is commonly underlain by dune sand. The thickest fill occurs as infill along the bluffs, and as backfill around drainage pipes and other utilities. In the near-surface, the fill consists of clayey or sandy angular gravel.

Dune Sand (Qd)

The thickness of the dune sand ranges from light cover at the tops of the highest bluffs, and up to 50 feet inland of the coast. Near-surface dune sands tend to be poorly graded, fine to medium grained clean sand, whereas sands at depth may have light cementation or laminations.

Beach Sand (Qb)

Beach sand in the project vicinity consists of loose, well-sorted quartz and feldspar sand, which grades fine to coarse depending on its location in the surf zone.

Colma Formation (Qc)

The Colma Formation generally consists of oxidized, reddish brown, predominantly medium-grained quartz-feldspar arkosic sand with heavy mineral laminations, and bedding ranging from horizontal up to dipping 5 degrees east. Facies of the Colma Formation at depth may include fine-grained micaceous silty sand, silt, thin clay lenses, and lenses of rounded fine gravels consisting of red chert, green chert, Monterey formation laminated rock, and blue schist.

Merced Formation (Qm)

The Merced Formation consists of an accumulation of poorly consolidated sand, clay, gravel and silt sediments, which were deposited almost continuously in the late to early Pleistocene. Based on the tectonic history of the Serra Fault, the Merced Formation can show bedding ranging from near-horizontal in the project vicinity, to up to 25 degrees and striking northeast in the vicinity of Fort Funston and Mussel Rock.

5.2.2. Faulting and Seismicity

The site is not located within an Alquist-Priolo earthquake fault zone (CGS, 2007). Therefore, the risk from surface fault rupture is considered to be very low.

The project area is located in a seismically active region subject to periodic earthquakes causing strong to violent ground shaking of the site. The San Andreas Fault is about 1½ miles southwest of the site and is the major fault system in the region. Further from the project site are the San Gregorio Fault, which is about 5 miles southwest of the site, the Hayward Fault, which is about 17 miles to the northeast; both are also significant seismic sources. Other major active faults considered capable of causing significant shaking at the project site include the Point Reyes, Monte Vista-Shannon, Mount Diablo Thrust, Calaveras, Green Valley, West Napa, Greenville and Great Valley faults.

5.2.3. Groundwater

Groundwater levels recorded in previous borings and monitoring wells generally range from approximately Elevation +5.5 to +13.5 feet. In addition, the published groundwater level monitoring data from the SFPUC Annual Groundwater Monitoring Reports for the Westside Basin were reviewed. Based on that, groundwater levels at Elevation +16 feet (for the North, EQR and Rubble Reaches), Elevation +18 feet (for the Bluff Reach) and Elevation +19 feet (for the South Reach) are recommended for preliminary conceptual design purposes.

5.3. Preliminary Geotechnical Recommendations

5.3.1. Seismic Design Criteria

Based on the methods of SFPUC General Seismic Requirements (SFPUC 2014 GSR), site specific spectral accelerations were developed for the project. According to Section 2.2.3 of the SFPUC GSR, design ground motions for structures in Seismic Performance Class III should be based on a 5 percent probability of exceedance in 50 years (975-year return period). The design ground motion need not exceed a deterministic limit, taken as the 84th percentile level for the maximum earthquake, and not be lower than the deterministic Maximum Considered Earthquake (MCE) as defined in Section 21.2.2 of ASCE 7. For preliminary conceptual design of the proposed secant pile wall, the recommended acceleration response spectrum corresponding to 5 percent structural damping ratio is as follows:

Table 5-2: Recommended Acceleration Response Spectrum

Structural Period	Probabilistic MCE_R	Deterministic MCE_R	Deterministic Lower Limit	Design Response Spectrum
(sec)	(g)	(g)	(g)	(g)
0.01	0.73	0.87	0.65	0.73
0.02	0.73	0.87	0.69	0.73
0.03	0.73	0.86	0.74	0.74
0.05	0.80	0.91	0.83	0.83
0.08	0.97	1.04	0.96	0.97
0.10	1.12	1.18	1.05	1.12
0.15	1.33	1.38	1.28	1.33
0.20	1.48	1.56	1.50	1.50
0.25	1.61	1.76	1.50	1.61
0.30	1.73	1.95	1.50	1.73
0.40	1.83	2.21	1.50	1.83
0.50	1.81	2.27	1.50	1.81
0.75	1.53	2.06	1.50	1.53
1.00	1.27	1.81	1.50	1.50
1.50	0.92	1.40	1.00	1.00
2.00	0.70	1.08	0.75	0.75
3.00	0.46	0.71	0.50	0.50
4.00	0.32	0.49	0.38	0.38
5.00	0.24	0.36	0.30	0.30
7.50	0.12	0.18	0.20	0.20
10.00	0.07	0.11	0.15	0.15

5.3.2. Liquefaction

Soil liquefaction is a phenomenon in which saturated, loose to medium dense cohesionless soils lose their strength during a major earthquake. Soils most susceptible to liquefaction are loose, clean sands. Silty sands and low-plasticity silts may also liquefy during strong ground shaking.

The liquefaction analysis was conducted according to the method set forth in Idriss and Boulanger (2014) using the following parameters:

- Magnitude 8.05 earthquake;
- PGA_M 1.02g; and

Groundwater at Elevation +16 feet (for the North, EQR and Rubble Reaches), Elevation +18 feet (for the Bluff Reach) and Elevation +19 feet (for the South Reach)

The analysis results generally indicate that there is a layer of potentially liquefiable soils in the upper zone (primarily consisting of loose to medium dense fill and dune sand below the groundwater table) that is approximately 5 to 7 feet thick and located at depths between approximately 15 and 25 feet below the existing ground surface. Below that, the sands within the Colma and Merced Formations are mostly dense to very dense and, in general, their potential for liquefaction is low. Some relatively thin intermittent layers of medium dense sands were encountered within the Colma and Merced Formations that may be locally liquefy during a major earthquake. However, considering that they are generally localized, relatively thin and at greater depths, their potential impact on the project is considered to be low.

5.3.3. Tsunami

The Tsunami Inundation Map for Emergency Planning (San Francisco North Quadrangle, June 2009, State of California) indicates that the project site is within an area at risk for tsunami inundation. The tsunami inundation line extends from the shoreline up to and including the Great Highway between Station 12+00 and Station 22+00. Between Station 22+00 and Station 33+00, the tsunami inundation line extends to the immediate west of the southbound lane of the Great Highway.

5.3.4. Secant Pile Wall

The upper 4 feet of soil cover for the ultimate backslope (slope stabilization layer) will be constructed by either in-situ soil-cement mixing or controlled low strength material to provide resistance to wave run-up over the top of the wall. Adequate drainage should be provided behind the grade beam such as installation of a subdrain system discharging to a suitable free-drainage outlet. The discharge system should be designed properly to avoid causing any slope instability.

Tiebacks would be installed at the grade beam, extending back into the landward side beneath the LMT with a minimum clearance of 5 feet. The geotechnical recommendations for tiebacks are presented in the “Tiebacks” section.

The drilled piles for the secant pile wall should be designed such that the vertical, horizontal or rotational loads are within the design and operational limits. In addition to the weight of the wall, grade beam and backfill placed above, the vertical loads on the drilled piles should also include the downdrag load from the tiebacks. On a preliminary basis, for vertical compression (downward) loads, the drilled piles should be designed for an allowable downward skin friction of 500 pounds per square foot (psf) in dense soils for dead plus live loads. This value includes a factor of safety of 2 and may be increased by 1/3 to include wind and seismic loads. Uplift resistance may be calculated to be 75 percent of the skin friction in compression. The drilled piles should extend to a depth below the potentially liquefiable zones with zero skin friction in the liquefiable soils and account for liquefaction-induced downdrag force of 20 tons.

The secant pile wall would be designed to resist lateral earth pressures based on the ultimate retaining condition as described in the “Project Description” section (when the bluff in front of the wall has resulted loss of soils to a beach level of Elevation +2 feet). Preliminary geotechnical recommendations on lateral earth pressures are presented in the “Lateral Earth Pressures” section.

Based on a review of the existing data and the subsurface conditions encountered in AGS’ field exploration for this geotechnical investigation, caving and seepage in sandy soils should be expected during drilling of the pile holes. Casing (preferably rotated down with the drilling equipment) or use of slurry displacement method would be required to maintain an open pile hole for installation of reinforcing steel and placement of concrete. Concrete would be required to be placed by tremie method to displace the water out of the pile holes.

It is important to confirm that the drilled piles installed are structurally sound and do not contain significant defects. Therefore, post-construction integrity testing (such as crosshole sonic logging or gamma-gamma) should be performed to evaluate the quality of the completed drilled piles. In general, sonic logging is most suited for integrity evaluation within steel cage and consists of vertical access tubes (steel or PVC pipe) installed in the drilled piles before placing the concrete. Once the drilled piles are completed, a compression wave source is lowered down one tube and a receiver down another while taking readings of the wave propagation through the drilled piles. Voids, if present, will show up as anomalies in the wave propagation pattern. Similarly, gamma-gamma testing ensures sufficient concrete cover over steel cage. The testing utilizes an electric winch to pull a 4-foot probe with the radioactive source at the end, up through PVC pipes installed in the concrete. As the probe moves up through the tubes, it reads average concrete densities at set intervals. These intervals are then plotted and analyzed for average bulk density versus pile depth. Deviation in average bulk density are used to identify pile anomalies or defects and to assess pile/concrete quality.

5.3.5. Lateral Earth Pressures

Lateral earth pressures on the secant pile wall with tiebacks are based on apparent earth pressure diagrams (trapezoidal pressure distribution) using the methods recommended in American Association of State Highways and Transportation Officials (AASHTO) Design Specifications (2012), California Department of Transportation (Caltrans) Memo To Designers (MTD) 5-12 (2012) and Federal Highway Administration (FHWA) Geotechnical Engineering Circular (GEC) No.4 (1999) for design of anchored walls. For conceptual design, preliminary lateral earth pressures were developed using the soil properties presented below.

Table 5-3: Soil Properties for Lateral Earth Pressures

Reach (STA)	Design Groundwater Elevation (Feet)	Layer	Top of Layer Elevation (NAVD88) (Feet)	Total Unit Weight (pcf)	Friction Angle (degree)	Cohesion (psf)
North (16+00)	+16	Fill	+31	120	33	0
		Dune Sand	+16	120	34	0
		Colma Formation	+8	125	36	0
		Merced Formation	-47	125	27	300
EQR (22+00)	+16	Fill	+30	120	33	0
		Dune Sand	+12	120	34	0
		Colma Formation	+8	125	36	0
		Merced Formation	-36	125	27	300
Rubble (28+00)	+16	Fill	+29	120	33	0
		Dune Sand	+18	120	34	0
		Colma Formation	+11	125	36	0
		Merced Formation	-40	125	27	300
Bluff (34+00)	+18	Fill	+35	120	33	0
		Dune Sand	+20	120	34	0
		Colma Formation	+15	125	36	0
		Merced Formation	-33	125	27	300
South (40+00)	+19	Fill	+45	120	33	0
		Dune Sand	+15	120	34	0
		Colma Formation	+10	125	36	0
		Merced Formation	-33	125	27	300

In additional to the lateral earth pressure and hydrostatic water pressure for static condition, seismic lateral earth pressure should also be included in the design of the secant pile wall for seismic condition. The additional seismic lateral earth pressure increment can be obtained by the Mononobe-Okabe method. According to Section 7 of the 2014 SFPUC GSR, hydrodynamic water pressure should also be considered using the method recommended in Ebeling et al. “The Seismic Design of Waterfront Retaining Structures” (1992).

As discussed in the “Liquefaction” section, if the soils behind the secant pile wall liquefy during a major earthquake, the lateral earth pressure exerted on the wall would be momentarily increased due to liquefaction-induced excess pore water pressure. For those soils that will be subjected to liquefaction behind the wall, the liquefaction-induced lateral earth pressure can be calculated using an equivalent fluid pressure of 120 pcf. The liquefaction-induced lateral earth pressure and the seismic lateral earth pressure discussed above are two different scenarios that will not occur simultaneously. The secant pile wall design should be checked against both to see which scenario is more critical.

If vertical surcharge loads are anticipated within the zone above an imaginary 45-degree line projected up from the long-term exposed bottom of secant pile wall (Elevation +2 feet), the additional lateral earth pressures from the surcharge should be included in the secant pile wall design.

5.3.6. Tiebacks

Design Criteria

Due to the long-term exposed height of the secant pile wall ranging from approximately 16 to 19 feet with backslope ranging from 3H:1V to 2H:1V, tiebacks would be installed to provide the necessary lateral support. The subsurface conditions on site generally consisting of sandy soils below groundwater would be susceptible to caving. The drilling method to install tiebacks at various locations should consider the potential for caving. Where caving is anticipated to occur, drilling fluids or casing should be used to stabilize the drill hole.

Based on the current concept plans, the tiebacks are being proposed to be installed at an inclination of 9H:12V (approximately 53 degrees below the horizontal). It is understood that this relatively steep angle of installation is to meet the required clearance with the LMT and to keep the construction work within the project limits.

Tiebacks are typically installed at inclination between 15 and 30 degrees below the horizontal and inclination up to 45 degrees below the horizontal can generally be installed by most contractors. If possible, consideration should be given to moving the secant pile wall further seaward (perhaps by approximately 5 feet). This would allow easier installation of tiebacks at the more common 45 degrees (or less) to attract more qualified contractors and to increase tieback efficiencies (with larger horizontal component of tieback load).

For preliminary design purposes, an allowable soil/grout bond strength of 2,000 psf (beyond the active zone defined by a plane sloping up at 60 degrees with the horizontal and from a point H/5 away from the bottom of the wall, at Elevation +2 feet, where H is the long-term exposed height of the wall) may be considered. This preliminary allowable soil/grout bond strength includes a factor of safety of 2. It should be noted that the bond strength of tiebacks will depend on the construction method used by the contractors. The project specifications should allow for modification of the bond strength based on values that are demonstrated from field verification testing.

The tiebacks should be designed for a marine environment anticipated in the long-term condition. Double corrosion protection would be required with factory pre-grouted encapsulation of the bar within a corrugated plastic sheath. Also, the tieback system should be re-stressable, if needed, when the top of the secant pile wall is exposed in the future.

Testing and Acceptance Criteria

It is recommended that at least two sacrificial tiebacks (at each reach) be selected for verification testing to verify the bond strength used in the design. All production tiebacks should be proof-tested to at least 1.5 times the design load. Detailed recommendations on verification and proof testing procedures would be provided in AGS' geotechnical reports. The verification and proof testing should be performed under the observation of the project geotechnical engineer.

Tieback-induced Downdrag Force

As noted above, in addition to the weight of the wall, grade beam and backfill placed above, the vertical loads on the drilled piles should also include the downdrag force from the tiebacks. The downdrag force from the tiebacks is essentially the vertical component of the tieback load. Therefore, by increasing the inclination of the tiebacks, the vertical component of the tieback load also increases, thus increasing the vertical load on the secant pile wall and the underlying foundation material. The downdrag force on the secant pile wall from tiebacks can be estimated from the equation: $F \times \sin \alpha$, where F is the design load in the tieback and α is the inclination of the tieback below horizontal.

5.3.7. Controlled Low Strength Material (CLSM)

The use of CLSM may be considered to improve the upper 4 feet of soil cover for the ultimate backslope. The requirements of CLSM should include:

1. The in-situ density should be no more than 130 pcf;
2. If the CLSM needs to be easily excavatable in the future, the 28-day unconfined compressive strength should be no less than 50 pounds per square inch (psi) and not more than 150 psi;
3. If the CLSM does not need to be easily excavatable in the future, the 28-day unconfined compressive strength should also be no less than 50 psi but can be higher than 150 psi;
4. The physiochemical properties should not be harmful to the LMT; and
5. The slump should be less than 12 inches but not less than 6 inches.

5.3.8. Earthwork

Site Preparation

The work limits should be properly marked and traffic controlled in accordance with City and County of San Francisco requirements, and then cleared of any obstructions, including pavements and any debris hindering work. Vegetation and landscaping (if any) in the construction areas should be stripped and disposed of outside the construction limits. Safety fencing should be installed in accordance with OSHA, and all other applicable requirements, including warning fencing placed near the edge of deep open excavations and silt fencing or other environmental protective fencing required by environmental compliance manager. Affected structures, equipment, and debris should be abandoned, disassembled, or demolished and disposed of outside the construction limits. Based on review of the LMT as-built plans, there is an existing Army Bunker with invert at approximately Elevation +23½ feet near the south end of the secant pile wall (approximately Station 42+00). It is anticipated that the secant pile wall would have to either locate away from the existing Army Bunker or bridge over it. Likewise, the secant pile wall would also have to be designed to bridge over the existing 12-foot by 12-foot SWOO structure at approximately Station 36+50.

Existing underground utilities located within the project site, if affected by construction activities, should be relocated or protective measures taken prior to construction. All debris generated from the demolition of underground utilities, including abandoned pipes, should be removed from the site as construction proceeds.

During excavation, any observed soft or loose zones should be compacted in-place or excavated and replaced with properly compacted backfill. Upon completion of excavation, backfill may be placed in accordance with the recommendations presented below.

Excavation Characteristics

The Contractor should review the available data, in order to independently evaluate the type of equipment required to complete the proposed excavations to the required depths. Based on review of the existing data and the subsurface conditions encountered in the field exploration for this study, it appears that conventional earth moving equipment may be used to remove most of the on-site soils. Existing underground utilities or other structures may require jackhammering or hoe-ram to remove.

Unshored Excavations

During construction, the contractor must maintain safe and stable slopes and provide shoring as necessary. All cuts deeper than 4 feet must be sloped or shored in accordance with the current requirements of OSHA and Cal-OSHA. Shallow excavations above the groundwater level may be sloped if space permits. Soils at the site appear to generally be OSHA Class C soils, and may be sloped no steeper than 1.5H:1V. Sloping of excavations should conform to OSHA requirements, and should be monitored by the contractor to verify stability to ensure worker safety.

Heavy construction equipment, building materials, and excavated soils should be kept away from the edge of the excavation at least a distance equal to, or greater than, the depth of the excavation.

During wet weather, runoff water should be prevented from entering excavations, and collected and disposed of outside the construction limits. To prevent runoff from entering the excavation, a perimeter berm may be constructed at the top of the slope. In addition, it is recommended that the sidewalls of the excavation be covered by plastic sheets to prevent saturation of the earth material.

Fills and Backfills

Fills and backfills may be placed under and around the grade beam of the secant pile wall, utility trenches, and pavement during construction of this project.

Fills and backfills may either be structural or nonstructural. Structural fills and backfills are those defined as providing support to foundations, and pavements. Nonstructural fills and backfills include all other fills such as those placed for landscaping, and not planned for future structural loads. Structural fills and backfills should be compacted to at least 95 percent relative compaction (as determined by ASTM D1557-12); nonstructural fills and backfills should be compacted to at least 90 percent relative compaction.

Due to the concern of potential damage that may be caused by compaction of fill and backfill to the existing LMT, the use of heavy compaction equipment directly above the LMT should be avoided. In those areas, the addition of a layer of geotextile (such as Mirafi 600x or approved equivalent) placed underneath the CLSM (if used as the upper 4 feet soil cover for the ultimate 3H:1V backslope) could be considered.

All structural fills and backfills should be granular fills with no pieces larger than 3 inches in any dimension, no more than 20 percent passing the No. 200 sieve, a Liquid Limit of 35 or less, a Plasticity Index of 12 or less, and should be placed in 8-inch lifts, moisture-conditioned to near-optimum moisture, and compacted to 95 percent relative compaction (as determined by ASTM D1557-12). Non-structural fills should meet the same requirements, but should be compacted to at least 90 percent relative compaction.

Samples of imported fill and backfill materials should be submitted to the project geotechnical engineer prior to use for testing to establish that they meet the above criteria.

The existing on-site soils are generally suitable from a geotechnical perspective for use as engineered fill, provided they are free of debris, hazardous materials and other deleterious matter.

The fill and backfill materials should be placed and compacted under the full time observation and testing of the project geotechnical engineer.

5.3.9. Dewatering and Groundwater Considerations During Construction

Groundwater levels at the site will fluctuate due to rain and other factors. As discussed above, groundwater levels at Elevation +16 feet (for the North, EQR and Rubble Reaches), Elevation +18 feet (for the Bluff Reach) and Elevation +19 feet (for the South Reach) are recommended for preliminary conceptual design purposes. Therefore, excavations for construction of the grade beam and installation of tiebacks for the secant pile wall may extend below the groundwater level.

The contractor should make an independent evaluation of the groundwater levels at the site, and be responsible for providing an adequate dewatering system during construction. During excavation for construction, it is recommended that the water level be maintained at least two feet below the bottom of the excavation until construction is complete, and until the weight of the constructed structure (or installed utilities) is sufficient to resist buoyancy. Selection of the equipment and methods of dewatering should be left up to the contractor, and the contractor should be aware that modifications to the dewatering system may be required during construction, depending on conditions encountered.

The hydraulic conductivities of the subsurface materials vary in response to the heterogeneous, anisotropic media. Within the proposed excavation depth for construction of the secant pile wall (including construction of grade beam and installation of tiebacks), granular deposits were generally encountered. Granular deposits encountered in AGS' borings generally consist of poorly graded sand with silt, silty sand, and clayey sand with hydraulic conductivities probably in the range of $1x10^{-1}$ to $1x10^{-3}$ cm/s.

5.3.10. Flexible Pavement

For the SFPUC access road, any new asphalt concrete pavement should be designed based on the Caltrans Flexible Pavement Design Method with an assumed R-Value of 15 and Traffic Index (TI) as determined by the project civil engineer.

The uppermost 12 inches of all pavement subgrade soils should be moisture conditioned to near optimum moisture content and compacted to at least 95 percent relative compaction (as determined by ASTM D1557-12) to provide a smooth, unyielding surface. All fill and backfill materials should be placed in lifts not exceeding approximately 8 inches in loose thickness. If zones of soft or saturated soils deeper than 12 inches are encountered during excavation and compaction, deeper excavations may be required to expose firm soils. This should be determined in the field by the project geotechnical engineer.

Class 2 aggregate base should be placed in thin lifts in a manner to prevent segregation; uniformly moisture conditioned; and compacted to at least 95 percent relative compaction to provide a smooth, unyielding surface.

The performance of pavements will be dependent upon a number of factors, including subgrade conditions at the time of paving, runoff, and loading. Runoff should not be allowed to seep below pavements from adjacent areas. Proper drainage below the pavement section helps prevent softening of the subgrade and has a significant impact on pavement performance and pavement life. Periodic maintenance should be performed throughout the life of the proposed pavements including periodic seal coats and crack maintenance/sealing.

Should import material be used to establish the proper grading for the new pavement, the import material should be approved by the project geotechnical engineer before it is brought to the site. The select import material should meet the following requirements:

- Have an R-value of not less than 30;
- Have a Plasticity Index not higher than 10;
- Not more than 15 percent passing the No. 200 sieve;
- No rocks larger than 3 inches in maximum size;
- Have a pH of 6.5 to 7.5;
- Have a minimum resistivity of 5000 ohms/cm; and
- Have a maximum soluble sulfate content of 0.2 percent by weight.

5.3.11. Corrosion Potential

Based on the soil resistivity classification presented by National Association of Corrosion Engineers (2010) and the results of corrosivity testing at the site, the onsite soils are classified as “extremely corrosive” to “moderately corrosive”. According to ACI 318-11, the sulfate concentration measured in one of the corrosivity samples tested for AGS’ geotechnical investigation indicates a Soil Exposure Class S1.

Corrosive soils may adversely affect the foundations and buried utilities. It is recommended that all buried metal piping and reinforced concrete be properly protected against corrosion depending upon the critical nature of the structure. A corrosion engineer should be consulted for the development of long-term site-specific corrosion protection measures.

6. Civil

6.1. Background

The project site extends from the north at the intersection of Sloat Blvd. and the Great Highway to the south at the intersection of the Great Highway and Skyline Blvd. The low-profile wall extends from the intersection of Sloat Blvd. and the Great Highway to near the existing Wastewater Treatment Plant and will be approximately 3,200 feet long. Refer to Figures 6-1 to 6-4 showing the topographic plan of the project site and the alignment of the Lake Merced Transport Tunnel (LMT).

The LMT and The Great Highway are both in a north-south alignment. The Great Highway is a four-lane road with two lanes in each direction near the intersection with Sloat Blvd. The 14-foot diameter LMT constructed in the early 1990's is aligned under the southbound lanes of the Great Highway for most of its length. The tunnel crown is approximately 20 feet below existing grade near Sloat Blvd, gradually becoming as much as 50 feet below existing grade at the south end of the project as the grade of the Great Highway increases. The LMT extends further along The Great Highway to Skyline Blvd. The LMT turns inland over this section, and does therefore not need to be protected by a low-profile wall.

On the west side (beach side) of the Great Highway near Sloat Blvd, there is a rest room building and paved parking lot. The beach has been eroding and several countermeasures have been taken to prevent further erosion. Rock and large sandbags have been placed along the bluff at several locations.

The existing Westside Pump Station is located east of the Great Highway just south of Sloat Blvd. The existing treatment plant is also located east of the Great Highway near the south end of the project.

There are two abandoned pedestrian tunnels that crosses above the existing LMT Tunnel. Both abandoned tunnels are 10 ft tall x 8 ft wide. One tunnel is located approximately 250 ft south of Sloat Blvd, and the other is located approximately 1,300 ft south of Sloat Blvd. The top of these abandoned tunnels is approximately 5 ft below existing grade.

At approximately 600 feet from the south end of the project, the South West Ocean Outfall (SWOO) crosses under the Great Highway and the LMT. The SWOO is a 12 ft square reinforced-concrete box. The box connects to a 12-ft diameter reinforced-concrete pipe that discharge the treated wastewater into the ocean.

The Westside Pump Station delivers the Oceanside Water Pollution Control Plant's influent via a 48-inch diameter sewer force main pipe; the pumping station's wet weather overflow connects directly to the South West Ocean Outfall via an 84-inch diameter reinforced-concrete pipe (RCP). These two pipes are approximately 40-ft east of the LMT and buried approximately 10 to 15 feet below grade.

There is a separate Zoo Pump Station near the Sloat entrance to the Zoo that is a separate structure, and should not be confused with the Westside Pump Station.

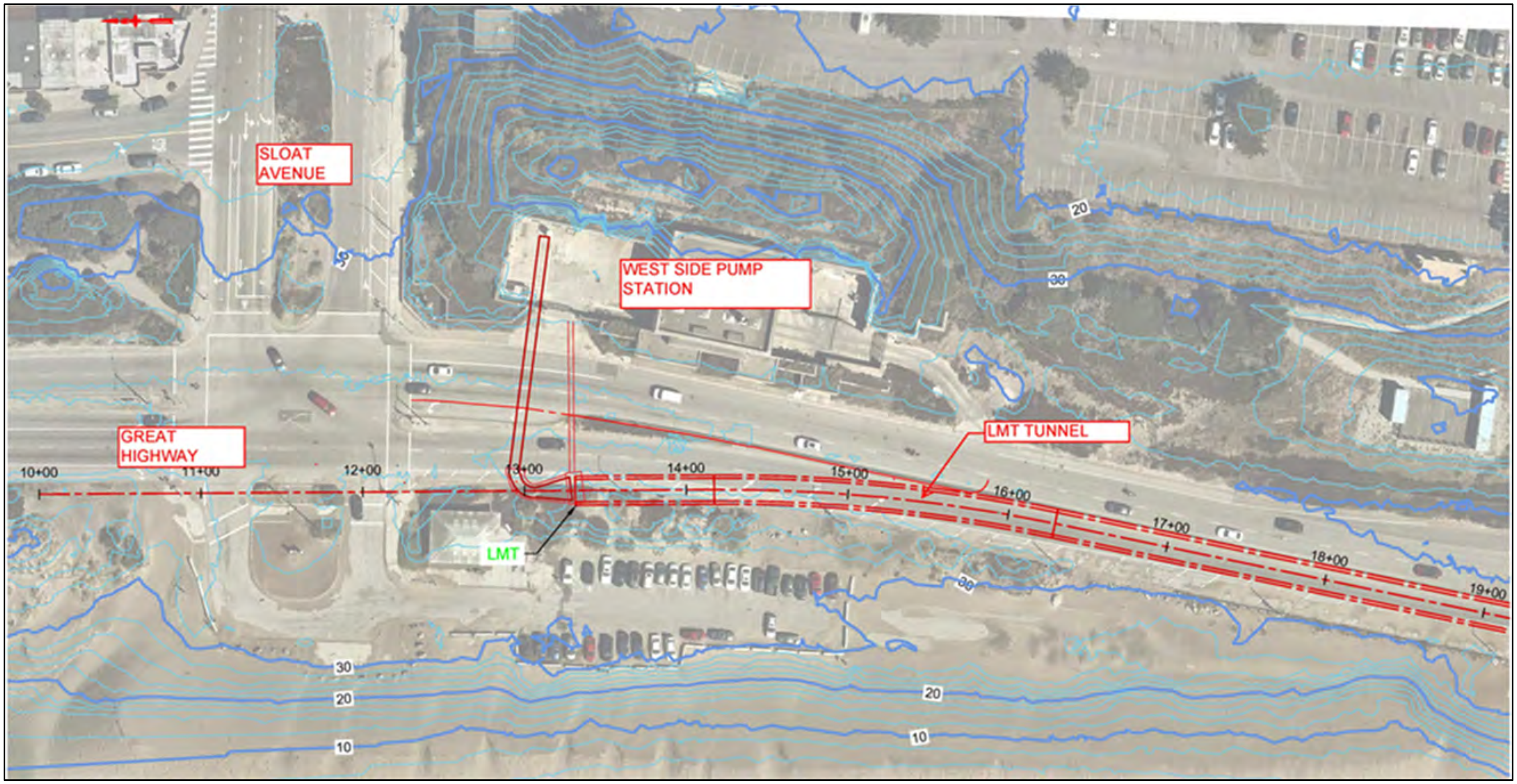


Figure 6-1: Project Site Plan, 1 of 4.

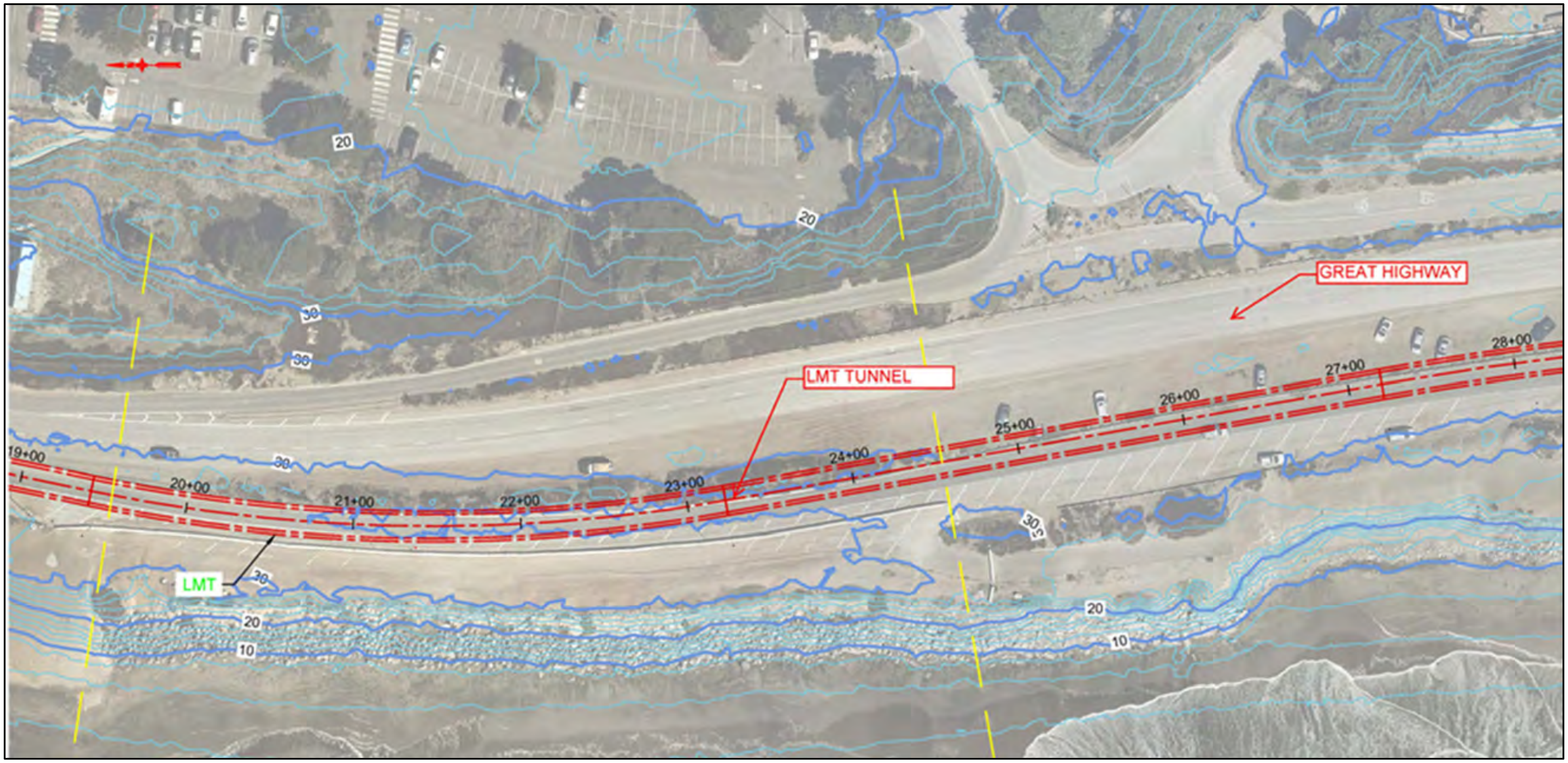


Figure 6-2: Project Site Plan, 2 of 4.

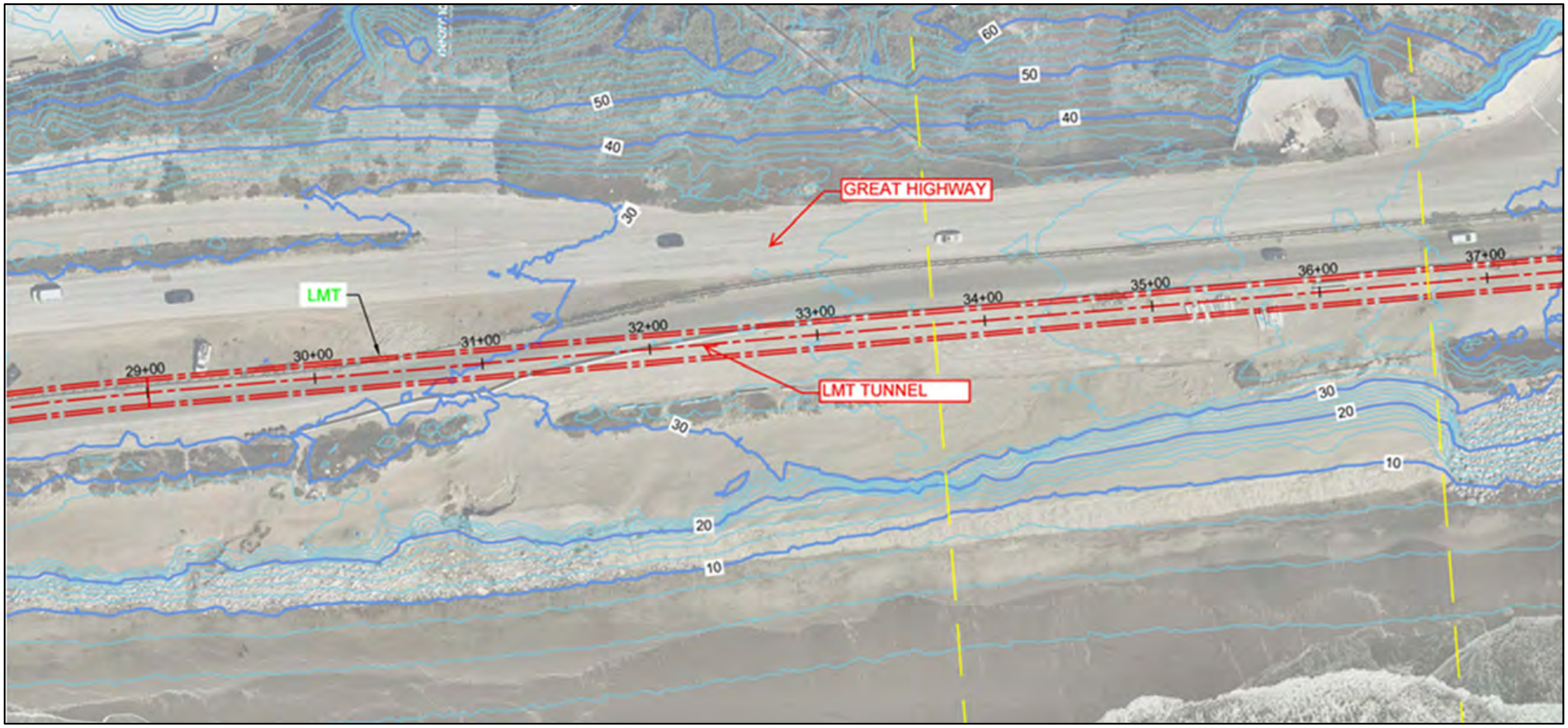


Figure 6-3: Project Site Plan, 3 of 4.

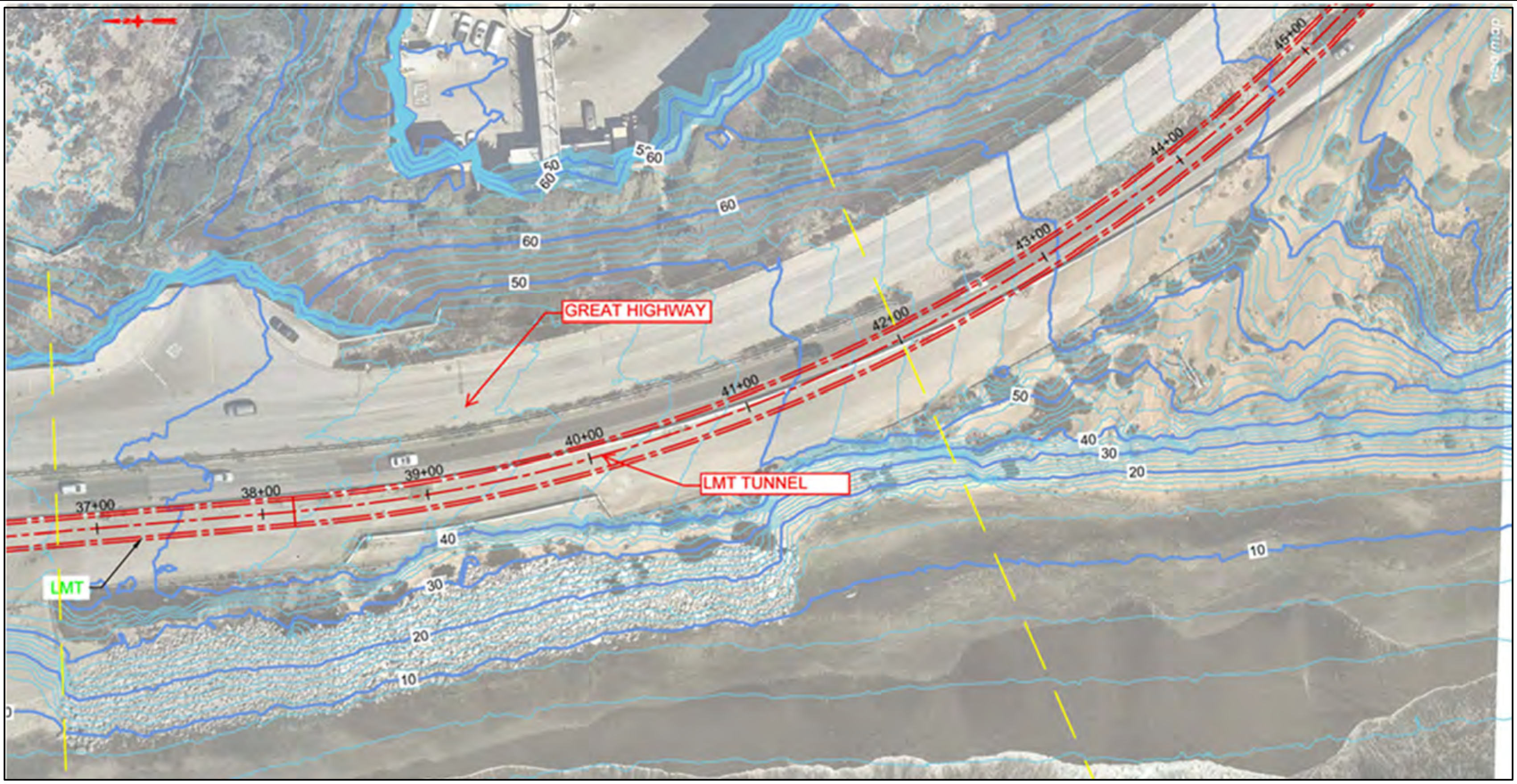


Figure 6-4: Project site Plan, 4 of 4.

6.2. Vertical Datum Reference

San Francisco City Datum is 11.326 feet above the North American Vertical Datum 1988 (NAVD 88), and 8.616 feet above the National Geodetic Vertical Datum 1929 (NGVD 29).

Table 6-1 relates the various datum planes as referenced to San Francisco City Datum (SFCD), North American Vertical Datum of 1988 (NAVD88), and Mean Lower Low Water (MLLW) Tidal Datum.

Table 6-1: Relationship between Vertical Datums.

Elevation (feet)			Datum	Remarks
SFCD	**NAVD88**	**MLLW**		
0.00	+11.33	+11.25	SFCD	San Francisco City Datum
-5.41	+5.92	+5.84	MHHW	Mean Higher High Water
-6.02	+5.31	+5.23	MHW	Mean High Water
-6.70	+4.63	+4.55	OHWM	Ordinary High Water Mark
-8.13	+3.20	+3.12	MSL	Mean Sea Level
-8.62	+2.71	+2.63	NGVD29	National Geodetic Vertical Datum of 1929
-10.11	+1.22	+1.14	MLW	Mean Low Water
-11.25	+0.08	0.00	MLLW	Mean Lower Low Water
-11.33	0.00	-0.08	NAVD88	North American Vertical Datum of 1988

The elevations in this report are all based on NAVD88 unless noted otherwise.

6.3. LMT Tunnel Alignment

The horizontal alignment of the LMT is shown on Figures 6-1 to 6-4. It runs approximately along the southbound lanes of the Great Highway. The alignment was generated using the information outlined on the as-built drawings dated 1993. Drawing number SW-12 of the SFPUC's Lake Merced Transport Tunnel as-built drawings include the survey data tabulating the LMT point of intersection (PI) Coordinates. It was determined that these coordinates were referenced to NAD27 horizontal datum. To convert the coordinates into the project's horizontal datum, NAD83, Meridian Surveying Engineering Inc. used 'Trimble Business Center', a geodetic surveying program, and checked the results using the National Oceanic and Atmospheric Administration's (NOAA) NGS Coordinate Conversion and Total Tool (NCAT) computer program. The alignment was then drawn using the converted PI points and the horizontal curve data shown on the drawings SW-1 to SW-12 of the 1993 as-built drawings. This alignment of the LMT was then overlaid on the geo-referenced 'Nearmap' aerials to show its location referenced to the existing roadway and bluff.

The LMT has a gradual slope of +0.00132 starting from the north end of the project and this slope remains constant up to the south end of the project - based on the as-built drawings. At the north end of the project (near Sloat Blvd), the invert elevation of the LMT is approximately at Elevation -6.52 ft NAVD88. The existing grade at this location is approximately +31.00 ft. NAVD88. At the south end of the project, the LMT invert elevation is approximately Elevation -2.30 ft NAVD88 and the existing grade is approximately at Elevation +48.00 ft NAVD88.

At the SWOO crossing which is at a skew angle of approximately 20 degrees with the LMT alignment, the invert of the LMT is approximately 3.64 ft above the top of the SWOO (based on as-built drawings of the LMT).

6.4. Proposed Wall Alignment

The alignment of the proposed wall is dictated by the following requirements and constraints:

- The top of wall shall be 6 ft above the crown of the LMT throughout the length of the LMT within the project. The crown is the elevation at the exterior top of the LMT – considering a structural wall thickness of 12-inches plus 9-inch pre-cast concrete wall segments for the LMT.
- The wall shall be located on the beach side of the LMT and provide adequate distance from the LMT for structural separation.
- The wall shall be located to provide adequate distance so that the sloped surface from the top of wall to the top of slope on the landside allows for a 30- feet wide Right-of-Way (ROW) Reservation. The ROW Reservation will be for a wastewater infrastructure access road and a Coastal Trail.

With the requirement and constraints, the selected alignment is shown in Figures 6-5 to 6-8. The wall is also divided into five reaches based on similarity of existing conditions within each reach. The Typical Section for each of the reaches are shown in Figures 6-9 to 6-13.

The alignment of the wall in the North Reach has a centerline located between 27 ft and 48 ft from the centerline of the LMT Tunnel. This alignment satisfies the requirements listed and mitigates the constraints. The upslope above the top of wall is at 3H:1V. Refer to Figure 6-9 for the Typical Section of the wall at this Reach.

For the Emergency Quarry Rock (EQR) Reach, Rubble Reach, and the Bluff Reach, the centerline of the wall is parallel to and 27 ft from the centerline of the LMT Tunnel. This horizontal distance between the centerlines satisfies the requirement listed and mitigates the constraints. The landside upslope above the top of wall is at 3H:1V. Refer to Figures 6-10 to 6-12 for the Typical Section for these Reaches.

For the South Reach, the wall will be on a horizontal curve with the centerline of the wall parallel to and 27 ft from the centerline of the LMT Tunnel. This horizontal distance between the centerlines satisfies the requirements listed. The upslope above the top of the wall shall vary from a 3H:1V slope to a 2H:1V slope to satisfy the 30 ft wide ROW Reservation at the top of slope. Refer to Figure 6-13. The wall ends at this reach and terminates where it is determined not necessary to provide protection for the LMT since the tunnel turns further away from the beach.

For the wall terminations in the North and South Reaches, returns will be provided to protect the wall from out-flanking in the event of catastrophic erosion of the Beach/Bluff beyond the wall limits. The configuration of the returns, and the contours of Bluff transition to the low profile wall final grading will be addressed during final design. The northern termination/transition will incorporate a ramp to access the beach from the Great Highway grade intended for use by vehicles and equipment to facilitate required maintenance, or other public safety purpose.

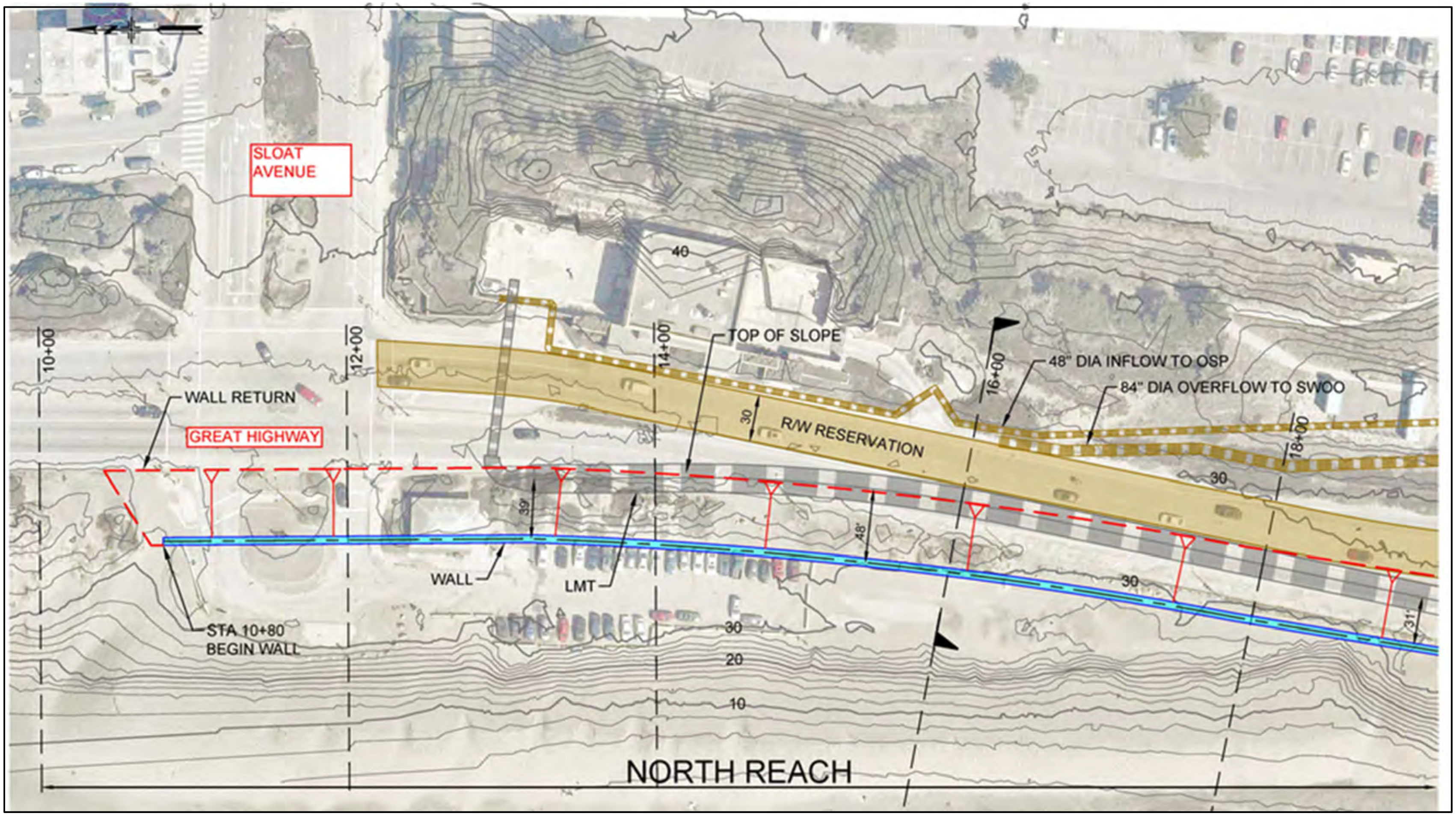


Figure 6-5: Wall Alignment Plan, 1 of 4.

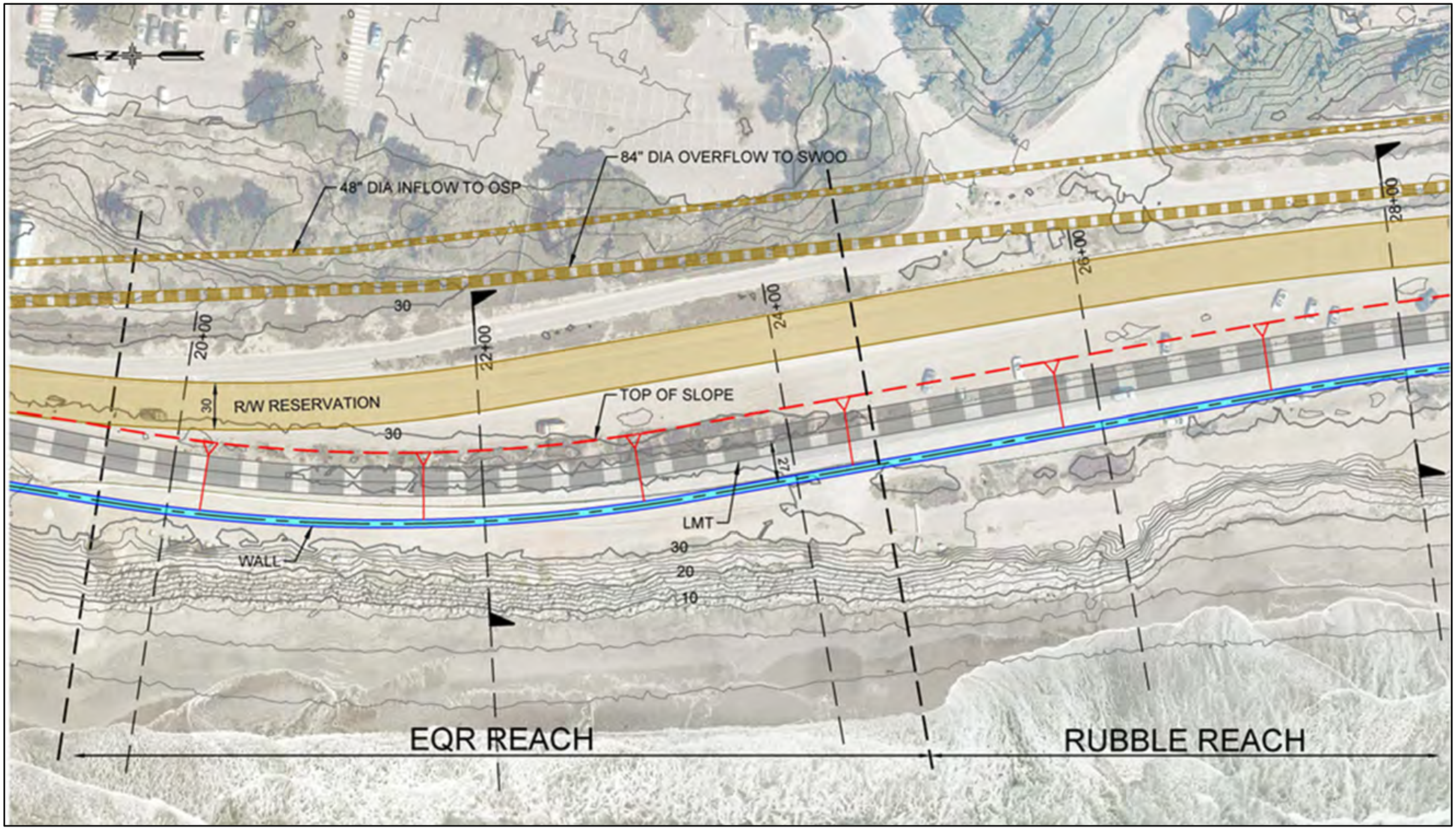


Figure 6-6: Wall Alignment Plan, 2 of 4.

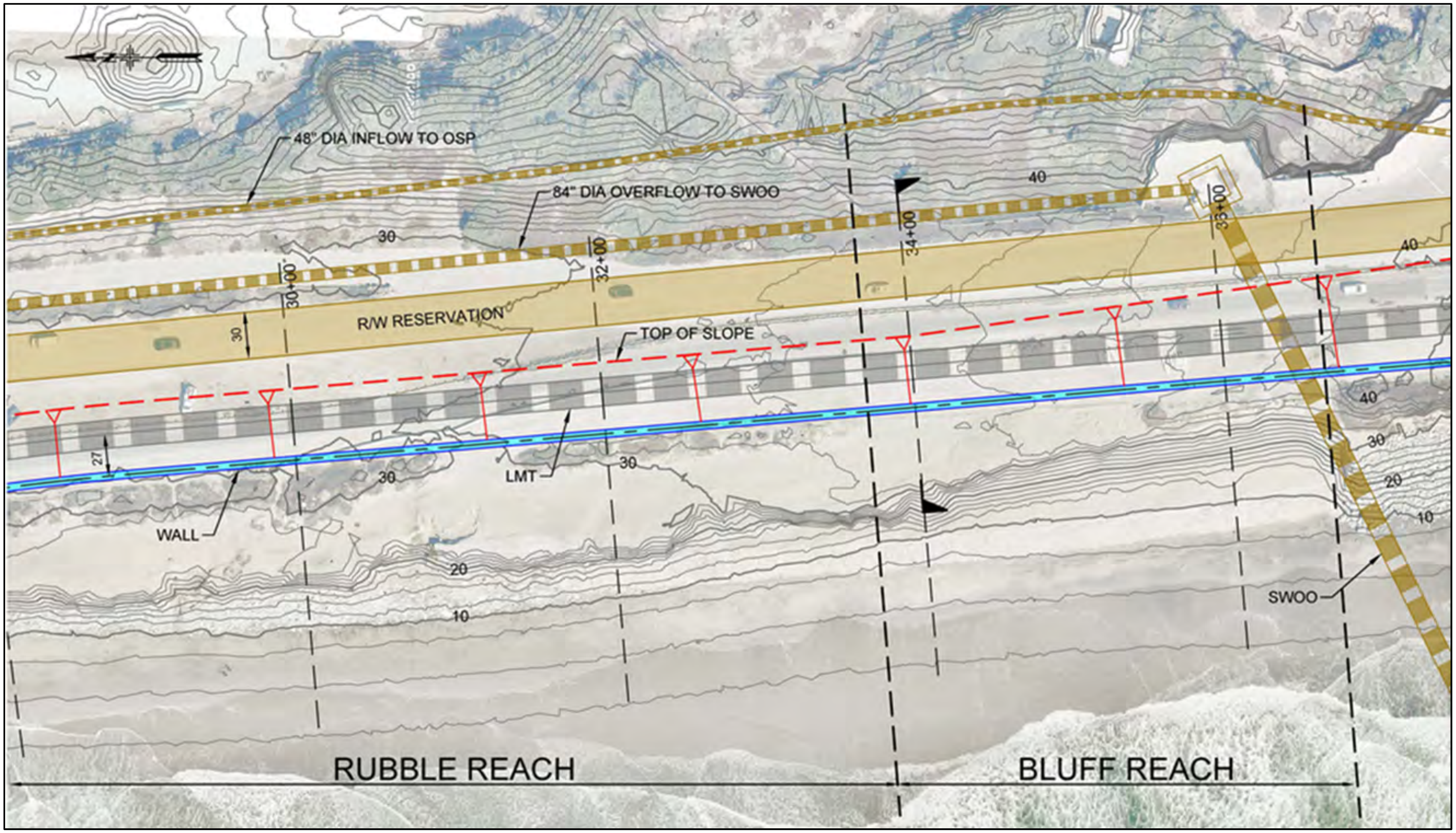


Figure 6-7: Wall Alignment Plan, 3 of 4.

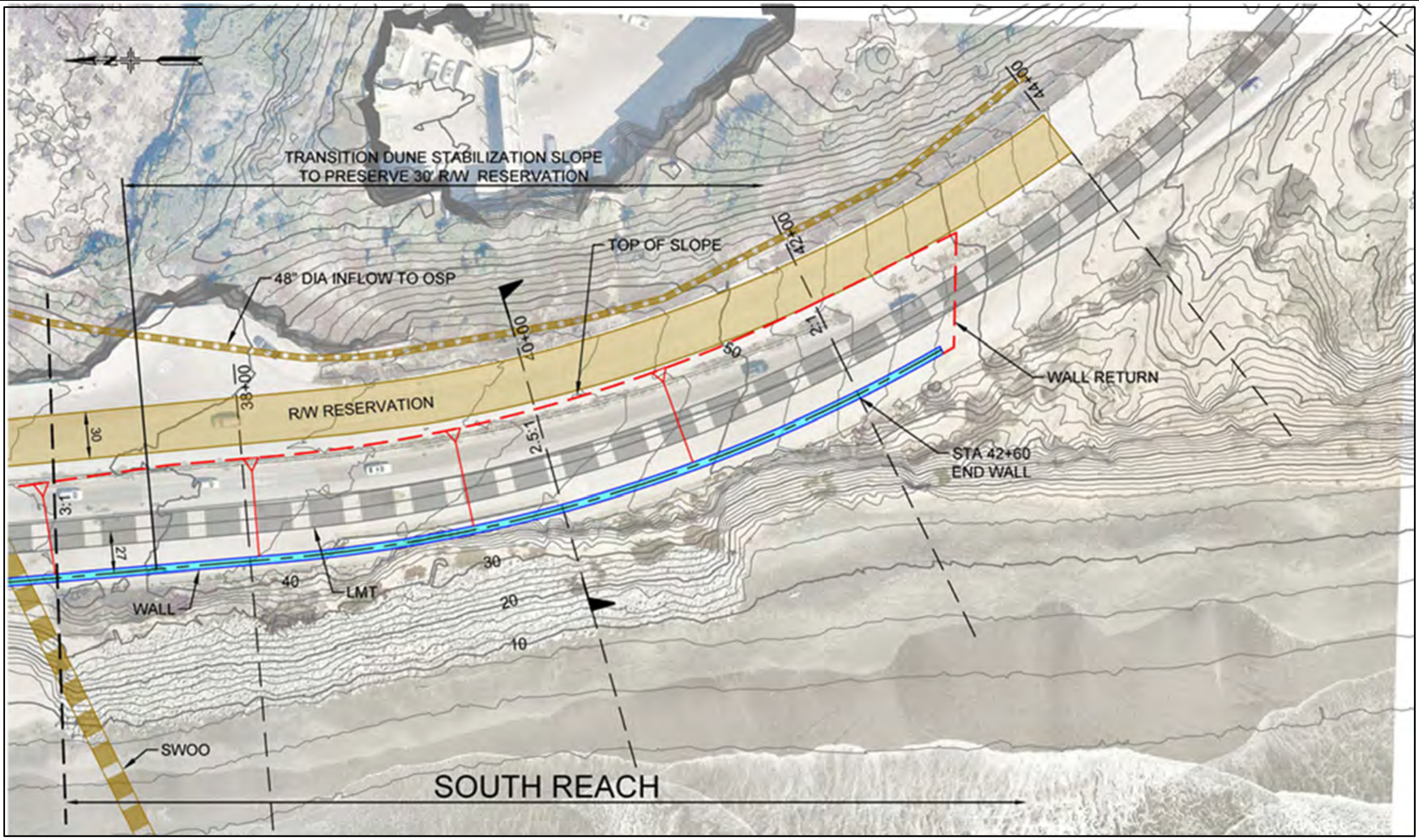


Figure 6-8: Wall Alignment Plan, 4 of 4.

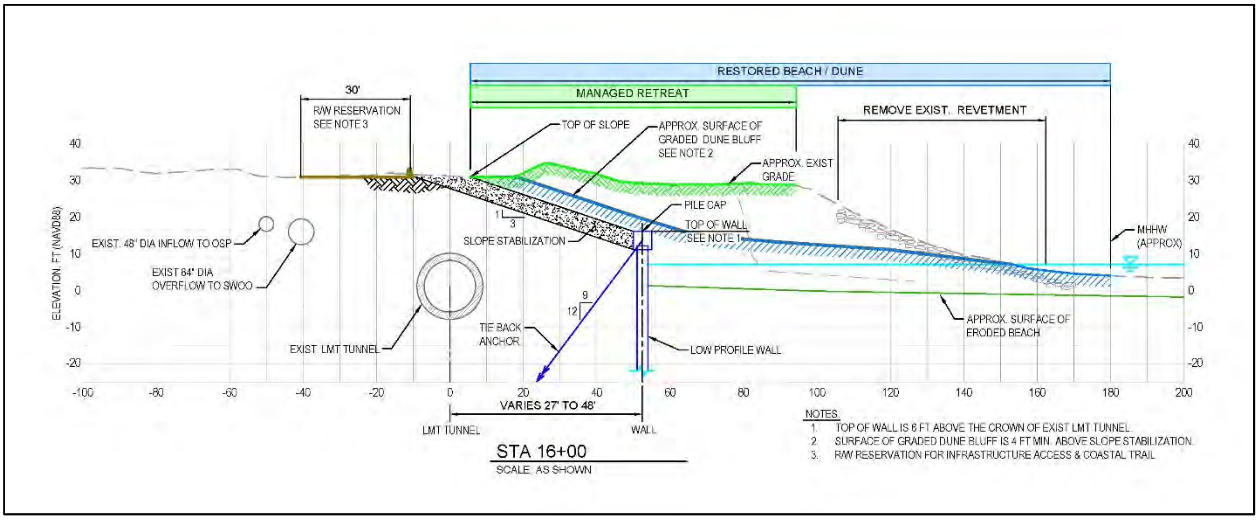


Figure 6-9: Low Profile Wall – North Reach Typical Section.

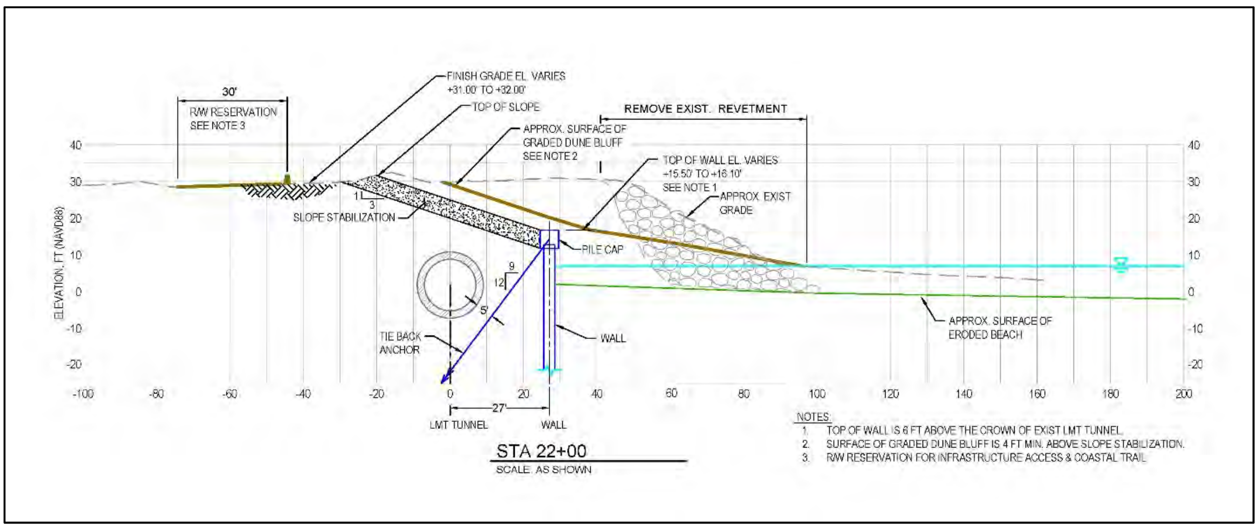


Figure 6-10: EQR Reach Typical Section.

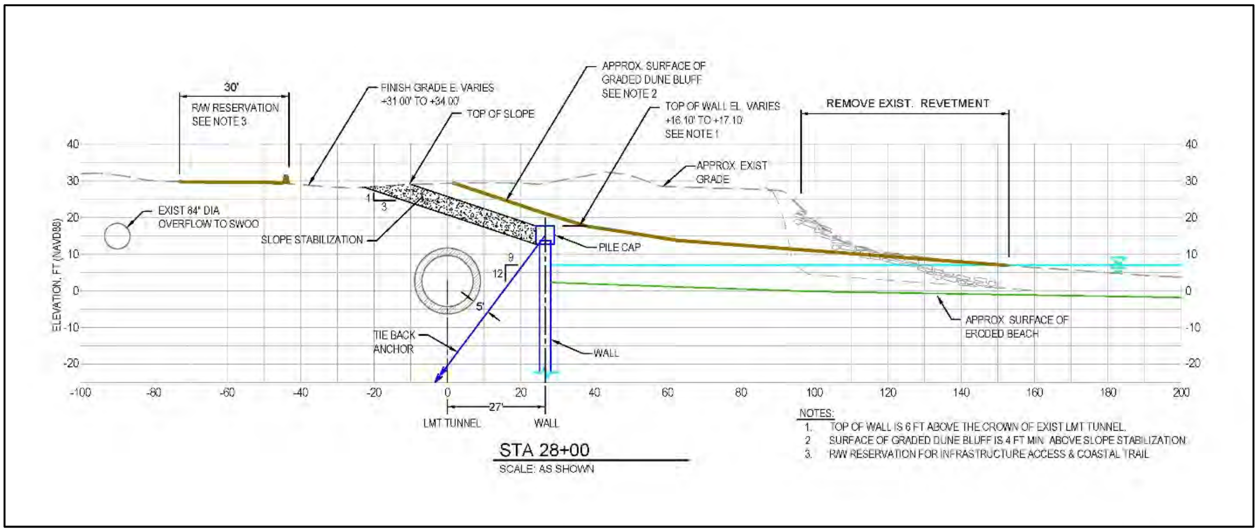


Figure 6-11: Rubble Reach Typical Section.

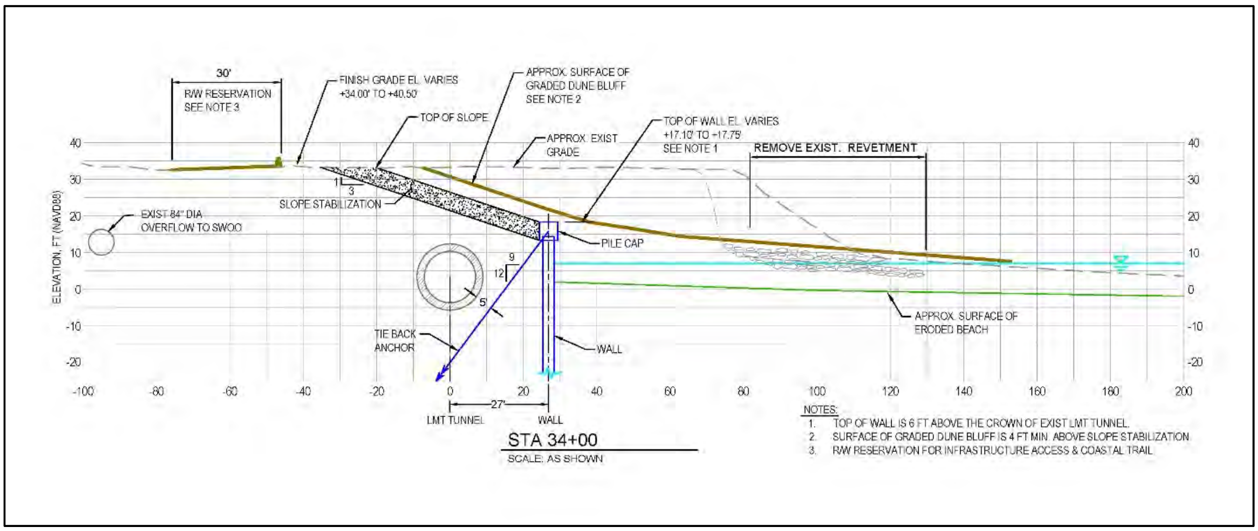


Figure 6-12: Bluff Reach Typical Section.

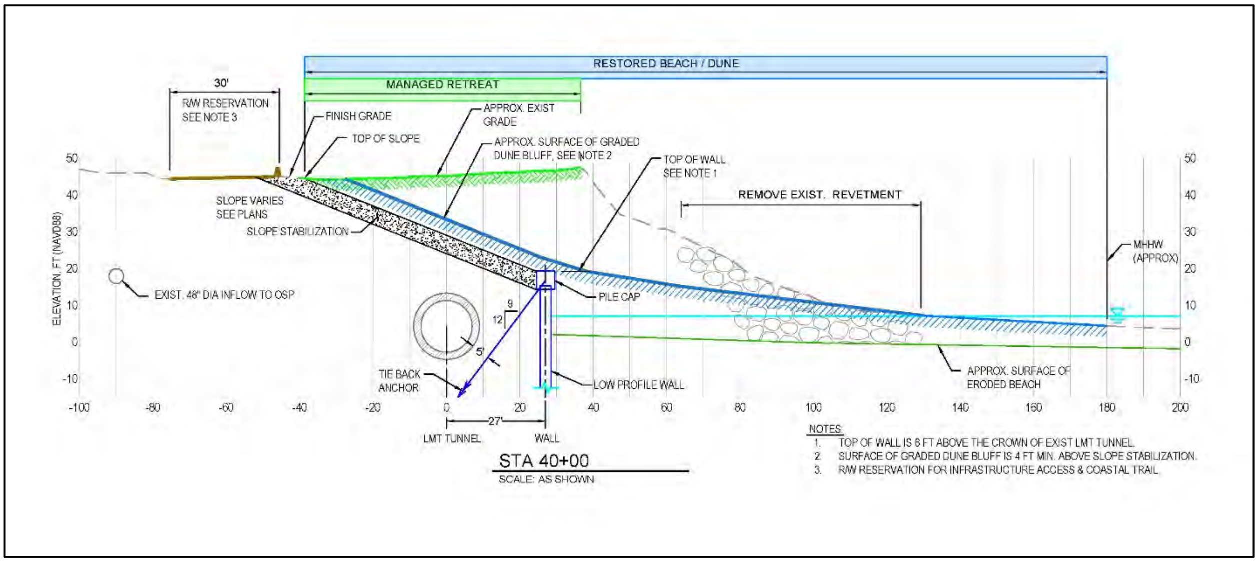


Figure 6-13: South Reach Typical Section.

6.5. Existing Utilities

Existing utility investigations were conducted to identify utilities surrounding the LMT and proposed wall alignment that may be affected by the project outlined in this report. Investigations of the site utilities include a site visit conducted on May 1, 2019, and by reviewing as-built drawings and CAD bases provided by the affected city jurisdictions and utility companies. Utility locating and potholing was not included in this investigation.

Below is a compiled list of utilities that may cause some interference with the proposed wall alignment and construction efforts. See Figures 6-14 to 6-17 for the compiled information on the existing utilities systems.

- Sewer/Stormwater: Most of the major sewer/stormwater facilities are along the eastside of the Great Highway for the WPS and OSP. According to as-built drawings provided by SFPUC, there is also a sewer/stormwater line that collects stormwater runoff along the coast side of the Great Highway from OSP to the parking lot across from the WPS before it turns and connects to LMT. Whether it is still active remains uncertain as most catch basins and manholes are buried by the sand along the coast. The as-built drawings show possible interference from the laterals that crosses over the LMT and proposed wall alignment from catch basins along the Great Highway (see Figures 6-14 to 6-17). There is also a portion of the combined sewer line that appears to cross over and will interfere with the proposed wall alignment (see Figure 6-15).

- Water: Information of the water utilities in this scope was limited to SFPUC's "Westside Pump Station Reliability Improvements" plans dated July 2018, which only shows a fire hydrant and valve, two meters and valve going into the WPS and a small section of water main on the eastside of Great Highway (see Figure 6-14). On the southwest corner of Sloat Blvd. and Great Highway, there is a building that contains restrooms and showers and a drinking water fountain in the plaza next to the building. Since the proposed wall alignment crosses this area, further investigation will be required to locate the water lateral into the building and to the water fountain.

- Natural Gas: According to the PG&E's as-built drawings, there is an abandoned 4" gas main that run west along the south side of Sloat Blvd. It then turns 90 degrees south along the west side of Great Highway for about 750 feet before it turns and crosses over to the east side of the highway (see Figure 6-14). There will be interference since this segment runs along and on top of the LMT.

- Electric: According to PG&E's as-built drawings, there is an electric line that runs along the south side of Sloat Blvd. and crosses over the Great Highway to a transformer located outside of the restroom building. No information was provided on how it feeds the restroom building therefore it is unclear if it will interfere with the construction efforts, but it is expected to be minimal.

- Traffic Signal: According to SFMTA's as built drawings, the traffic signal facilities are limited to the intersection of Sloat Blvd. and Great Highway, about 20 feet away from the proposed wall alignment (see Figure 6-14). Based on the information provided, there should be little to no interference to the construction efforts from the traffic signal utilities.

- Street Light: According to drawings provided by SFPUC and site investigations, most of the street light facilities are located at the intersection of Sloat Blvd. and Great Highway. However, there are street lights that run along the west side of Great Highway for about 780 feet south from Sloat. This segment runs along and on top of the LMT and will therefore interfere with the construction efforts.

Figure 6-14: Existing Utilities Plan 1.

Figure 6-15: Existing Utilities Plan 2.

Figure 6-16: Existing Utilities Plan 3.

Figure 6-17: Existing Utilities Plan 4.

7. Traffic and Intersection Layout

The closure of the Great Highway to through traffic from the junction of Skyline Boulevard (SR 35) to Sloat Boulevard will affect traffic circulation patterns and volumes in the project area. Access considerations for SF Zoo, due to the removal of the entry and exit lanes at the southern entrance to the lot, will also have to be addressed as part of the project. Finally, the removal of the restroom and parking spaces at the foot of Sloat Boulevard will affect coastal access parking, which will have to be addressed. The CER has identified the following work elements to address the above, as part of the proposed project:

- Redesign of Great Highway at Sloat Boulevard Intersection
- Activate Herbst Road Access to the Zoo
- Redesign of Great Highway at Skyline Boulevard Intersection
- Construct a new parking lot for coastal access (see Section 8 of CER)

A summary of existing conditions and proposed project features for all four of the above described elements is included in this section.

7.1. Existing Conditions

The Upper Great Highway is a four-lane roadway north of the Sloat intersection; the Great Highway is a continuation of Upper Great Highway south of the Sloat intersection, with two lanes in each direction with the southbound lanes reducing to one lane just south of Sloat. It carries approximately 25,000 daily vehicle trips during an average weekday and approximately the same on weekends[2]. Approximately 80 percent of the traffic in this segment of Upper Great Highway comes from the northbound Great Highway and approximately 20 percent come from Sloat Boulevard. The posted speed limit along Upper Great Highway is 35 miles per hour (MPH). There is an access to the OSP and to the San Francisco Zoo parking lot from the northbound lanes. There is a landscaped berm on the east side of Upper Great Highway with a ten-foot wide shared-use (pedestrian and bicycle) path. The intersection of Upper Great Highway and Sloat Boulevard has pedestrian crosswalks on all four sides. The proposed closure of Great Highway south of Sloat Boulevard is not expected to

[2] Automatic 7-day 24-hour tube counts collected at Great Highway south of Sloat Boulevard from August 22-28, 2018.

substantially change traffic volumes north of this intersection but would require traffic to detour to Skyline Boulevard and Sloat Boulevard in both directions.

The Lower Great Highway is a local access road, approximately 115 feet east of Upper Great Highway (north of Sloat Boulevard), separated by a raised landscaped berm on the west side. It provides local access to residential uses along the east side, terminating at Sloat Boulevard, and occasionally used as a bypass when Upper Great Highway is closed.

Sloat Boulevard is a four lane east-west road with two lanes in each direction and unregulated on-street parking in the center median in the vicinity of the Upper Great Highway and Sloat intersection. There is a primary pedestrian entrance to the San Francisco Zoo and a separate inbound only vehicular driveway to the Zoo parking lot on the south side of Sloat and residential and commercial uses on the north side. There are also bike lanes in both eastbound and westbound directions. Average daily traffic volumes along Sloat Boulevard in the project area are 10,000 vehicles per day on a typical weekday and approximately the same on weekends[3]. Sloat Boulevard effectively terminates at the intersection with Upper Great Highway. The posted speed limit along Sloat Boulevard is 35 MPH. Traffic volumes on hot summer weekends could be higher as there are more Zoo patrons and people heading to the beach. Currently the intersection of Sloat Boulevard and Upper Great Highway is signalized.

Skyline Boulevard is a four-lane road connecting Daly City to San Francisco and is Caltrans State Route 35 (SR35). It carries approximately 39,000 vehicles per day during an average weekday and approximately the same during weekends[4]. The posted speed limit along Skyline Boulevard is 50 MPH south of Great Highway and reduces to 40 MPH as it approaches Sloat Boulevard. As presented above, the closure of Upper Great Highway south of Sloat Boulevard will cause traffic to divert to Skyline Boulevard and Sloat Boulevard to reach Upper Great Highway north of Sloat Boulevard. Currently the intersection of Skyline Boulevard and Upper Great Highway is stop-controlled, with two westbound left turn lanes and one exclusive westbound right turn lane from Skyline Boulevard, however; Caltrans project is underway to signalize the intersection and add an ADA compliant crosswalk.

[3] 5-day 12-hour counts collected at Great Highway/Sloat Boulevard and Skyline Boulevard/Great Highway intersections from May 11-15, 2019.
[4] Ibid

7.2. Great Highway at Sloat Boulevard Intersection Redesign

The proposed project includes closure of the Great Highway to through traffic south of Sloat Boulevard, removal of the Muni bus stop/layover at the foot of Sloat Boulevard west of the Great Highway, and closure of the SF Zoo parking lot entrance and exit on Great Highway. The construction of the low profile wall will also require removal of the remaining coastal access parking lot and restroom near this intersection; a new parking lot will be constructed at the southern end of the Great Highway just west of the Skyline intersection to replace coastal access parking (see Section 8 of CER). The design of the intersection will address the following key issues:

- Muni Line 23 route, last outbound and first inbound stops, and the layover area.
- Bicycle and pedestrian access to the beach.
- Traffic control at the intersections of Sloat Boulevard at Upper Great Highway, Lower Great Highway, and 47th Avenue.
- SF Zoo access – The Great Highway closure will remove the SF Zoo parking lot access and exit along Great Highway. This change will require modification to both inbound and outbound access to the Zoo parking from Sloat Boulevard only or a combination with an access from both Sloat Boulevard and Herbst Road.
- SFPUC facilities access – The Great Highway closure will remove the SFPUC Westside Pump Station and Oceanside Water Pollution Control Plant access along the Upper Great Highway. A single northbound access lane will likely be retained along the coast Highway from Skyline Boulevard to Sloat Boulevard as an SFPUC service road. Other options are being considered.

The intersection is currently signalized with two lanes in each direction for the north, south, and east legs of the intersection. Both the northbound and southbound approaches have one left turn, two through, and one right turn lane with protected left turn signal phasing. The eastbound approach has one unmarked travel lane. The westbound approach has one right turn, one left turn, one through/left turn lane, and a bike lane. The eastbound and westbound traffic have split phasing. Since the SF Zoo's major pedestrian and parking access are located on the south side of Sloat Boulevard, this intersection is used also by vehicles accessing the San Francisco Zoo's parking lot.

West of the intersection, the terminus of Sloat Boulevard is "U" shaped with one wide lane for vehicles to turn around and provides access to the public beach parking lot and public restroom facilities. It also serves as the Muni Line #23 Monterey bus turnaround and layover area facility.

7.2.1. Intersection Layout

The concept design for this intersection is based on the following considerations:

Traffic Volume and Level of Service (LOS): Traffic volume estimates and intersection LOS analyses for both future year (2040) with and without closure of Great Highway between Skyline Boulevard and Sloat Boulevard have been analyzed in prior studies (2014 Ocean Beach Master Plan Transportation Operations and Alternatives Study). The 2014 Study estimated future traffic volumes will increase by approximately 0.5% percent annually; it also concluded that the intersection of Sloat Boulevard and Upper Great Highway could accommodate estimated future year (2040) traffic volumes with a reconfigured "L" shape design with two travel lanes in each direction. On the basis of the above findings, an "L" shape design for the Great Highway at Sloat Boulevard intersection has been adopted.

Entrance to Zoo: Given the closure of the SF Zoo parking lot entrance and exit on Great Highway, traffic to/from the Zoo parking lot would have to be accommodated from the existing Sloat Boulevard access point. There have also been discussions with Zoo staff about opening the Zoo Road access for public use; currently it is a gated facility reserved for Zoo employees only. The Zoo Road option is also described in this CER. All options are still being discussed with the Zoo and final agreements on the appropriate elements will be included in the design documents.

Accommodating ingress/egress from the existing Sloat entrance could be accomplished by either incorporating one additional lane (reversible during peak times) for a total of three lanes, or widening the entrance to four lanes (two entry and two exit), which will require relocating the Zoo access to 47th Street. Both design options have been developed for the CER for discussions with the Zoo, as described in this section. Other options may be considered during 35% design.

Muni Line 23 Turnaround: Given the proposed removal of the Muni Line 23 Monterey bus turnaround and stop/layover at the foot of Sloat Boulevard, an alternative for bus turnaround and layover will have to be accommodated. Three separate options were developed for the CER as described in this section; subsequently discussions were held with MTA.

<u>Intersection Option 1: Zoo Access Expanded to Three Lanes</u>

This option would implement an "L" shape intersection design with two travel lanes in each direction (westbound Sloat to Upper Great Highway, and southbound Upper Great Highway to Sloat) and widening of the Zoo driveway to three lanes as presented in Figure 7-1. It will include a one-way northbound SFPUC access road that will allow northbound SFPUC vehicles to connect to Upper Great Highway and Sloat Boulevard. Other intersection modifications include:

- A new westbound U-turn only lane within the existing Sloat Boulevard median to allow for those who wish to change their path of travel prior to entering Upper Great Highway.
- A new dedicated left turn lane in the westbound Sloat Boulevard direction at 47th Avenue, to allow a direct access to SF Zoo parking lot.
- Signal modifications with necessary interconnect to allow proper progression and safe pedestrian and bicycle crossings.
- Existing pedestrian crossing on the north side of the intersection will be removed to allow for safer pedestrian and bicycle crossing at the east crossing and to reduce the number of conflicting movements at the intersection.
- Existing pedestrian crossing on the west side of the intersection will be removed as there is no more crossing at this location.
- Existing pedestrian crossing on the south side of the intersection will be reduced to the width of the northbound local access road.
- Existing pedestrian crossing on the east side of the intersection will be widened to 12 feet as this will be the only pedestrian crossing at the intersection.
- The east side pedestrian crosswalk will include a separate 10-foot wide dedicated bicycle crossing alongside.

This option would maintain the existing bike lanes along Sloat Boulevard and Upper Great Highway and the shared-use path on the berm east of Upper Great Highway. The layout would add new protected bike areas, including the new proposed multi-use trail for shared pedestrian and bicycle use south of Sloat Boulevard.

The existing Zoo driveway at Sloat Boulevard would be widened by approximately 10 feet and restriped to accommodate one ingress and one egress, and one reversible travel lanes to allow for two ingress lanes during inbound peak hours and two egress lanes during outbound peak hours. The current Zoo peak entry and exit on a summer weekend is approximately 200 vehicles per hour. Two lane entry or exit would be sufficient to accommodate the demand. Egress traffic is limited to right- out only turns onto eastbound Sloat Boulevard. To minimize the number of U-turns at the Sloat/Great Highway intersection, this option includes adding a dedicated westbound left-turn in the median to allow access into the Zoo parking lot. This new westbound left turn lane would be extended upstream past the existing Sloat Boulevard and 47th Avenue intersection, creating sufficient left turn storage space for Zoo patrons.

This option would require signal interconnect and coordination of traffic signals at the Upper Great Highway and 47th Ave intersections, so the proposed westbound left turn movement will have a protected left turn phase at the 47th Ave intersection. A signal offset will be added to the 47th Ave intersection so that the eastbound vehicles coming from Upper Great Highway will clear the intersection prior to the westbound left turn signal turning green. Additional signage and striping will be required at the Zoo driveway to direct safe pedestrian and bicycle crossings.

Intersection Option 2: Zoo Access Moved to 47th Ave. Intersection and Expanded to Four Lanes

This option would implement an "L" shape intersection design similar to Option 1, realignment of the existing Zoo driveway on Sloat Boulevard to the 47th Avenue intersection to ease traffic operation, two ingress and two egress lanes, and a dedicated right-turn lane and left turn lane to the Zoo parking as shown on Figure 7-2. New high-visibility crosswalks would be striped to align with the new Zoo driveway across the south and west legs of the Sloat Boulevard and 47th Avenue intersection. This option would provide a conventional intersection layout for the Zoo access, easier for drivers to enter and exit the Zoo parking, but would require significant grading, removal of miscellaneous structures and operational changes within the zoo. Other intersection modifications would be the same as described for Option 1. Upon discussions with SF Zoo staff, this option has been deemed infeasible and has been eliminated from further discussions.

Muni Bus Operations

Three options have been identified to accommodate Muni #23 bus routing, stops, and layover location as described below and as presented on Figure 7-3 through Figure 7-5:

- Option 1: Layover at existing last bus stop on Sloat Boulevard
- Option 2: Layover on south side of Sloat Boulevard
- Option 3: Layover on Lower Great Highway

Option 1 – Layover at existing last bus stop on Sloat Boulevard: This option would maintain the existing bus stop (last stop) located along the north side of Sloat [Boulevard between Lower Great Highway and 47th Avenue, but would reroute #23 bus from Sloat Boulevard clockwise to Lower Great Highway, Wawona Street, and 47th Avenue, back to Sloat eastbound. The layover space will share with the last stop and Muni employees will use the existing L-Taraval employee restroom. The Sloat Boulevard and 47th Avenue intersection is signalized, making it easier for #23 buses to reach the first return stop at the existing bus stop located just east of the SF Zoo main pedestrian entrance.

This option is SFMTA's preferred option because it provides a simple routing change and safe and efficient access for #23 buses to turn back to its return route. The bus layover location would be within a short walking distance to the existing L-Taraval employee restroom and away from heavy traffic on Sloat Boulevard.

Option 2 - Layover on south side of Sloat Boulevard: This option would reroute the #23 bus route to make a U-turn from westbound Sloat Boulevard onto eastbound Sloat Boulevard, after making its final outbound stop at the existing bus stop along the north side of Sloat Boulevard just west of 47th Avenue. A new 40-foot-long on-street bus layover facility would be provided along the south side of Sloat Boulevard (west of the Zoo driveway). This layover space would also be within 250 feet of a proposed new public restroom at the southwest corner of Sloat Boulevard and Great Highway. After buses have completed their layover, they would pull-out onto eastbound Sloat Boulevard and make their first stop at the existing bus stop at 47th Avenue.

This option is not recommended by SFMTA because it would require buses to cross two lanes of traffic to make the U-turn, which could be difficult during the peak traffic period. However, this option would not require additional right- of-way, changes in Muni bus routing and no removal of street parking. The proposed bus layover facility will be relocated to the south side of Sloat Boulevard.

Option 3 - Layover on Lower Great Highway: This option would relocate the last outbound bus stop on westbound Sloat Boulevard from the west side of 47th Avenue to the east side of the intersection. Buses would use 47th Avenue, Wawona Street to reach Lower Great Highway for the return trip and the bus layover space would be located on the west side of Lower Great Highway. The new layover facility would be immediately adjacent to the existing (closed) Wawona public restroom on Lower Great Highway. The return trip back to Sloat Boulevard would require redesign of the intersection of Lower Great Highway and 47th Avenue by modifying the existing median.

While bus routing is simple, and bus layover area would not impact access to any residential parking, (relocation of only three parking spaces on the west side of Lower Great Highway needed), it would significantly affect the design of the intersection due to the mid-block bus crossing and the need for adding another traffic signal at Lower Great Highway. SFMTA does not recommend this option.

7.3. Great Highway at Skyline Boulevard Intersection

The intersection is currently a three- way stop controlled intersection, with free northbound through and eastbound right movements. There are no pedestrian crosswalks or on-street bicycle facilities at this intersection. However, a shared pedestrian and bicycle path is located on the east side of Skyline Boulevard along Lake Merced.

The design of this intersection would require the following key issues to be addressed:

- Bicycle and pedestrian access to the existing trails and to the beach.
- Coordination with Caltrans
- Bus line 57
- Traffic control

7.3.1. Intersection Layout

Vehicular access to the proposed 50+ space parking lot (see Section 8 of CER) will need to be maintained at this intersection; access for SFPUC service vehicles will also have to be maintained. Access to the SFPUC Westside Pump Station and the Oceanside Water Pollution Control Plant would be maintained via a single northbound service/maintenance vehicle lane. The 2014 Ocean Beach Master Plan Transportation Operations and Alternatives study analyzed various future year (2040) configurations of Great Highway from Skyline Boulevard and Sloat Boulevard, ranging from no change to full closure. It indicated the intersection of Great Highway and Skyline Boulevard would operate at LOS E and F conditions with Great Highway operating as a two lane road (one lane in each direction) in both directions.

Due to safety concerns, Caltrans and San Francisco Public Works (SFPW) have developed a concept plan for signalization of this intersection as presented in

Figure 7-6. The design would change the existing traffic control from stop-controlled to signal controlled to allow for a pedestrian/bicycle crosswalk to be installed on the south leg of the intersection. It also reconfigures the southbound free right lane from Skyline Boulevard to Great Highway.

The Caltrans/SFPW concept plan will be refined once the parking lot location and access to the parking have been determined. The traffic signal will also require reprogramming.

7.4. Activate Zoo Road

Zoo Road is currently a gated private road mainly for Zoo employees and deliveries. Zoo Road is a connecting road between the SF Zoo parking lot and Herbst Road. Herbst Road is a one-way southbound public road between Skyline Boulevard and Armory Drive. The inbound access to Herbst Road from Skyline Boulevard is signalized with an exclusive northbound left-turn lane with protected signal phasing. The outbound access from Herbst Road to Skyline Boulevard is side-street stop controlled. East of Armory Drive Herbst Road is a forked road divided by the Pomeroy Center into a single lane westbound entry with perpendicular parking on both sides of the road and a single lane southeast bound exit also with perpendicular parking on both sides of the road along the Pomeroy Center frontage.

7.4.1. Access Concept

In general, Herbst Road is a low volume road serving access to the Pomeroy Center, the National Guard, and the SF Zoo employees and deliveries. The SF Zoo is considering opening their gated access road (Zoo Road) to the public to provide an alternate access to the Zoo parking lot.

The design of this road segment would address the following key issues:

- Open road to the public at the Zoo entrance
- Maintain current supply of unregulated on-street parking on Herbst Road
- Maintain traffic controls at Skyline Boulevard and coordination with Caltrans
- Narrow roadway width along Zoo Road
- Design animal crossing for safe passage of zoo animals to clinic

In order to allow Zoo patrons to access the existing Zoo parking lot or proposed parking area via Herbst Road, a portion of Zoo Road will need to be widened by approximately 10 feet and/or some on-street parking removed. Currently, there is a 200’ stretch of Herbst Road just west of the Employee Parking and Truck Delivery driveways that is 20’ wide with on-street parallel parking allowed on the north side of the road. Refer to Figure 7.7 for a satellite view of the area. To open this stretch of roadway to provide public access to the Zoo parking lot, Zoo Road should have a minimum of 22 feet width with a centerline stripe for two-way vehicular traffic circulation. In addition, Zoo Road currently does not have any pedestrian facilities. If Zoo Road is to be opened to allow for pedestrian access, a minimum 6’ wide pedestrian pathway/sidewalk will need to be constructed along with installation of crosswalks at the intersection of Herbst Road and Armory Drive. Additional intersection traffic control measures may be required at the intersection of Herbst Road and Armory Drive. This modification would allow sufficient roadway capacity (400 vehicles per hour in each direction) to accommodate Zoo parking lot access. Current summer weekend peak entry and exit to the Zoo parking lot is less than 200 vehicles per hour. No additional improvements are anticipated at the Herbst Road and Skyline Boulevard intersections.

7.5. Design Criteria for Traffic and Intersections

Intersection and lane geometry design will follow standards and guidelines established by the Caltrans Highway Design Manual, California Manual on Uniform Traffic Control Devices (CAMUTCD), Federal Highway Administration's (FHWA) Manual on Uniform Traffic Control Devices (MUTCD), American Association of State Highway and Transportation Officials (AASHTO) A Policy on Geometric Design of Highways and Streets, and National Association of City Transportation Officials (NACTO) Urban Bikeway Design Guide.

- Travel Lane width – No change to current lane widths. (Generally, 11 feet for Upper Great Highway and Skyline Boulevard. Lane width along Sloat Boulevard will be 10-12 feet.)
- Bicycle lane width – No change to current lane widths.
- Sidewalks width – No change to current width.
- Speed limit – No change to current posted speed limits.
- Traffic Control – Additional traffic signal control will be assessed based on the CA MUTCD traffic signal warrant analysis.
- Traffic volumes – Future year (2040) traffic volumes for the intersection design will be based on the Ocean Beach Master Plan Transportation Operations and Alternatives Analysis, prepared by AECOM on June 20, 2014.

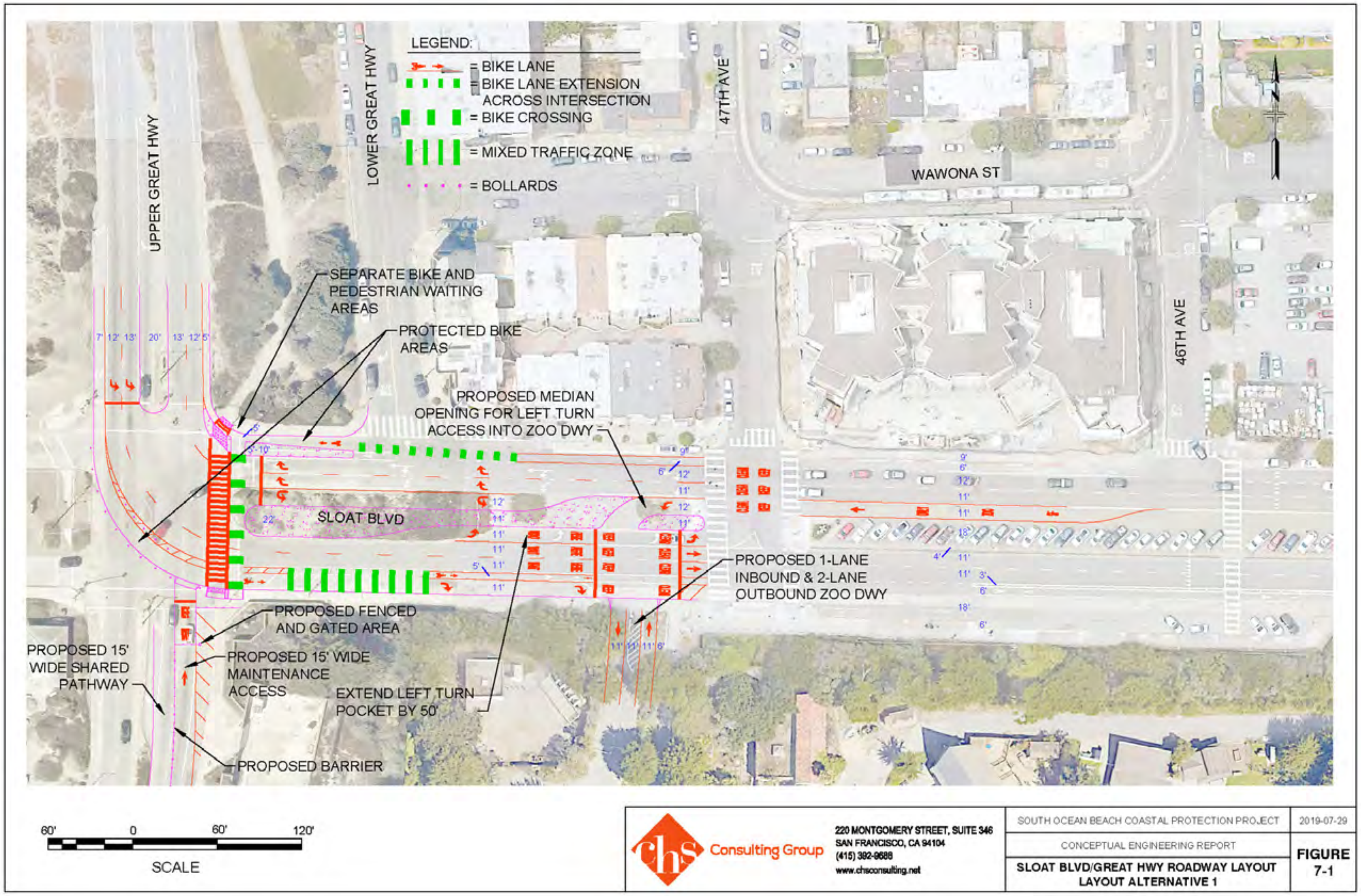


Figure 7-1: Great Highway at Sloat Boulevard Intersection – Alternative 1

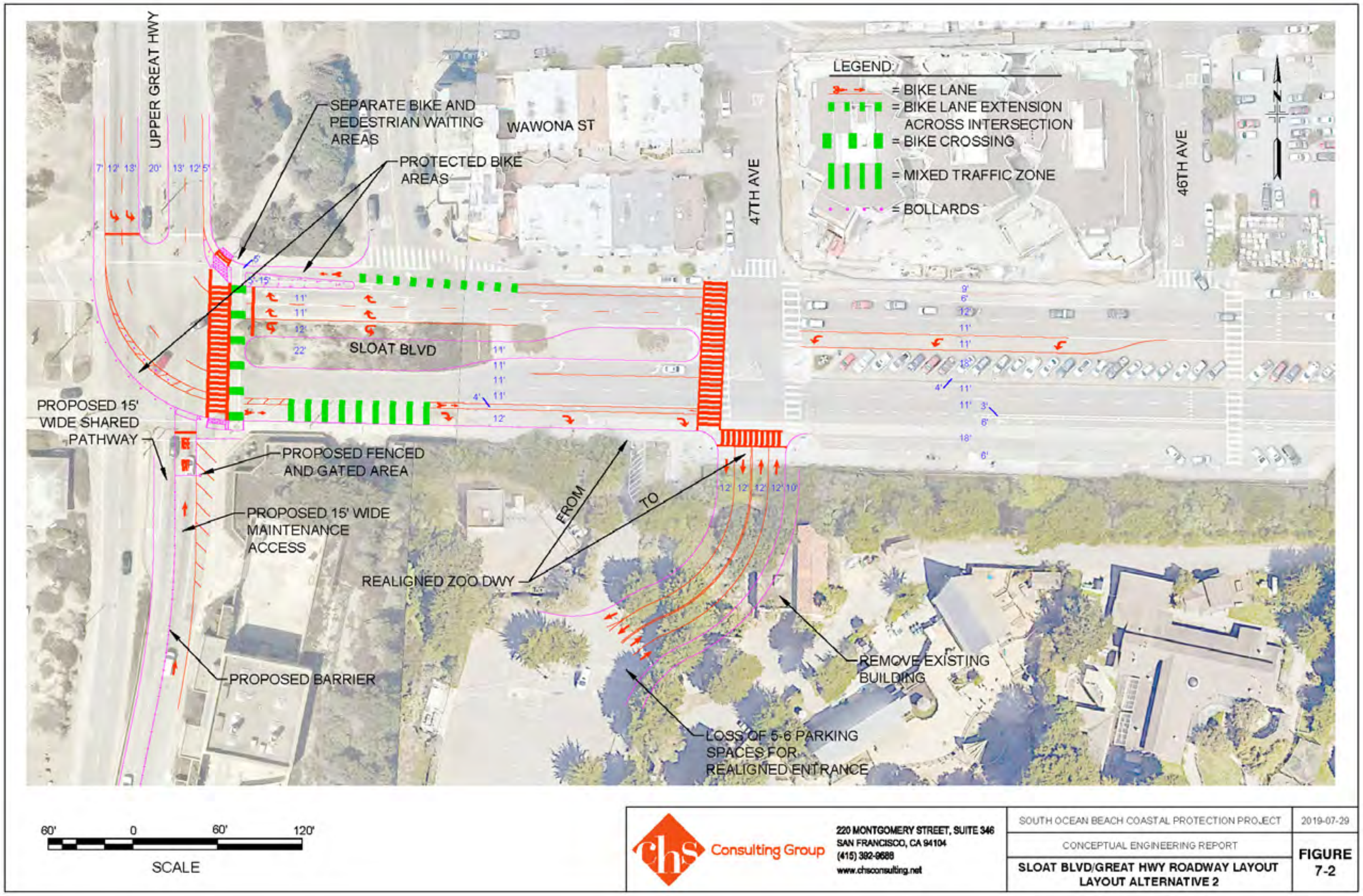


Figure 7-2: Great Highway at Sloat Boulevard Intersection – Alternative 2

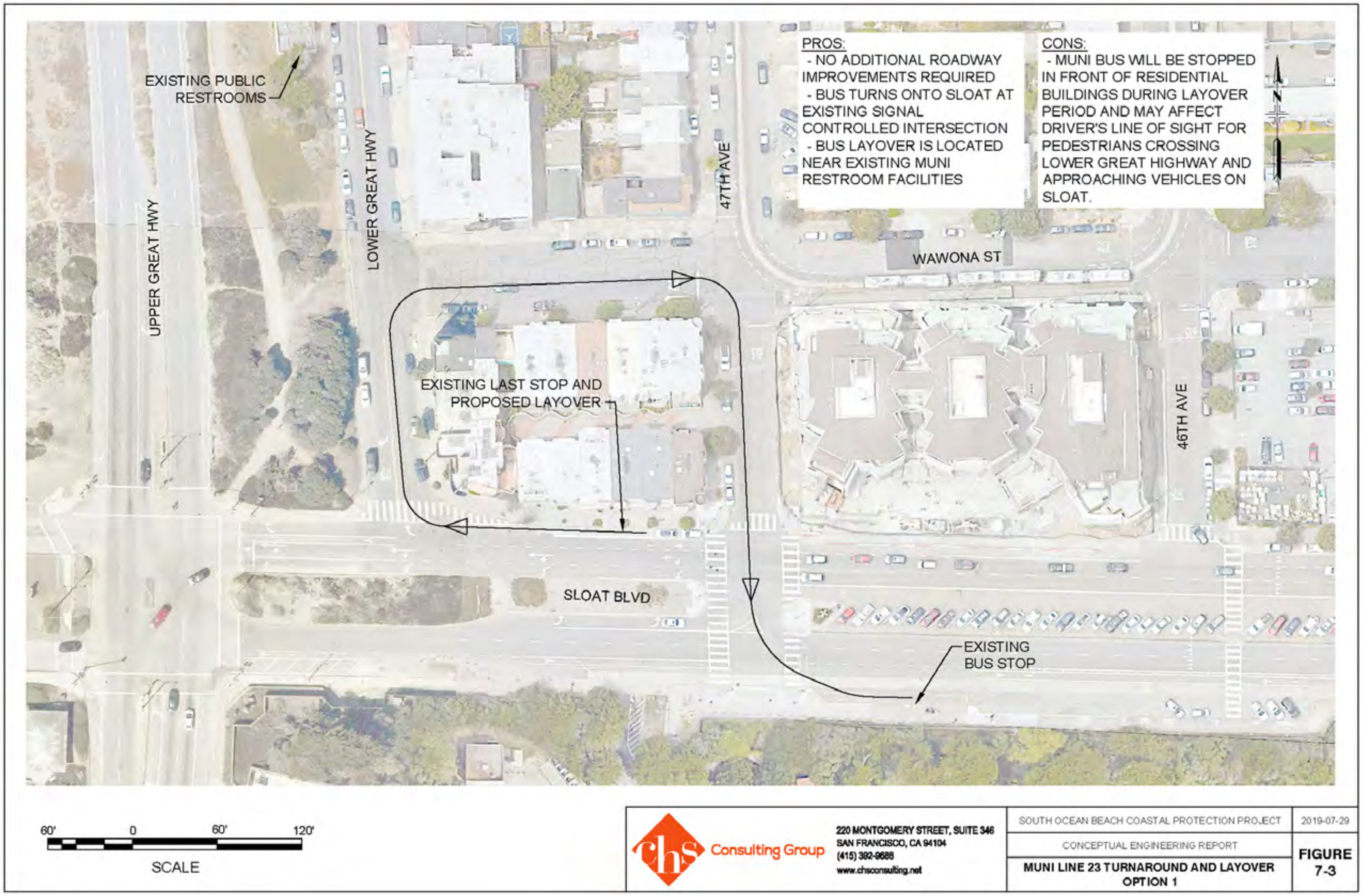


Figure 7-3: Muni Line 23 Turnaround and Layover – Option 1

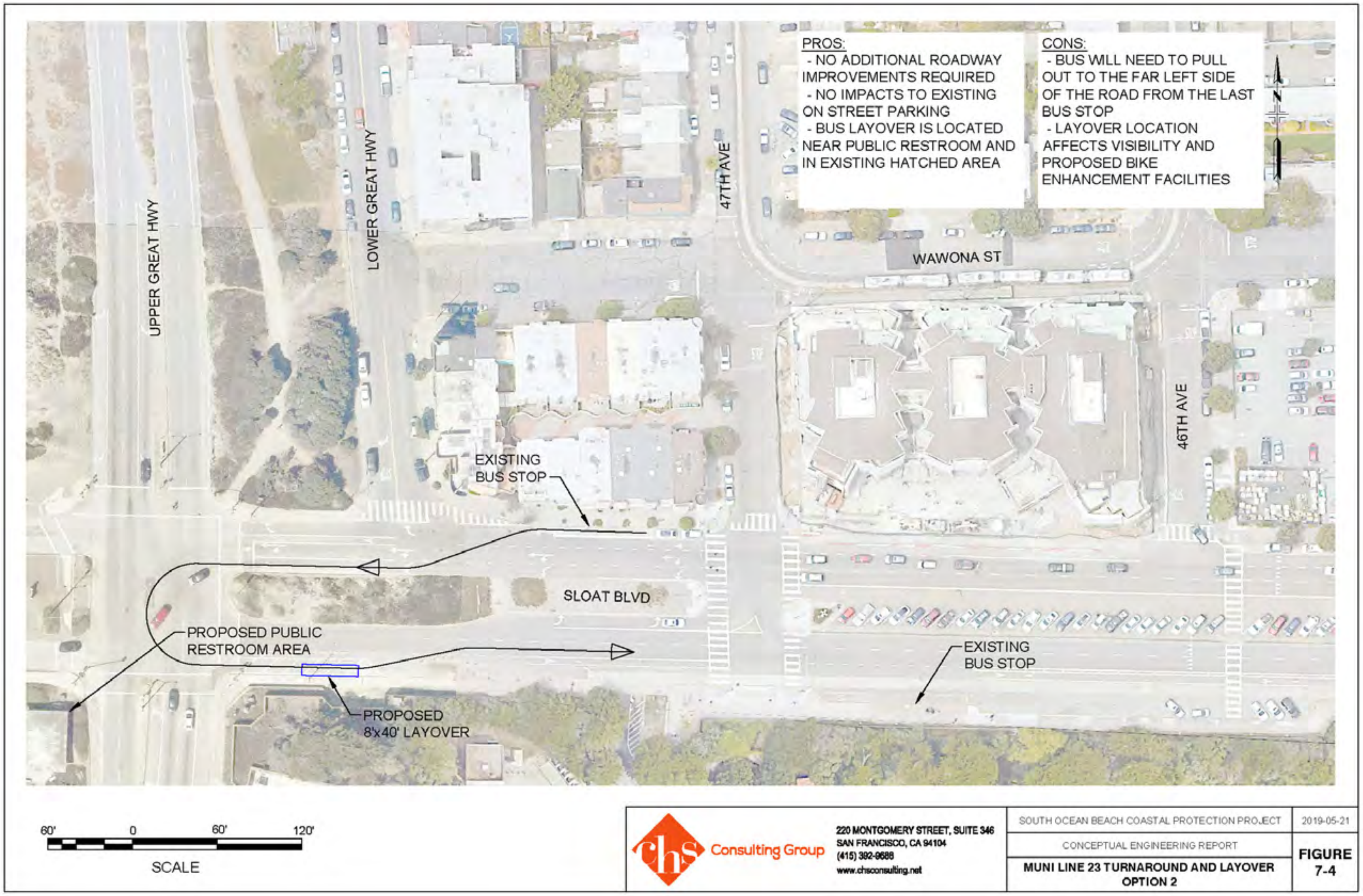


Figure 7-4: Muni Line 23 Turnaround and Layover – Option 2

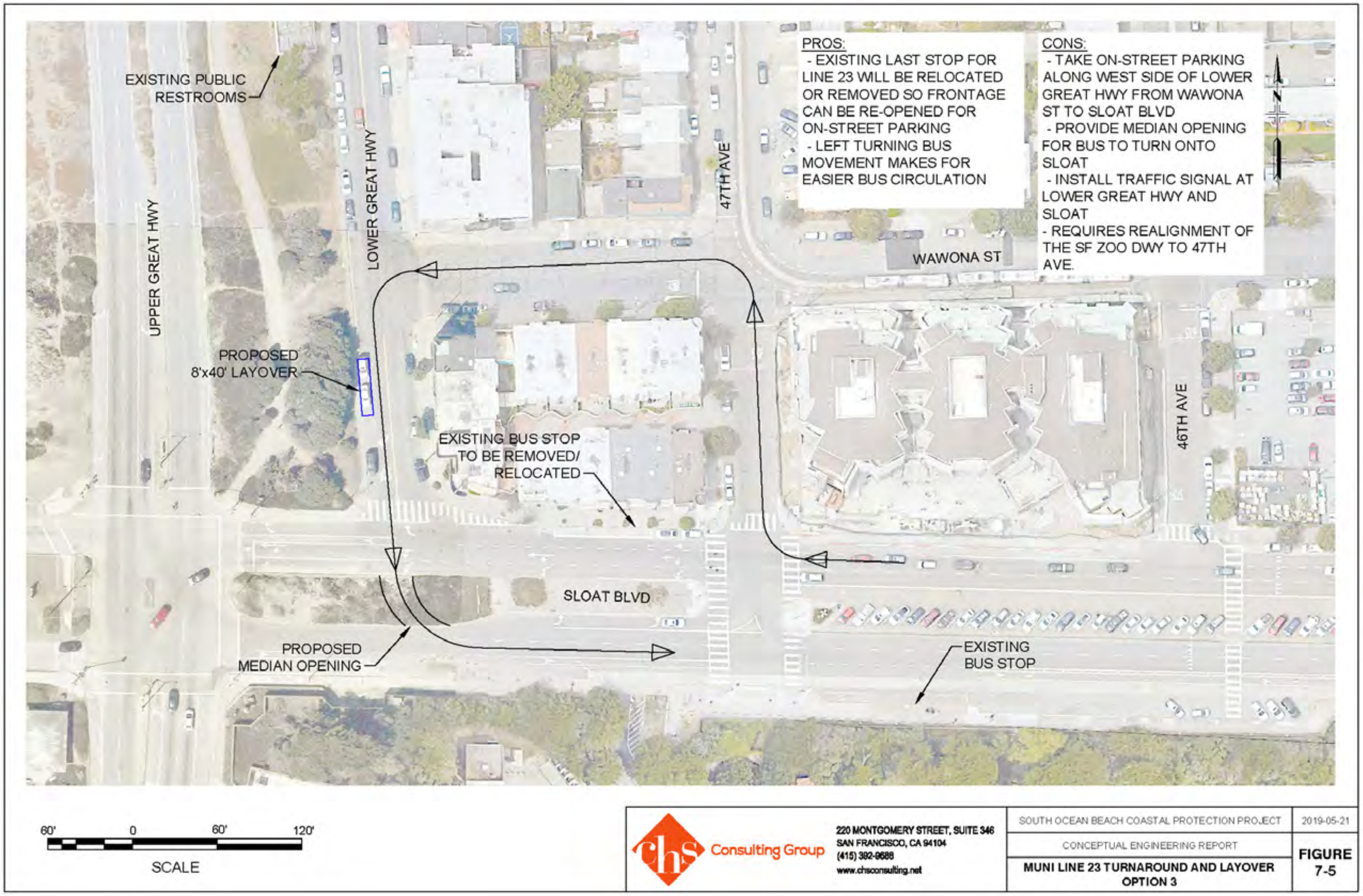


Figure 7-5: Muni Line 23 Turnaround and Layover – Option 3

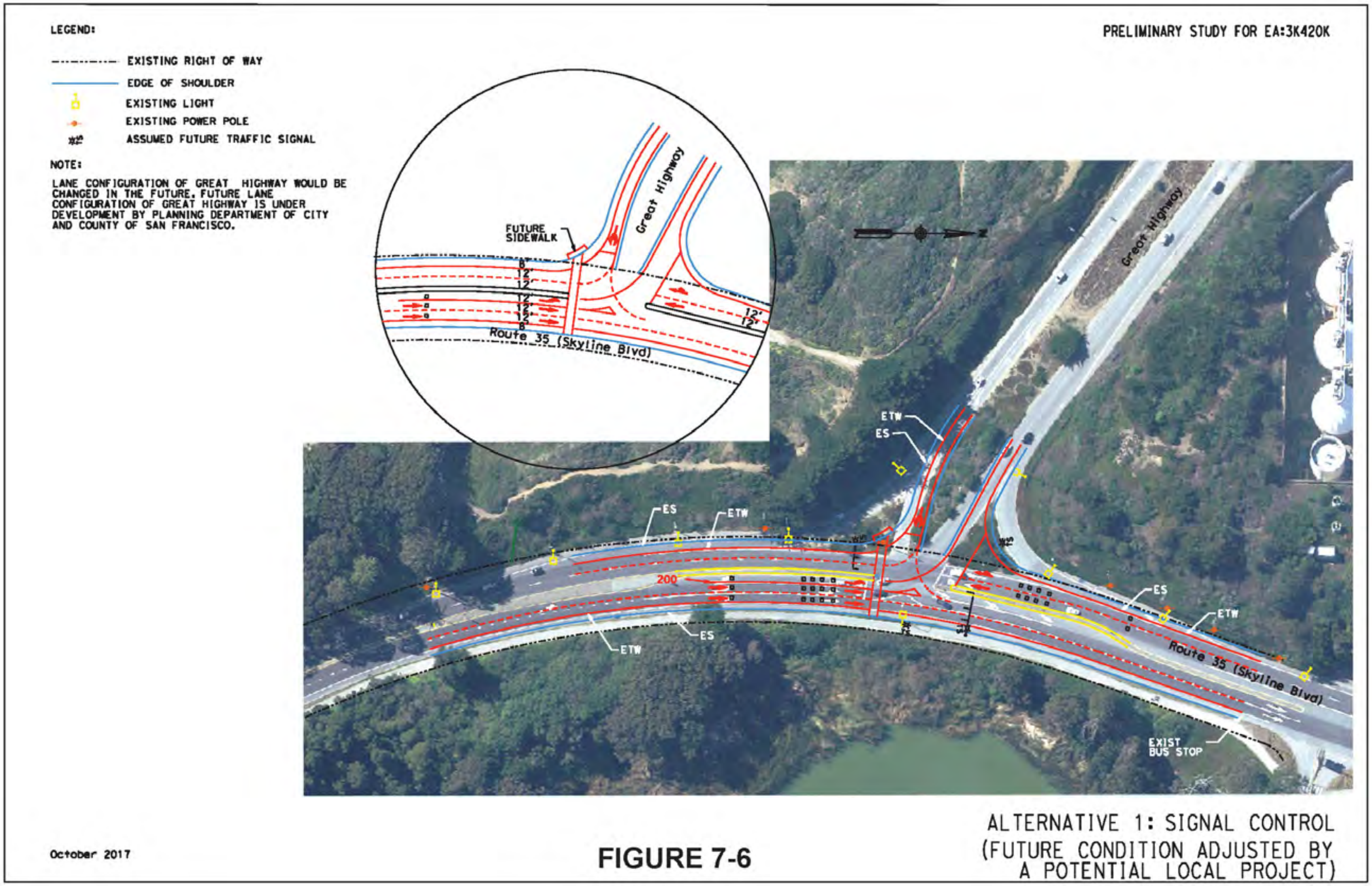


Figure 7-6: Great Highway at Skyline Boulevard Intersection (Caltrans/SFDPW Concept)

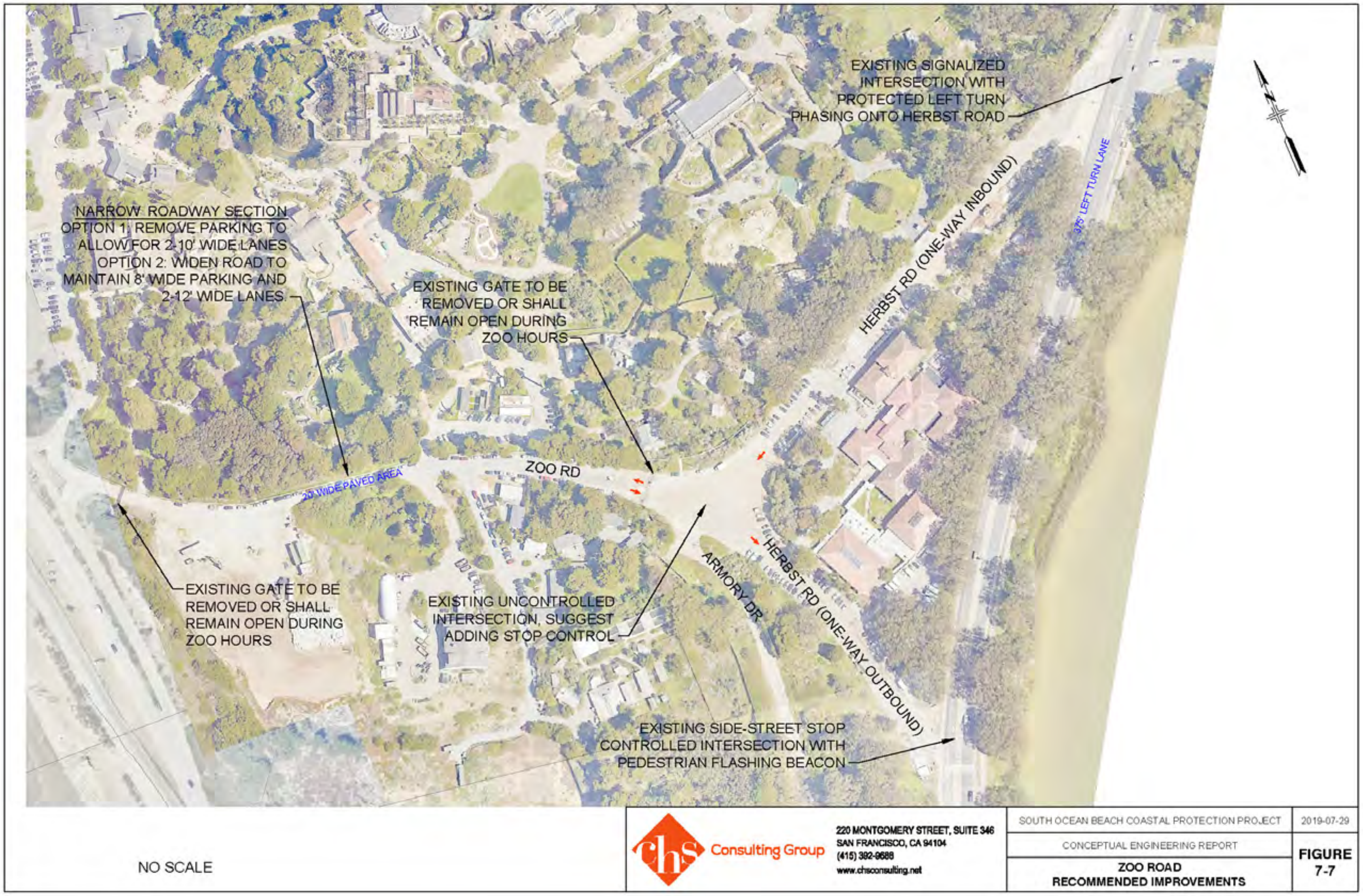


Figure 7-7: Zoo Road Access Concept

8. Multi-Use Trail, Beach Access, Parking, and Service Road

8.1. Open Space/Multi-Use Trail and Beach Access

The Lower Great Highway multi-use trail located on the existing median parkway between the Upper Great Highway and the Lower Great Highway, connects from Sloat Boulevard 3.5-miles north to near the foot of the Cliff House at Balboa Street. The existing trail is owned and operated by San Francisco Recreation and Parks. The proposed project would extend this trail from Sloat Boulevard, along the Great Highway, to Skyline Boulevard.

As identified in the *Ocean Beach Master Plan* there is a significant gap in bicycle and pedestrian connectivity between Ocean Beach and Lake Merced. Key Move 1 of the *Master Plan* calls for the introduction of a new multi-modal coastal trail to provide this connection, while allowing enhanced beach access along this southernmost reach.

The proposed multi-use trail will link the new proposed parking lot at Skyline Boulevard to the reconfigured terminus of Sloat Boulevard at Great Highway as shown on Figure 8-1.

<u>Trail Alignment and Access Points</u>

The trail alignment will run parallel to the proposed SFPUC Service Access Road that will link Skyline Boulevard to the Westside Pump Station. For trail user safety and to prevent conflicts with PUC vehicles, the trail will be separated from the Access Road by a low visual-impact vertical barrier (see photo in Figure 8-2).

The southernmost end of the trail will extend from the reconfigured and signalized intersection at Skyline Boulevard, and allow access connectivity from the Lake Merced loop trail, the Fort Funston trail network, and the new proposed parking lot.

Between Skyline Boulevard and Sloat Boulevard, beach access stairs are proposed (two are shown on Figure 8-1, an additional southern access point is in development) that will allow pedestrian access down to the beach level. Based on discussions with Coastal Commission staff, the third access point near the southern end of the project area will likely be added to the project.

The northern end of the new trail will terminate at Sloat Boulevard and connect to the existing Lower Great Highway park trail.

Trail Width, Striping and Vertical Clearance

Widely accepted multi-use trail guidance defines appropriate trail widths for two-directional shared pedestrian and bicycle use at a minimum of 10'-wide (Federal Highway Administration, *Evaluation of Safety, Design, and Operation of Shared-Use Paths, Final Report,* 2006; AASHTO, *Guide for the Development of Bicycle Facilities, Fourth Edition,* 2012.)

The proposed width for the new trail is 12'-15' wide to allow for anticipated moderate to high level-of-service (LOS). During detailed design, a Level of Service study and estimate may be warranted to verify the assumptions in determination of the trail width.

Centerline striping for the new multi-use trail is not recommended. USDOT/Federal Highway Administration research studies have found negative effects on Level of Service performance from centerline striping on multi-use shared paths (Federal Highway Administration, *Evaluation of Safety, Design, and Operation of Shared-Use Paths, Final Report,* 2006).

Due to design for bicycle use, the new path should include vertical clearance of a minimum of 8' with 10' clearance preferred. Any new tree or large shrub vegetation planting along the new trial corridor should be placed with consideration of the trail clearance requirements.

Trail Surfacing, Slopes, and Drainage

The multi-use trail surfacing should meet firmness, stability, and slip-resistance criteria to ensure universal accessibility performance over time. Preferred surfacing is asphalt, over a compacted, stable base course. Due to the sandy native substrate, asphalt will provide the most cost effective, durable surfacing over the long-term. It is likely that the trail and access road will be aligned along the existing northbound lanes and no new surfacing may be needed.

The trail surface longitudinal grades should strive to conform to less than 3% slope, with limited sections up to 5%. The finished trail bed should be cross sloped for drainage at 1.5%-2%.

Drainage of surface runoff from the trail should be considered in the appropriate design of the trail corridor. With proper grading design, trail runoff can be directed and diffused into the adjacent shoulders and restored landscape areas to avoid erosion. Shallow swales should be considered off the trail shoulder zone, to capture, direct, and infiltrate runoff.

Trail Lateral Clearance and Shoulders

Trail corridor clearance is important for user safety by providing space for avoiding collisions, running off the trail, or falling without risk of impacts from fixed objects.

Trail shoulders should be designed to be free of obstructions, and shoulder surfacing should be firm, stable, and meet the same cross slope requirements as the trail bed. Trail shoulders should also meet appropriate or required accessibility requirements.

Preferred lateral clearance for multi-use trails of any class is 1-meter to each side of the trail bed. This zone should be kept clear of any large obstructions such as boulders, vegetation, poles, etc. A 2-foot clear buffer should be provided between the outer edges of the trail and any post-mounted signage.

Trail Signage and Wayfinding [Under Development]

8.1.1. Concept Design – Beach Access and Amenities

The Concept Design includes five major components: Multi-Use Trail, Beach Access, Parking, Service Road, and Dune Restoration/Vegetated Slope Restoration, as depicted in Figure 8-1.

Multi-Use Trail & Beach Access

As described above in section 8.1, the Multi-Use coastal trail will provide pedestrian and bicycle access from Skyline Boulevard at Great Highway to the reconfigured terminus of Sloat Boulevard at Great Highway. A primary goal is to complete the connectivity gap that currently exists between the southern end of the Lower Great Highway Park Trail and Skyline Boulevard.

The Multi-Use Trail will be separated from the SFPUC Access Road by a vertical barrier to ensure trail user safety and to help discourage unauthorized public use of the Access Road.

Access Stairs

The proposed trail will also provide formal access points to the beach at two or three proposed new stairways. New access stairs will need to strike a balance between materials and a constructed aesthetic appropriate for the coastal trail setting on one hand, and structural integrity and durability to withstand wave action and the harsh marine climate, on the other. The Access Stairs are proposed to be constructed with pier/pile-supported sub-structure, and wood treads/risers and railings. See Figure 8-2.

Where the project right-of-way allows wider zones of restored dunes, beach access will be facilitated along boardwalk segments that will lead to new Access Stairs. In these zones, sand fences will be placed to mitigate sand migration to the upper dune and trail/road corridor.

Dune restoration through sand nourishment will occur following construction of the buried LMT protection wall. Through natural processes of wind, erosion and deposition, and to some degree wave action, the placed sand will reach an equilibrium morphology that will mimic many of the existing steep fore dunes along the beach. Figure 8-3 depicts a conceptual section showing the buried wall, restored dune and native plantings, and the trail/access road corridor.

Restroom

The managed retreat strategy along the southern portion of Ocean Beach includes removal of the existing parking lot and restroom at the terminus of Sloat Boulevard. These are amenities that have long served a large volume of surfers and beachgoers at the southern portion of the beach.

The planned reconfiguration of the Sloat Boulevard and Great Highway intersection will need to address the removal of the restroom. At this stage, two options are being considered - either a new restroom facility near the Westside Pump Station west of the trail, or rebuilding the existing restroom at the foot of Wawona St.

The new beach access point at the terminus of Sloat Boulevard should be primarily pedestrian-focused, serve as a gateway to the new multi-use coastal trail, and should be configured to provide a clear connection to the Lower Great Highway Park trail to the north, via a safe crossing of Sloat Boulevard.

The Restroom building should be of a high-quality durable construction owing to the heavy use pattern and harsh coastal exposure of the site. Given the user demand, the building should feature at least 3 plumbing fixtures for each gender, plus an All-Gender accommodation, or could alternatively utilize a 100% All-Gender design with an equivalent number of plumbing fixtures.

8.2. Parking

The existing parking locations have been closed due to the continual coastal erosion of the South Ocean Beach area between Sloat Blvd. and Skyline. The only accessible parking left is at the restroom area at the intersection of Sloat and Great Highway. The restroom area and adjacent parking will be demolished as part of the Wastewater Infrastructure Protection Project. New parking is needed to replace the parking lost due to erosion and construction.

A new parking area is proposed near the intersection of Great Highway and Skyline (see Figure 8-4). The parking lot will be accessible from Skyline Blvd in both the southbound and northbound directions. The existing intersection layout can be reused to access the parking lot to minimize reconfiguration of turning movements at the intersection. The parking lot will feature angled stalls to maximize the number of parking spaces and will provide direct coastal access and to the multi-use trail.

8.2.1. Concept Design – Parking at Skyline/Great Highway Intersection

Parking stalls are 9 feet wide and are angled at 30 degrees to fit the maximum number of stalls. 2 ADA compliant stalls that meet State requirements will be installed adjacent to the multiuse trail to reduce travel distance. Drive lane width will be approximately 18' wide to allow room to pass stationary cars waiting for parking stalls to clear.

8.3. Service Road

Existing roadway conditions are two lanes northbound and 2 lanes southbound. Northbound lanes allow access to the PUC treatment plant, pump station, and San Francisco Zoo. Coastal erosion has caused parts of the southbound lanes to narrow into one lane in certain areas.

The Ocean Beach Long Term Improvement Project will close access from Sloat Blvd. to Skyline Blvd. to the public. An SFPUC access road will be installed adjacent to the multi-use trail that will provide a connection from Skyline Blvd to Sloat Blvd. The access road will allow SFPUC to service their pump station, treatment plant, and appurtenant piping. The public access to the San Francisco Zoo from the northbound lane to the Great Highway will be removed.

8.3.1. Concept Design – Service Road

The SFPUC access road will be separated from the multi-use trail by a vertical barrier. The entrance to the access road will be from Skyline Blvd. from both the southbound and northbound directions. A gate will be installed just past the parking lot to only allow authorized vehicles from entering the access road.

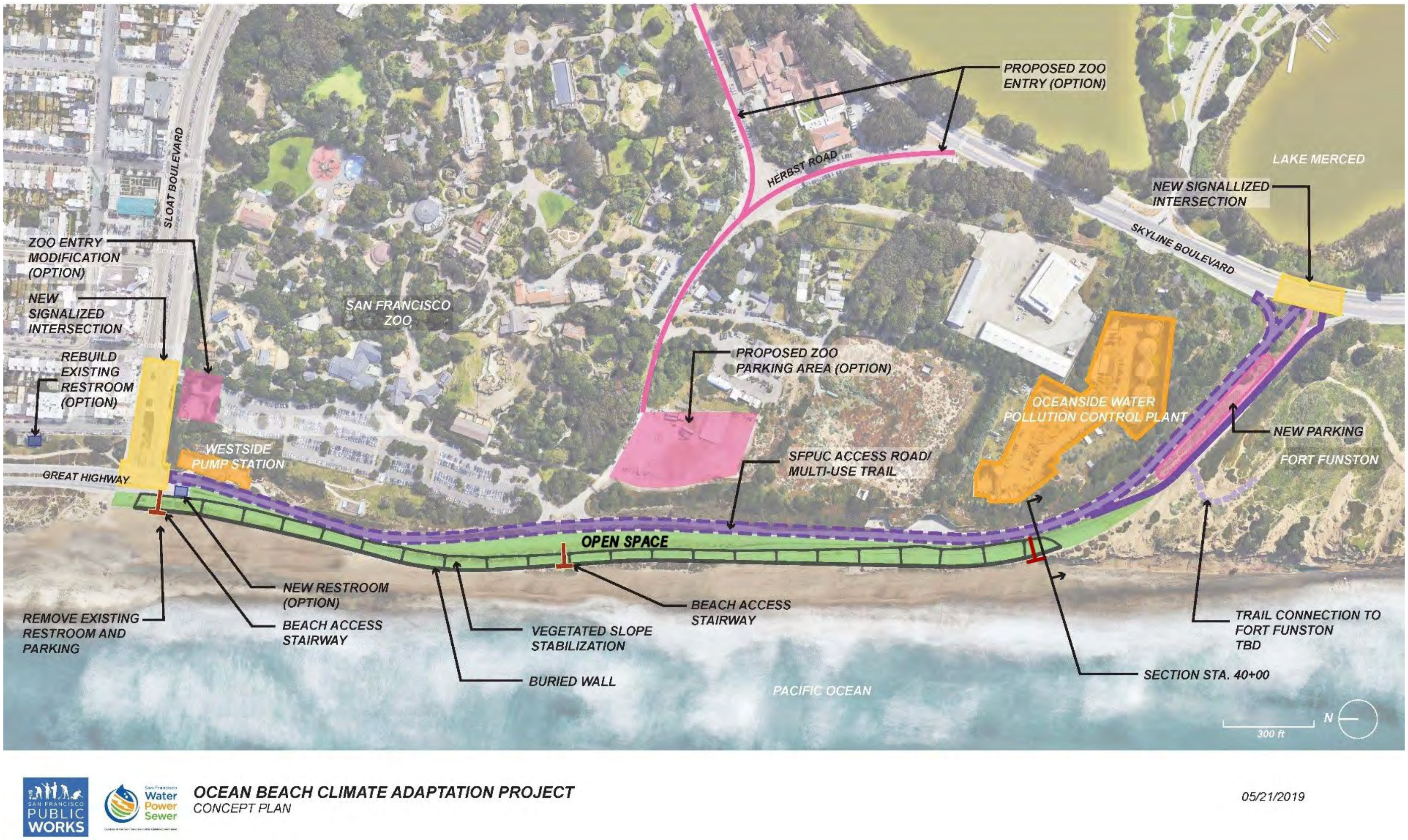


Figure 8-1: Overall Concept Plan

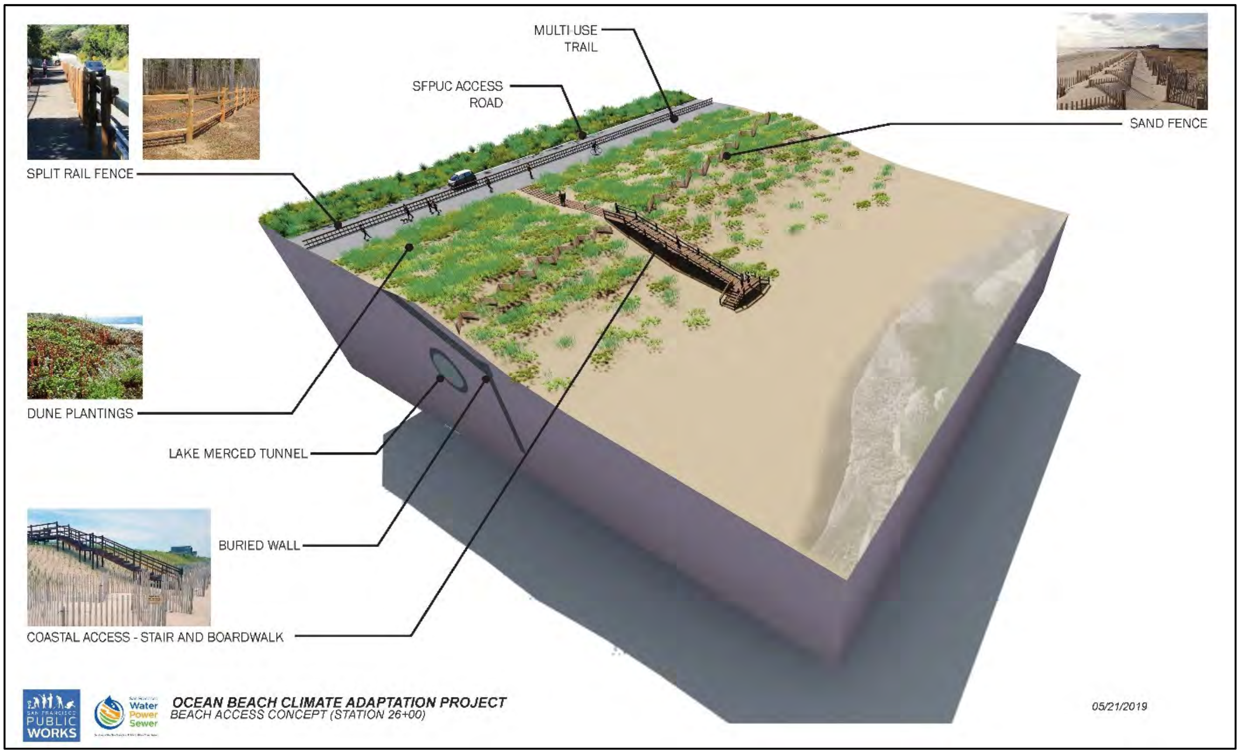


Figure 8-2: Beach Access Concept

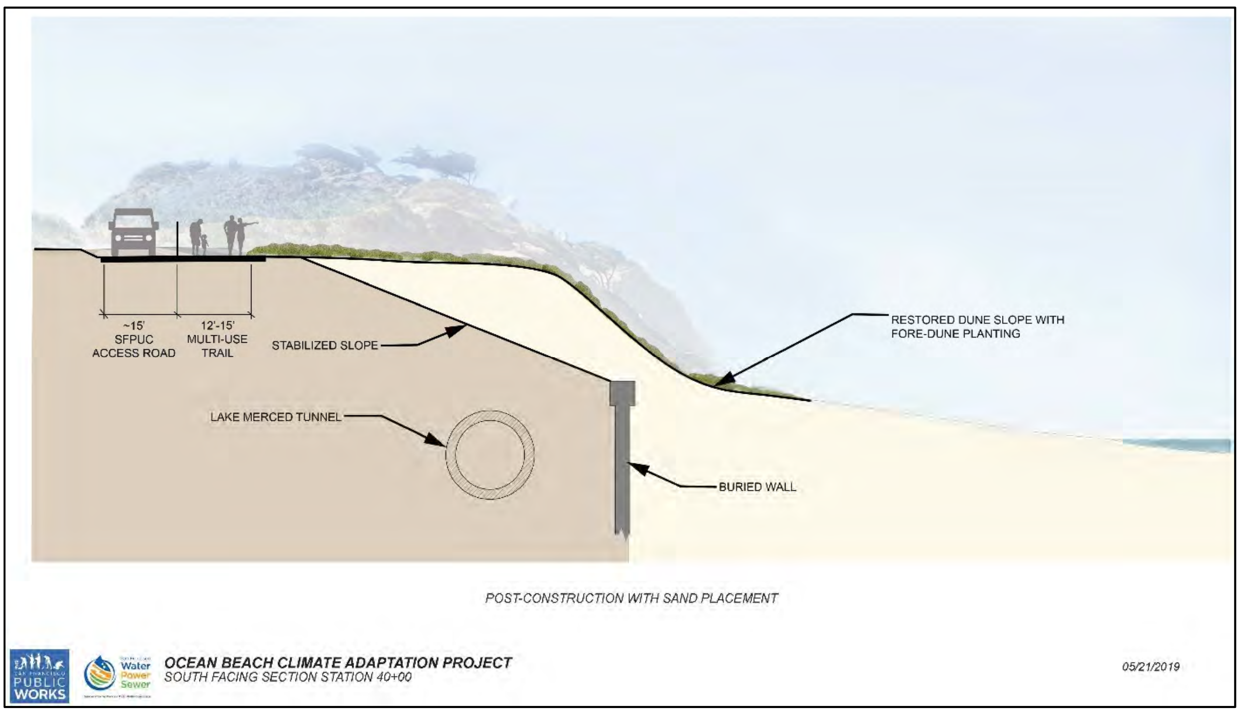


Figure 8-3: Concept Section

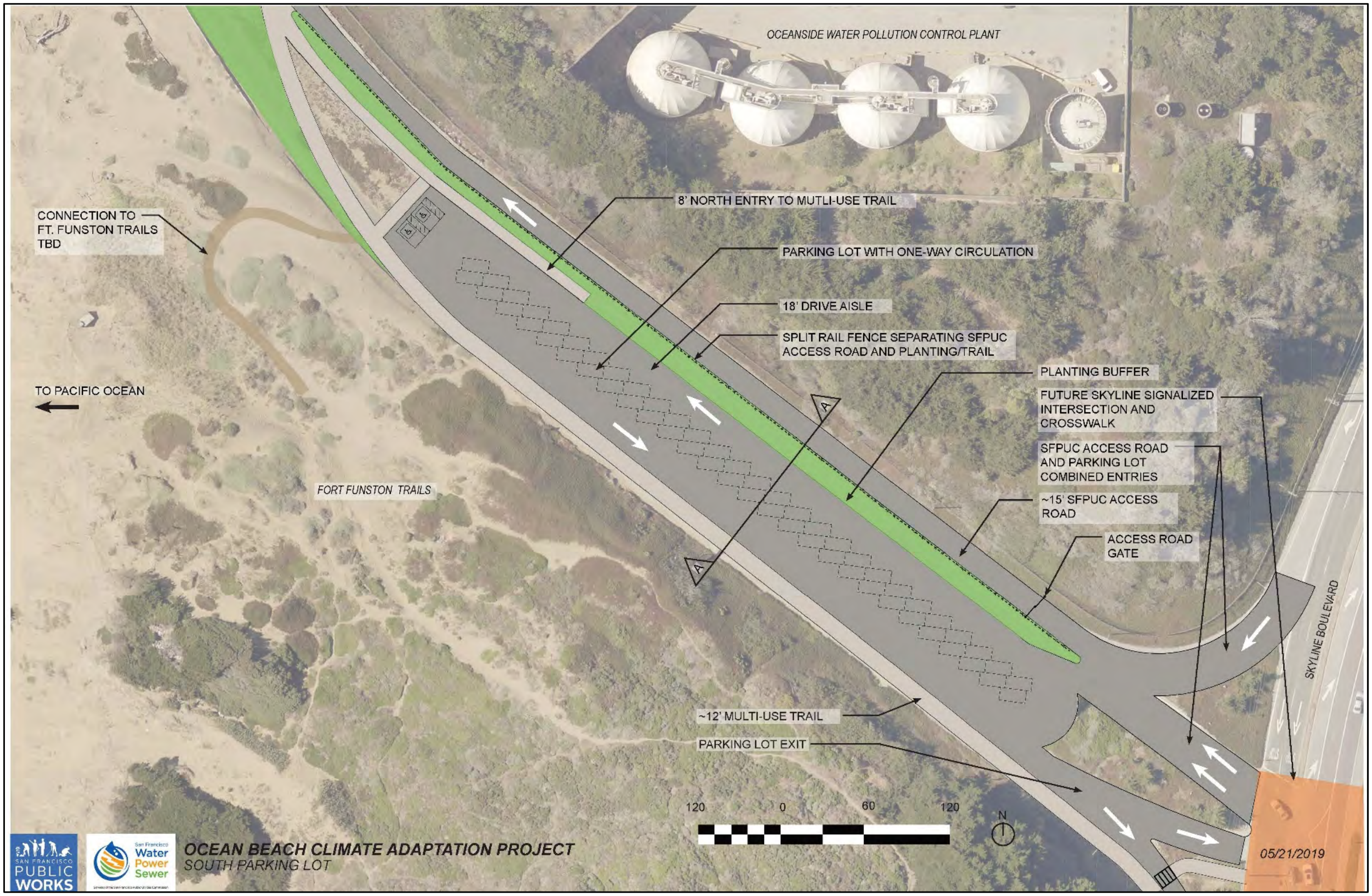


Figure 8-4: Parking Concept Layout

9. Structural

9.1. Background

The LMT is situated below the Great Highway between Sloat Blvd and Skyline Blvd at burial depths (above the LMT crown) of 20 feet to 50 feet. The low point invert is -6.53 (NAVD88) at project station 12+80. The LMT slopes up from the low point at a constant slope of 0.00132 ft / ft. The offset of the top of the bluff from LMT centerline varies from 25-ft to more than 100-ft. The unprotected bluffs undergo erosion during winter storms that result in loss of bluff and consequent protection for the LMT. This loss of bluff offset and overburden present a risk to the LMT. Loss of bluff, left unchecked, will eventually expose the LMT. Loss of overburden may allow the LMT to undergo buoyant lift due to seasonal high ground water. A protective low-profile wall west of the LMT will assure the LMT is not exposed due to bluff erosion and that adequate overburden is maintained over the LMT.

9.2. Wall Description

The selected concept for the low-profile wall is a secant pile system utilizing soil anchors (tiebacks) to reduce lateral displacements. Initially the wall will be buried only to be exposed when sufficient bluff loss due to erosion has occurred. The SFPUCs beach replenishment program will restore lost sand in front of the wall on a seasonal basis. One of the wall load conditions is when the sand in front of the wall has eroded down to an elevation of +2 feet (NAVD88).

The wall consists of 3-foot diameter unreinforced (primary) piles and reinforced (secondary) soldier piles. The toe elevation of the primary piles is approximately -10 feet (NAVD88). This primary pile toe depth is chosen so that the wall is never undermined (due to bluff erosion) but will permit groundwater flow from the backfield to the beach. The 3-foot diameter secondary piles overlap and are drilled into the edges of the primary piles and the wall module is 5’-0” considering the secondary pile overlap. The secondary piles have toe depths as required by analysis. Both primary and secondary pile tops are at soffit of the 5-foot wide by 4-foot deep continuous grade beam. The top of the grade beam (also the top of the wall) is nominally 6-feet above the crown of the LMT. Soil anchors at 10 foot spacing along the grade beam extend from the grade beam to below the LMT and provide lateral restraint to the top of the wall. The soil anchors significantly reduce wall displacements compared to a cantilever walls system. Typical wall plan and elevation are shown on Figure 9-1 and Figure 9-2.

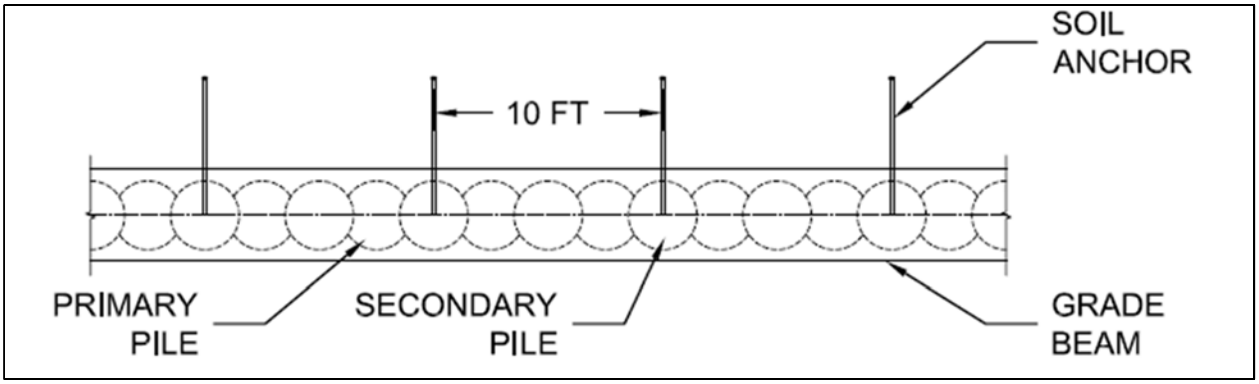


Figure 9-1: Typical Low-Profile Wall Plan.

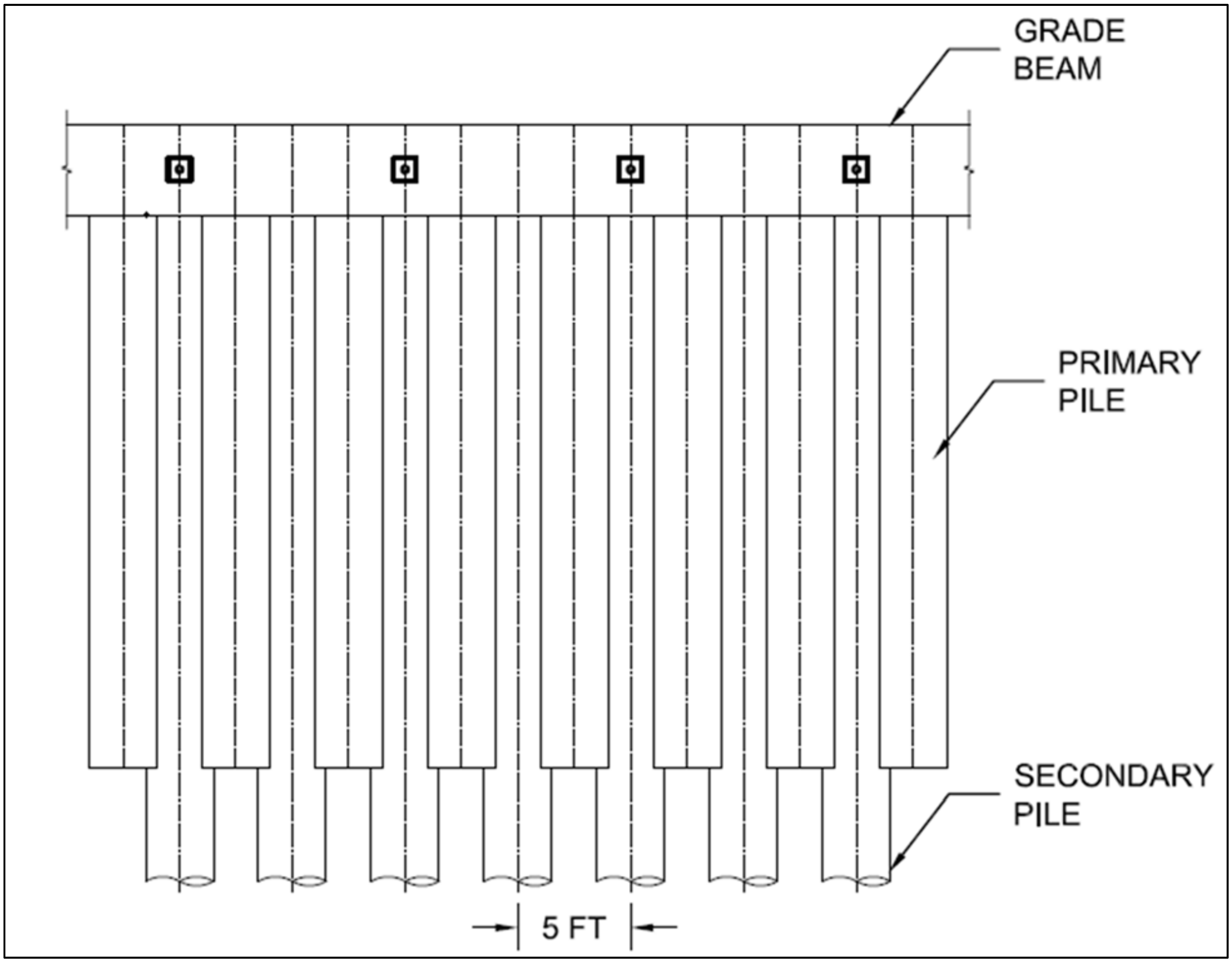


Figure 9-2: Typical Low-Profile Wall Elevation.

9.3. Wall Construction

The wall will be constructed by setting a drilling template a few feet below existing grade. The primary piles will be drilled to elevation of -10 ft (NAVD88) in a cased hole. Casing is required because of the potential for caving during hole drilling and pile construction. The piles up to soffit of grade beam will be filled with concrete. The pile hole above the soffit will be filled with sand or other material to be removed when the grade beam is constructed. Shortly after completion of the primary piles, the secondary piles will be drilled (in cased holes) to depth determined by analysis. Rebar cages and concrete will be placed up to the grade beam elevation. Above that, the holes will be filled with sand or fill material.

Backfield soil improvement will be done to strengthen the surficial soil following pile construction. This will allow reduced effort to excavate for the grade beam.

Following improvement of the backfield, a trench will be made to construct the grade beam. Secondary pile dowel reinforcement will be developed into the grade beam. After placement and curing of the grade beam, soil anchors will be drilled through the grade beam, grouted and stressed. The soil comprising the slope behind the wall (and over the LMT) will be improved as necessary to provide resistance to wave run-up over the top of the wall. Finally, the excavation will be backfilled and final grading of the slopes in front and behind the wall prepared as shown in the typical wall sections (Figure 6-9 through Figure 6-13) of section 6 in this report.

9.4. Wall Reaches and Representative Sections

Five representative sections are considered based on soil strata and properties, bluff offset from LMT and soil overburden. The LMT slopes up from Sloat Blvd to the intersection with Skyline Blvd resulting in about 4-foot change in design wall height from start to end of the project. The reaches and location of the representative stations are described below.

Table 9-1: Reach Descriptions.

Name	Start STA	End STA	LMT Setback from Bluff (ft)	Depth of LMT Crown (Min/Max) (ft)	LMT Crown Elevation (Beginning / End) (NAVD88)	Representative Station
North Reach	10+00	19+65	40	20/20	9.47 / 10.31	16+00
EQR Reach	19+50	24+50	38	20/20	10.31 / 11.15	22+00
Rubble Reach	24+50	33+60	80	20/22	11.15 / 11.88	28+00
Bluff Reach	33+60	36+60	35	22/30	11.88 / 12.55	34+00
South Reach	36+60	42+75	28	30/50	12.55 / 13.33	40+00

9.5. Soils Properties for Structural Design

A geotechnical investigation was done for this project with the objective of characterizing the soils for analysis. The investigation includes a number of borings, CPTs and test pits. Soil properties and characterization are developed based on analyses and tests of soil samples. The present program of borings and CPT's and geotechnical analysis is complemented by previous geotechnical investigations at the site (see Section 5). Information summarized below is based on information presented in Section 5.

9.5.1. Soils Characterization

The drilled piles pass through several different soil layers. These include Artificial Fill, Dune Sand, Colma Formation and the Merced Formation. These soil layers are described in detail in Section 5. Soil properties for wall structural analysis are presented in Table 9-2.

Table 9-2: Soil Properties for Wall Analysis.

Soil Layer	Dry Unit Weight (PCF)	Friction Angle (Degrees)	Cohesion (PSF)
Artificial Fill	120	33	0
Dune Sand	120	34	0
Colma Formation	125	36	0
Merced Formation	125	27	300

The depth and thickness of the soil layers described above vary along the project. For the wall design the assumed depth from the surface to the top of each layer is shown in Table 9-3. The Artificial Fill is the topmost layer for all reaches.

Table 9-3: Soil Layer Depths for Wall Analysis.

Reach	Station	Depth to Top of Soil Layer (from Surface – ft)			
		Artificial Fill	Dune Sand	Colma Formation	Merced Formation
North Reach	16+00	Surface	15	23	78
EQR Reach	22+00	Surface	18	22	66
Rubble Reach	28+00	Surface	11	18	69
Bluff Reach	34+00	Surface	15	20	68
South Reach	40+00	Surface	30	35	78

9.5.2. Liquefaction

Two soil levels may undergo liquefaction during a seismic event. The upper level, consisting of loose to medium dense fill / dune sand located approximately 15 feet to 25 feet below ground surface and in thicknesses varying from 5 feet to 7 feet. The lower level are intermittent layers of medium dense sand within the Colma and Merced Formation. Potential liquification for the two layers is described in Section 5. Liquefaction settlement will exert downdrag forces on the piles and tiebacks.

Settlement of the upper layer is not expected to have significant impact to the wall as the upper layer is for the most part above the wall. The design of the piles and tiebacks will consider the anticipated liquefaction settlement. Piles will support downdrag forces and develop below the lower liquefaction level. Tie-backs will develop axial capacity below the lowest level of liquefaction.

9.5.3. Water Table

High ground water imposes an upward buoyant force on the LMT that must be resisted by soil overburden. The ground water also acts differentially on the wall and modifies active and passive soil pressures. The groundwater level may be as high as 16 to 19 ft NAVD88 moving from north to south along the project based on geotechnical recommendations. Generally, this water level is above the top of wall and the water table is taken as the top of wall for design.

The water table on the beach side of the wall is taken at the beach level in the eroded condition. This makes the water elevation at +2-ft for analysis in the beach eroded condition.

The water table for the beach side of the wall, seismic condition, is also taken as +2-ft.

9.6. Load Conditions and Design Load Conditions

Load conditions for design are based on independent loads that are combined into design loading conditions. Independent Load Conditions and Design Loading Conditions are described in the following sections.

9.6.1. Earth Pressure

The soil on the LMT (back) side of the wall exerts active soil pressure on the wall. The soldier pile system resists the applied forces through passive soil pressure at the front of the wall and the soil anchor restraint at the grade beam. Active and passive soil loads are as follows:

$Ka = ka*\gamma$ pcf (Active soil pressure)
$Kp = kp*\gamma$ pcf (Passive soil pressure)

γ – taken as 120 pcf or 125 pcf for dry soil above the water table.

γ – taken as 56 pcf or 61 pcf for soil below the water table.
Soil pressures are computed internally by the analysis programs (SupportIT & DeepEx)

Cohesive soils resist loads differently than granular soils. One of the soil layers (the Merced formation) exhibits both granular and cohesive behaviour. The analysis programs consider the resistance of the cohesive layer up to soils undrained shear strength during wall analysis.

9.6.2. Static Water Pressure

The standing water table exerts hydrostatic (triangular shaped) pressure on both sides of the wall in proportion to its height. For the wall design, an unbalanced condition is assumed based on water surface elevation described in paragraph 9.5.3.

9.6.3. Surcharge

Generally, walls with level backfill are designed for rectangular shaped surcharge loadings to represent traffic or construction loading behind the wall. Traffic lanes are well back from the wall and not expected to result in significant wall surcharge loading. The project incorporates sloping soil backfill profiles that develop increased wall loadings compared to level backfill. The added load (or decreased passive resistance) is reflected in the active and passive soil coefficients generated by the programs as part of the analysis. No externally applied traffic surcharge is included in the analysis.

9.6.4. Seismic

Seismic loading of the retained soil generates increased pressure on the back of the wall. The pressure is dependent on the flexibility of the wall system. The following represent seismic surcharge pressures based on wall flexibility.

- Non-yielding retaining (movement between 0.1%H and 0.2%H) – uniform rectangular with base of 32H where H is the height of the wall (taken from elevation -10-ft to top of wall).

- Rigid wall (movement less than 0.1%H) – straight triangular shape with base of 45H where H is height of the wall.

Seismic earth pressure is added to active pressure. The wall is considered non-yielding (movement between 0.1%H and 0.2%H).

9.6.5. Liquefaction

Liquefaction may result in loadings to the wall under seismic conditions. Refer to paragraph 5.3.6 for discussion on liquification-induced lateral earth pressure. Settlement loads due to liquefaction are considered separately in terms of downdrag on piles and loading to tieback anchors. Refer to paragraph 5.3.2.

9.6.6. Wave Forces

The face of the wall will be subjected to wave forces when the bluff erodes. The wave force is based on ASCE 7-16 and is shown on Figure 4-3. For the ASCE evaluation, the top of the pile cap is assumed to be at the wave runup elevation. The ASCE 7 approach is made up of two loading parts – a hydrostatic pressure and a dynamic wave pressure. The hydrostatic force is partially offset by the static water head on the back side of the wall (the loading distribution is triangular in both cases). The effects of wave forces are not expected to govern the wall design as these are offset by passive soil resisting forces on the back of the wall and are not analysed further at this time.

9.6.7. Design Loading Condition 1 – Static Condition

Design Loading Condition 1 (DLC-1) represents the condition where the bluff has eroded to the wall and the beach elevation is at elevation +2-ft. The water table is at the top of the wall (LMT side) and soil profile sloped back at 3 horizontal to 1 vertical except at the South Reach where it is sloped back at 2 horizontal to 1 vertical. Soil active and passive pressures are based on soil properties described in Sections 9.5 and 9.6 Figure 9-3 shows DLC-1 graphically.

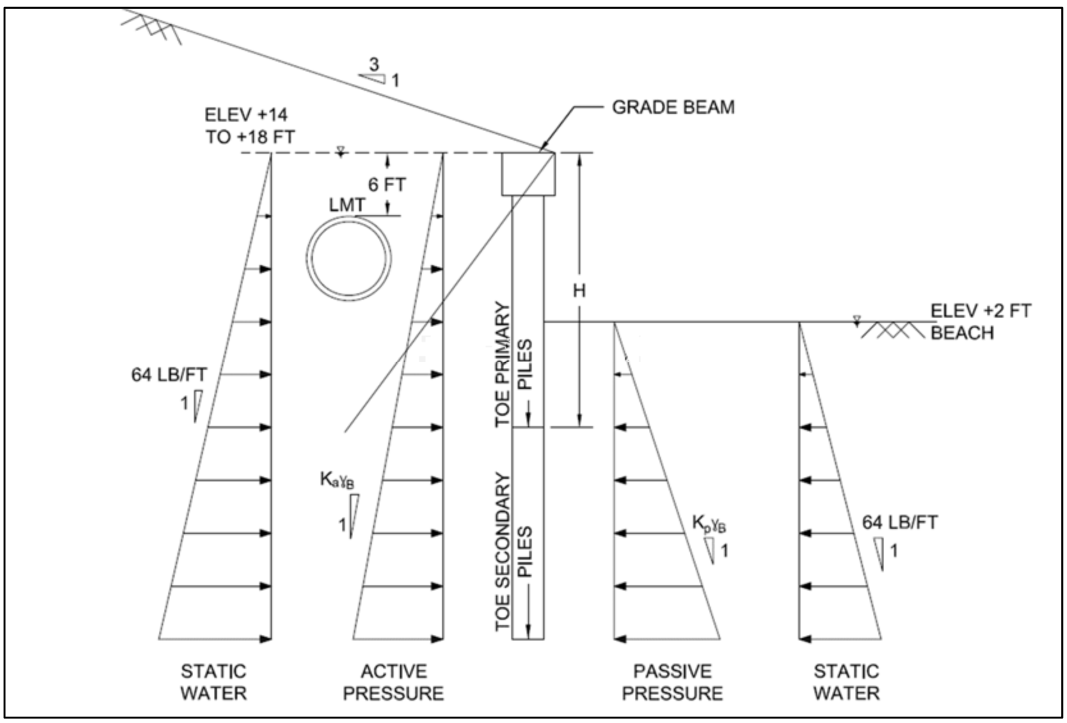


Figure 9-3: Design Loading Condition DLC-1.

9.6.8. Design Loading Condition 2 – Seismic Condition

Design Loading Condition 2 (DLC-2) represents the seismic loading condition where the wall just daylights into a stable beach slope condition. The hydrostatic water table is at the crown of the LMT on the back side of the wall and +2 ft NAVD88 on the front side of the wall. Seismic surcharge is applied to the back side of the wall to elevation -10 ft (the bottom of the primary piles). Figure 9-4 shows DLC-2 graphically.

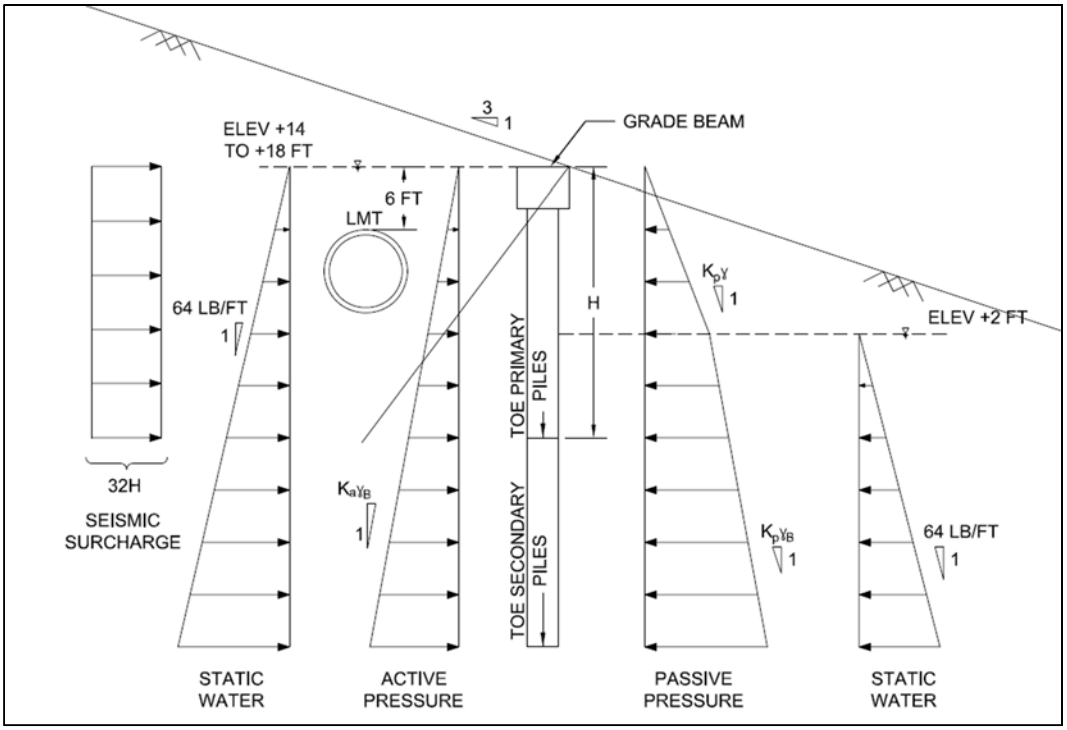


Figure 9-4: Design Loading Condition DLC-2.

9.7. Structural Analysis

Structural analysis of the wall for stability, required toe depth, pile forces, cap forces and soil anchor forces is determined from two dimensional models of sections representative of the five reaches described in Sections 9.4 and 9.5. Loadings are described in Section 9.6. The wall is analysed using software programs as described in the following paragraphs.

9.7.1. SupportIT

The initial analysis of the wall is done using the program 'SupportIT.' This is a general shoring analysis program. The program is capable of modeling sloping backfills, multiple soil layers, soil anchors or tiebacks and user input loads. The program solves for wall stability based on balancing applied loads, active soil pressure, passive soil pressure and tieback loads. The analysis assumes rigid system i.e., fixity at the anchor location. The program output is wall displacement and forces, soil anchor forces (per foot) and soil pressures. Toe depth is determined based on a zero displacement and then an additional length of embedment of 10 feet is added to provide a safety factor against secondary (soldier) pile toe movement. Due to the close spacing of soldier piles (a 2 foot gap between every 3 foot diameter pile) full arching is assumed at depth below the toe of the primary piles.

9.7.2. DeepEx

'DeepEx' is an advanced shoring analysis program. It has all the capabilities of SupportIT with some additional capabilities such as secant wall analysis and options for both rigid and flexible systems. The flexible system option is used to determine the lateral displacement at top of the wall. The subgrade modulus required for each layer of soil for the flexible system analysis was derived from API-RP2A (Table 9-4). The program outputs wall displacement and forces, soil anchor forces (per foot) and soil pressures.

Table 9-4: Subgrade Modulus – API RP 2A

Soil Layer	**ϕ (deg)**	**Above Water (lb/in^3)**	**Below Water (lb/in^3)**
Artificial Fill	33	95	60
Dune Sand	34	115	70
Colma Formation	36	160	95
Merced Formation	27	28	28

9.7.3. SAP 2000

SAP2000 is a general-purpose structural analysis program. SAP2000 is utilized here to investigate specific sections such as at the existing Southwest Ocean Outfall that requires special wall treatment.

9.7.4. Analysis Summary

The initial analysis to determine the pile size and toe depths of the wall was performed using SupportIT. Since the program assumes rigid anchor system resulting in large anchor forces, DeepEx was used to model flexible anchors. There is agreement between the two programs for the rigid anchor system. Table 9-5 summarizes the results of the analysis performed in DeepEx for DLC-1 & DLC-2. The use of flexible anchor system reduces the anchor forces and allows lateral displacement at top of wall. The lateral displacements show that the wall can still be considered as non-yielding for the seismic surcharge load in DLC-2 i.e., displacements between 0.1% H and 0.2% H.

Table 9-5: Analysis Summary

Description	Sta 16+00	Sta 22+00	Sta 28+00	Sta 34+00	Sta 40+00
Secondary Pile Length (ft)[1]	52	60	60	60	78
Elevation Top of Pilecap (ft)	14.9	15.7	16.5	17.2	18.0
Anchor Lock-off Load (kips)	50	50	50	70	70
Anchor Spacing (ft)	10	10	10	10	10
Total Anchor Force (kips)	54.8	57.7	60.5	79.2	82.8
0.1% H to 0.2% H (in) [2]	0.3 to 0.6	0.3 to 0.6	0.3 to 0.6	0.3 to 0.6	0.3 to 0.6
Lateral Displacement (in)	0.32	0.32	0.60	0.28	0.35

[1] Length based on depth to zero moment + 10 ft. Liquefaction and downdrag are not considered.
[2] H = Total height from top of pilecap to toe of primary pile (elevation -10 ft)

9.8. Structural Design

Structural design of the reinforced concrete piles and pilecap are based on CBC 2019, ACI 318-17 and ASCE 7-16. Material Properties are as follows:

Concrete: F'c = 5,000 psi
Reinforcing Steel: ASTM A615, Fy = 60,000 psi
Soil Anchors: Dywidag 150 ksi rod or 270 ksi bridge strand

Soil anchors are designed based on requirements described in the Post Tensioning Manual, PTI, 5th Edition.

The piles and pilecap are designed based on following LRFD load combinations as described in ASCE 7 as follows:

1. 1.4D
2. 1.2D + 1.6H
3. 1.2D +Ev + Eh +L +0.2S
4. 0.9D –Ev + Eh

Where:

D – Structure dead load
H – Load due to lateral earth pressure or ground water pressure
E – Earthquake load (horizontal or vertical)
S – Snow load (not applicable here)
L – Live load (not applicable here)

9.9. LMT Structural Evaluation

A preliminary analysis has been performed to estimate final lining distortion for Lake Merced Tunnel due to long-term coastal erosion, i.e. bluff retreat and loss of existing overburden, as part of Engineering Services for South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection Project. This analysis is based on the previous preliminary analyses and incorporates the updated parameters, layouts and configurations. This analysis summary includes the assessments for the following aspects:

- Cross sectional distortion
- Longitudinal differential displacement

Lake Merced Tunnel was constructed with a segmental liner referred to as ‘junk segments’ because their structural capacity is ignored in the design of the cast-in-place structural liner. Only the structural lining of the tunnel was used in the modeling studies. The structural input for the liner was derived from Abramson (1993). A 12-inch thick reinforced concrete tunnel liner was modeled in the studies.

Input for the geo-structural parameters was extracted from the information presented in section 5 and input from AGS, the team’s geotechnical consultant. Input for the future structural/support elements, including secant pile wall, tieback and soil stabilization, were based on the Moffatt and Nichol concept design and common practice assumptions.

9.9.1. Analysis Approach

The Lake Merced Tunnel distortion was evaluated based on numerical analyses performed using the two-dimensional Fast Lagrangian Analysis of Continua - FLAC (Itasca, 2011). FLAC is able to predict lining distortion as a result of surrounding ground erosion/deformation. In FLAC2D analyses, soil medium is simulated as a continuum and the tunnel lining, secant pile wall and soil stabilization are modeled using beam elements. In addition, tie-back is simulated using a cable element.

The soil profiles at Stations 16+00 and 40+00 were selected as critical sections for this analysis. Plan view of the tunnel and cross sections for the selected stations are shown in Figure 9-5 and Figure 9-6, respectively.

To analyze the distortion for the tunnel lining, nodal displacements of the lining elements were extracted. The convergence of the tunnel for each two opposing points is calculated using the displacement data. The distortion is calculated as the change in diameter, ΔD, divided by the tunnel radius, D. Based on the common practice, the maximum allowable distortion is considered to be 1%.

In addition to cross sectional distortion, longitudinal deflection of tunnel was preliminarily evaluated. For this evaluation, the tunnel lining was assumed as a simply supported beam subject to a uniformly distributed load. Based on this assumption, the induced bending moment as a result of differential displacement between STA 16+00 and STA 40+00 sections was estimated. The fiber stress resulting from this bending moment was calculated and compared with tunnel lining strength.

9.9.2. Inputs to Analysis

Key inputs for the analysis include properties for soil layers, graded-sand, tunnel lining, secant pile wall, tie-back, and soil stabilization. Location and properties for soil layers are estimated based on the data provided in Section 5 and AGS recommendations.

Table 9-6 presents the soil layer elevations and descriptions as well as Groundwater elevations for each cross section. Elevations are based on NAVD 88 datum. In addition, Table 9-7 summarizes the properties of the soil layers and graded-sand. Graded-sand properties are assumed to be the same as Dune sand properties.

Table 9-6: Soil layers and Groundwater elevations.

Reach	STA	Layer	Layer Description	Top of Layer Elevation (NAVD 88) (feet)	Thickness (feet)	GW elevation (NAVD 88) (feet)
North	STA 16+00	Fill	Silty Gravelly Sand	+31	15	+16
		Dune Sand	Poorly Graded Sand	+16	8	
		Colma Formation	Poorly Graded Sand with Silt	+08	55	
		Merced Formation	Silty Sand and Sandy Silt	-47	>30	
South	STA 40+00	Fill	Silty Gravelly Sand	+45	33	+19
		Colma Formation	Poorly Graded Sand with Silt	+12	45	
		Merced Formation	Silty Sand and Sandy Silt	-33	>30	

Table 9-7: Soil and graded-sand properties.

	Unit weight	E	v	c	ø	K0
layer	pcf	(ksi)		(lb/ft2)		
Fill	120	1.5	0.27	0.0	33.0	0.46
Dune Sand	120	1.5	0.27	0.0	34.0	0.44
Colma	125	3.5	0.33	0.0	36.0	0.41
Merced	125	1.7	0.35	300.0	27.0	0.55
Graded-sand	125	3.5	0.33	0.0	36.0	NA

Table 9-8 presents the assumed properties for tunnel lining, secant pile wall and soil stabilization, and Table 9-9 shows tie-back parameters.

Table 9-8: Tunnel lining, Secant pile and soil stabilization parameters.

Structural element	E (Ksi)	v	UCS (psi)	Thickness (ft)	I (ft4)	Length (ft)
Tunnel lining	4028	0.2	5000	1	0.083333	NA
Secant pile wall	3605	0.2	4000	3	2.25	42
Soil stabilization (CLSM)	570	0.2	100	3	0	NA

Table 9-9: Tieback parameters.

									grout		Cable	
Structural element	Cable Diameter (inch)	Hole Diameter (inch)	Shear/bond strength (psi)	Rebar grade	Lock-off load (kips)	spacing (ft)	Unbonded Length (ft)	bonded Length (ft)	UCS (psi)	v	yield capacity (kips)	E (Ksi)
Tie-back	1.5	4	30	150	130	10	varies	30	2000	0.2	190	30000

9.9.3. Key Assumptions

The following key assumptions were made regarding the ground behavior:

- Soil layers are assumed to have an elasto-plastic behavior and are modelled by the Mohr-Coulomb failure criterion.
- Beach surface is assumed to be eroded down to elevation +2 for long-term erosion condition beyond the secant pile wall and soil stabilization.
- Ocean water level is assumed to be constant and at elevation +8.
- For long-term condition, a surcharge load equivalent to water height is applied where the ground surface will be under ocean water level, i.e. from secant pile wall to the end of the model (towards ocean).
- Beam elements are used to simulate secant pile wall, tunnel lining and soil stabilization. In addition, a cable element is used to simulate tie-back.
- Secant pile wall is simulated as a continuous wall in the out-of-plane direction. The length of piles is assumed conservatively to be 42 feet.
- Tie-back is assumed to be unbonded from borehole collar up to the furthest side of the tunnel. The bonded length of the tie-back is assumed to be 30 feet long.
- Tie-back rebar is assumed to be Grade 150 with a demand load of 130 kips.
- Tieback is designed to be installed at an angle of 53 degrees from the horizon with a minimum distance of 5 feet from tunnel outer diameter. The spacing between the tiebacks are assumed to be 10 feet.

- Ground displacement is set to zero before the construction stage initiates, so that the focus can be made on displacement variations as a result of construction stage and long-term erosion.
- Tunnel lining is assumed to be installed after 30% of relaxation (similar to the previous analysis) to simulate ground relaxation/deformation prior to tunnel support installation.
- No factor of safety is considered for the estimated tunnel distortion.
- Groundwater table is considered to be at the elevations indicated in Table 9-6 up to where the secant pile wall will be installed. Groundwater level is assumed to linearly reduce between the alignment of the secant pile wall and where the current ocean water level meets the shore.
- Tunnel effluent unit weight is assumed to be 65 pcf.
- No surcharges and external loads are assumed in this analysis.
- Only static loading is considered in this analysis.

9.9.4. FLAC2D Models and Procedure

Two models were generated for this analysis, one for STA 16+00 and another for STA 40+00. In total, five cases were evaluated. In addition to the main conditions assumed for both STA 16+00 and STA 40+00, the following cases were assessed for STA 40+00 as sensitivity analysis, since this station resulted in higher distortion values:

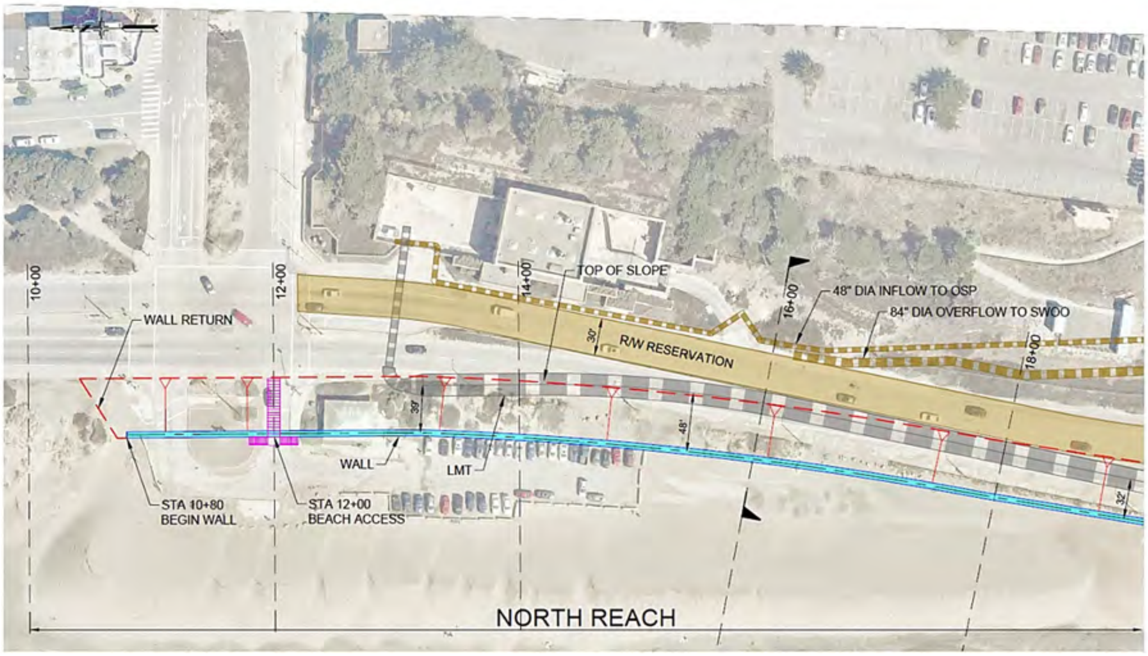
1. Gradual soil removal, as part of construction.
2. Assuming Groundwater elevation 2 feet above the AGS recommended elevation.
3. Assuming tunnel to be 50% full of effluent.

In all models, the tunnel was assumed to have an outside diameter of 16 feet. The general sequence of modeling is as follows:

1. Set up the initial soil geometry and apply initial stress and boundary conditions. Solve to equilibrium.
2. Excavate tunnel and solve to relax for 30% of support pressure.
3. Install tunnel lining and solve to equilibrium.
4. Reset ground displacements to zero to establish the baseline condition for “end of construction” and “long-term erosion” conditions.
5. Remove soil up to where the soil stabilization and secant pile wall will be installed.
6. Install soil stabilization, secant pile wall, and tie-back. Lower Groundwater level to ground surface elevation beyond secant wall. Solve to equilibrium.
7. Install graded-sand layer and solve to equilibrium. This stage is considered as “end of construction” condition.
8. Remove graded-sand layer and eliminate soil layer beyond the secant pile wall down to elevation +2.

9. Change water level to a flat line at elevation 8 beyond secant pile wall. Apply a surcharge load equivalent to water height above the eroded ground surface and solve to equilibrium. This stage is considered as “long-term erosion” condition.

Figure 9-7 and Figure 9-8 illustrate STA 16+00 and STA 40+00 model configurations, respectively, for initial, “end of construction” and “long-term erosion” conditions.



(a)

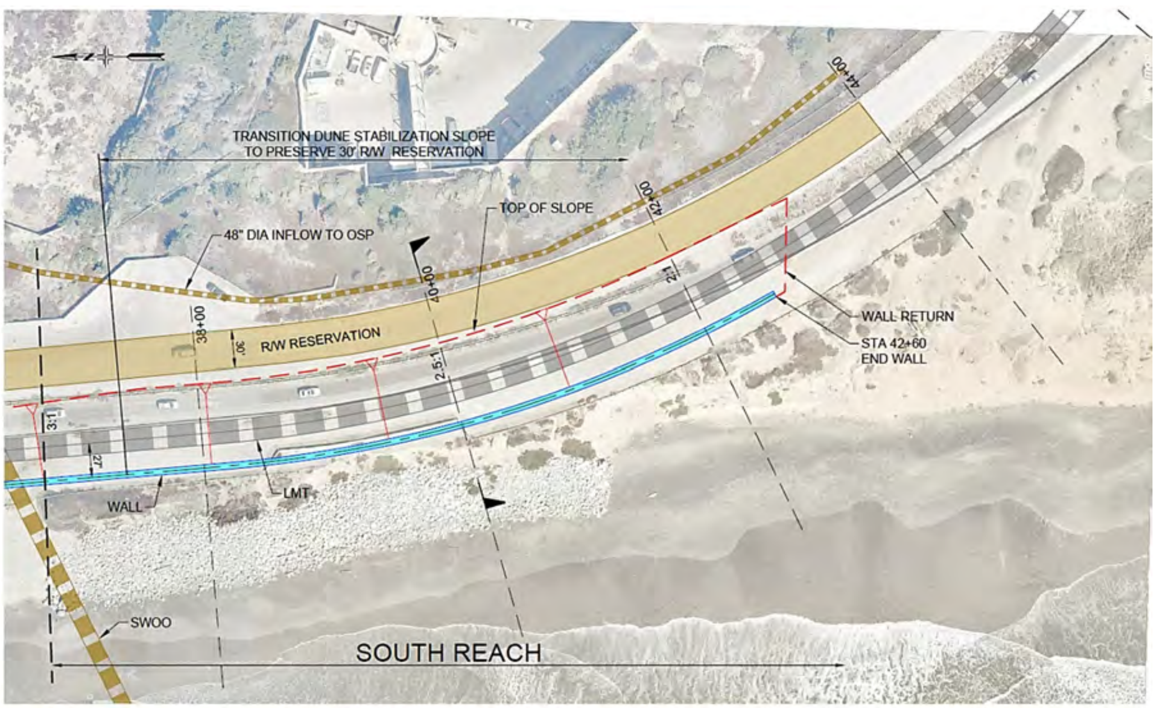
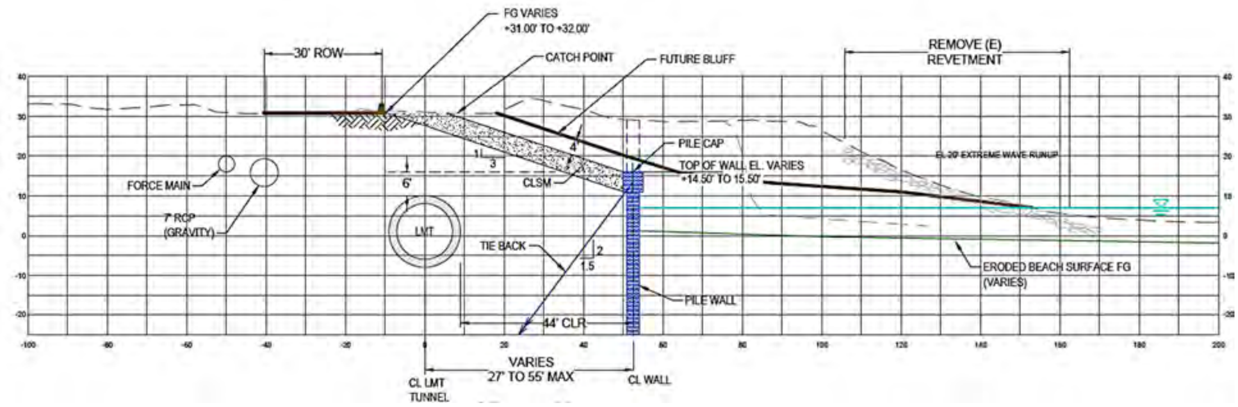
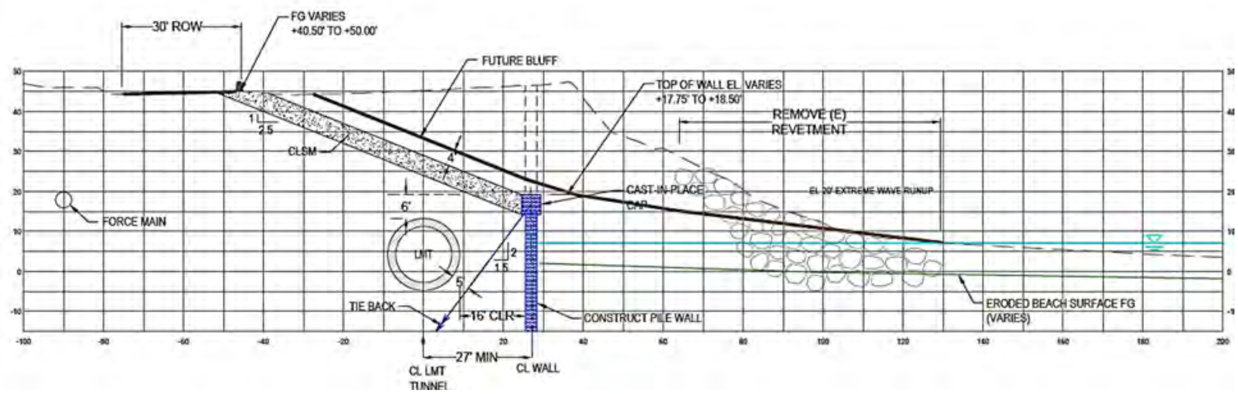


Figure 9-5: Tunnel plan view for (a) STA 16+00, (b) STA 40+00

(b)

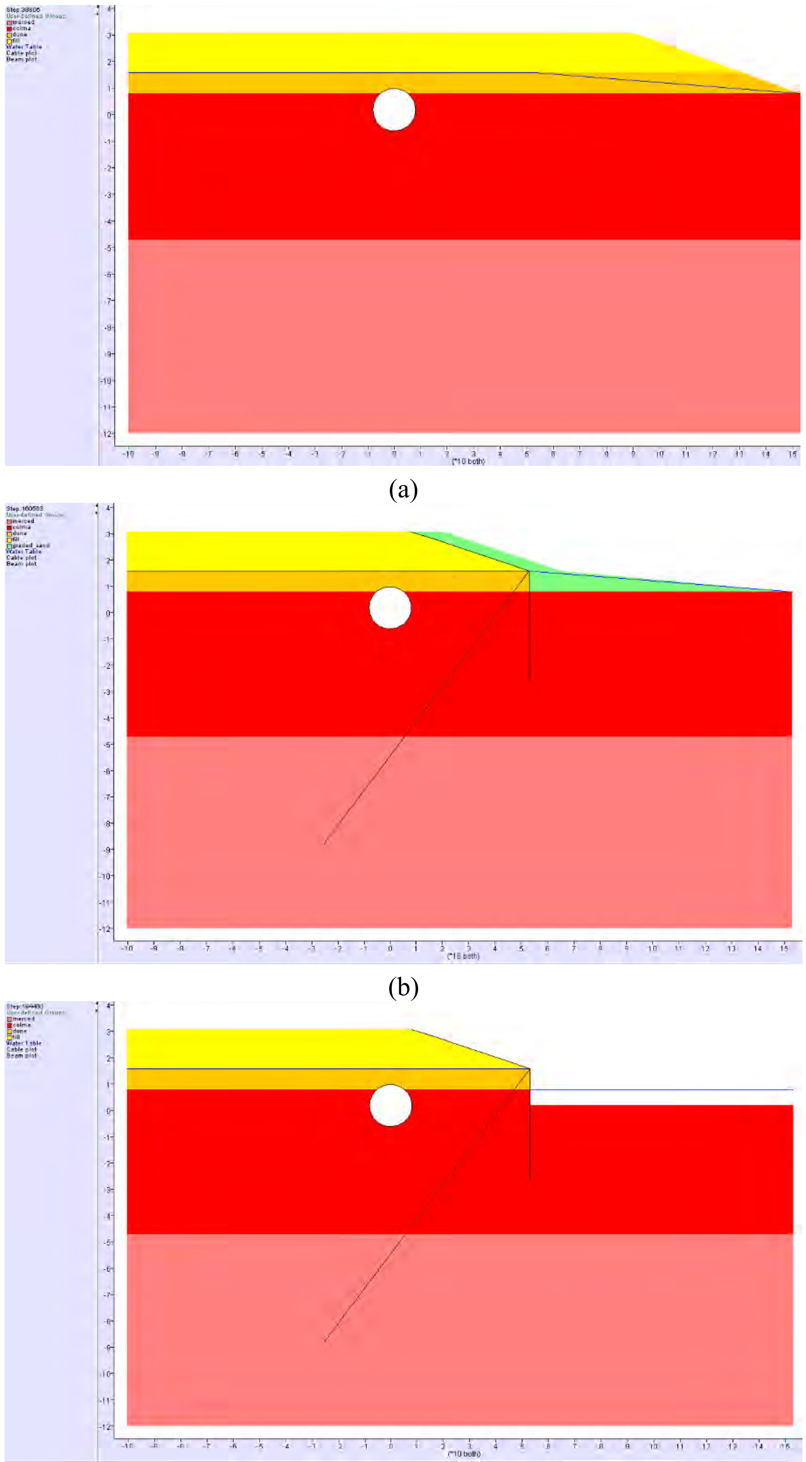


(a)



(b)

Figure 9-6: Cross sections for (a) STA 16+00, (b) STA 40+00.

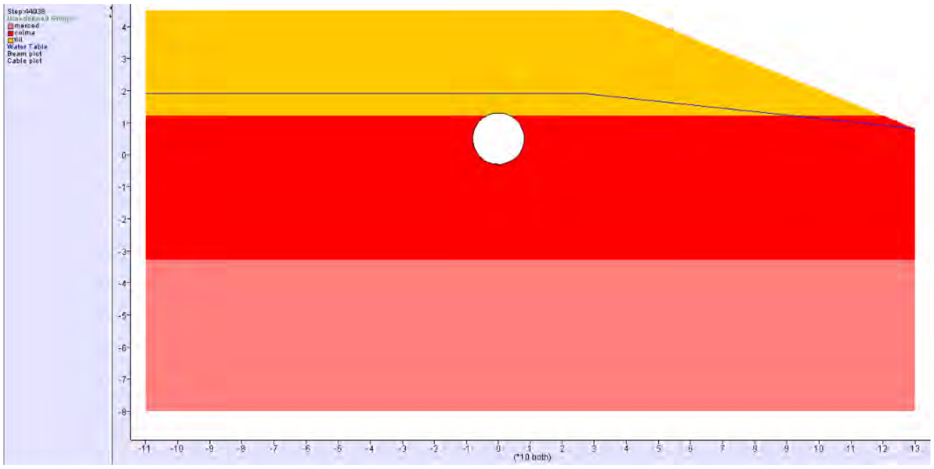


(a)

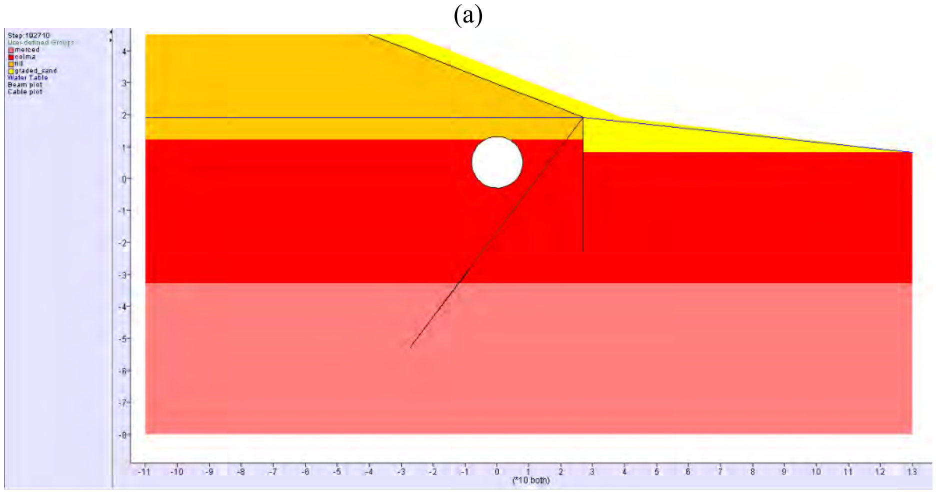
(b)

(c)

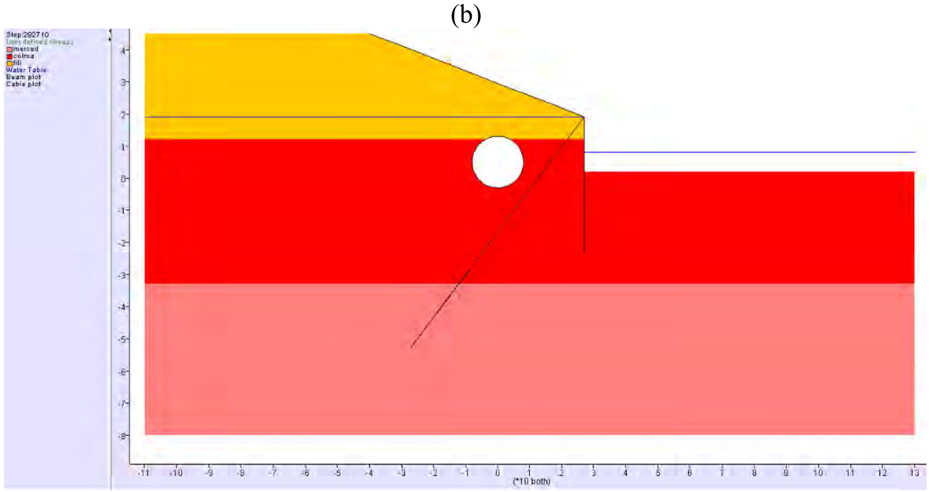
Figure 9-7: STA 16+00 model config. for (a) initial, (b) end of construction, and (c) long-term erosion conditions.



(a)



(b)



(c)

Figure 9-8: STA 40+00 model config. for (a) initial, (b) end of construction, and (c) long-term erosion conditions.

9.9.5. Results of Analyses

Cross Sectional Distortion

Table 9-10 and Table 9-11 summarize the average tunnel distortion for STA 16+00 and STA 40+00, respectively. Considering a maximum distortion limit of 1%, preliminary results are well below the limit and therefore the tunnel distortion criterion is unlikely to be exceeded under the assumed conditions.

Table 9-10: Summary of average tunnel distortion results for STA 16+00.

Tunnel condition	Distortion Results (with updated soil parameters)	Average Distortion (%)
Empty	End of construction	0.02
	After longterm (2050) erosion	0.04

Table 9-11: Summary of average tunnel distortion results for STA 40+00.

Tunnel condition	Distortion Results (with updated soil parameters)	Average Distortion (%)
Empty	End of construction	0.062
	After longterm (2050) erosion	0.092
	End of construction - Gradual soil removal	0.038
	After longterm (2050) erosion - Gradual soil removal	0.072
	End of construction - AGS GW+2 ft	0.066
	After longterm (2050) erosion - AGS GW+2 ft	0.094
50% Full	End of construction - Tunnel 50% full	0.060
	After longterm (2050) erosion - Tunnel 50% full	0.091

Longitudinal Deflection and Tunnel Lining Stresses

The numerical analyses completed for this phase of the study suggest that the bending moments and fiber stresses induced by differential displacement will be below the allowable limits and the tunnel lining will not be affected adversely by long-term erosion considering the proposed protection plan. However, additional engineering analyses would be needed to evaluate the implications of seismic forces on the tunnel for the different conditions analyzed above. In addition, we recommend additional engineering analyses be completed to evaluate the impacts of longitudinal deflections on the tunnel.

10. Constructability

10.1. Introduction

The proposed Ocean Beach Climate Adaptation Project is feasible in terms of constructability. The project involves primarily the construction of approximately 3,200 linear feet of low-profile pile wall along the ocean side of the existing LMT Tunnel. The excavation necessary to complete the pile wall, particularly the pile cap and to provide access for installation of the pile wall tieback anchor, requires the closure and removal of the southbound lanes of the Great Highway. One of the northern lanes may also require closure to provide an area within the Great Highway to be used as staging areas for construction. There will be no disruption to the operation of the LMT Tunnel for construction of the Wall.

The project site is along a coastline that has a highly energetic wave climate; therefore, project phasing and sequencing will have to be carefully addressed. Additionally, geotechnical conditions that will be encountered during construction could vary from information gathered in the soil borings, at the north end of the project where Lake Merced exited to the Ocean, as well as the vicinity of the SWOO where substantial past construction activities would have affected ground conditions. For example, construction fill and sand dune formation could be thicker and deeper at the location of the historic lake Merced Channel compared to what the soil boring indicated. The construction of the pile wall will necessitate some adjustments to the pile wall construction, where the assumed foundation layer varies from what was anticipated by the borings.

10.2. Obstructions and Constraints

The pile wall will cross the existing Southwest Ocean Outfall (SWOO). The design of the pile wall will have to include details for this crossing. Adequate clearances for construction equipment and structural separation of the pile wall from the existing SWOO must be considered. The construction documents must be written to provide strict requirements for procedures and method to be followed by the Contractor at the crossing location.

The SWOO is an important facility and therefore should be protected and not disturbed by the construction activities for the Pile Wall.

The pile wall will also cross two abandoned pedestrian tunnels. Both tunnels are 10 ft tall x 8 ft wide based on the as-built drawings for the LMT tunnel. Since both pedestrian tunnels are ‘abandoned’, there is no need to protect them. The proposed wall can be constructed with the secant piles penetrating across the abandoned tunnels. Furthermore, these abandoned tunnels are approximately just 5 ft below existing grade (top elevation); they can be partially demolished for the portion that will interfere with the proposed wall.

10.3. Traffic Disturbance

The Great Highway from Sloat Boulevard to Skyline Boulevard will be closed permanently during construction and will not be re-opened. Therefore, detours will have to be configured and notices to the public will have to be made in advance for the closure of the Great Highway before construction. The rerouting of traffic shall be clearly explained and publicly acknowledged.

Refer to Section 7 “Traffic and Intersections” of this report for further traffic information.

10.4. Construction Activities and Sequencing

It is anticipated that construction of the pile wall will start at a point in the ‘Rubble Reach’ near the mid-length of the wall. The assumption to start the pile wall construction at the ‘Rubble Reach’ is based on the distance of the proposed pile wall from the bluff which is farther compared to the other reaches wherein the bluff is closer to the pile wall. Starting construction at the North Reach could interfere with construction activities for the work on the roadway intersection of the Great Highway and Sloat Boulevard. The construction of the secant piles can proceed in a variety of direction – with two crews and equipment going north and the other crew and equipment going south assuming the contractor will have two sets of crew and equipment.

The construction of the Secant Pile wall is a specialized type of construction wherein the equipment used by the Contractor are specific to the design. There are a few contractors who would be capable of constructing the secant wall according to the design. These contractors are very knowledgeable for the procedures and method to be followed in constructing the secant wall with tie-back anchors. There should be no problem in getting a qualified contractor for the project – and there could be Contractor value engineering benefits when the project goes to construction.

Two methods of construction of the Secant Pile Wall could possibly be employed for constructing the pile wall, and herein described as Alternative 1 and Alternative2.

For Alternative 1, the secant piles will be drilled from the existing grade. A shallow trench will be excavated for a guide template. The excavation to reach the bottom of the concrete pile cap will be done after the piles have been filled with concrete.

For Alternative 2, the excavation to reach the bottom of the concrete pile cap is done first. The excavated material shall be temporarily stored on the beach which will consist mostly of fill and dune sand. The holes for the secant pile wall will be drilled from the elevation of the pile cap bottom.

For either or both alternatives, a guide template will be installed before the start of the drilling operations used to define the location and alignment of the pile wall for attaining the structural design requirements for the wall. A steel casing may be required to keep the drilled hole from caving in. Bentonite slurry mix may be used in lieu of the steel casing to support the drill operation and the bentonite will be displaced with a Tremie concrete placement of the pile with the slurry recirculated and reused in adjacent drill operations

The guide template and / or the steel casing will assure the installation of the drilled piles to be within the alignment tolerance acceptable.

The drilled piles will be filled with cast-in-place concrete. The primary piles will not have reinforcing bars while the secondary piles will be reinforced with a fabricated rebar cage, including testing tube pipe lowered into the drilled secondary piles. The contractor will determine the length of a section of the pile cap wall they would cast at a time. A sample of the possible sequence of construction for the wall is as follows:

- Excavate trench along wall alignment and install guide template
- Drill for primary piles
- Place concrete in primary piles and allow to cure
- Drill for secondary piles
- Install rebar cage into secondary pile
- Place concrete in secondary piles and allow to cure
- Form, place reinforcing, and place concrete for pile cap and allow to cure
- Drill for tieback anchors
- Install and lock- off tieback anchors
- Place grout for tieback anchors block-outs at pile cap

- Remove existing rock, broken stone, and sand-bag shoreline protection in the bluff (along the length of the project)
- Place graded Dune Bluff layer
- Construct PUC access road and Coastal Trail
- Install landscaping, Street Furniture and Signage.

The Slope Stabilization on the upper slope behind the wall and above the LMT may be constructed at the same time drilling and construction of the secant pile wall occurs – The sequence for this portion of construction will be up to the Contractor's 'means and methods' and the type of material for use as Slope Stabilization.

Section of the pile cap can also be constructed after the excavation to the top of wall is completed and the pile cap constructed.

The excavated material which will be mostly sandy material (from fill, dune sand, and Colma Formation), can be re-used for Graded Dune Bluff layer. The exposed material will be a natural-looking dune bluff.

Rock Slope Protection will be excavated as necessary and used as necessary in field observations as needed or stockpiled at the South end of the Project for future use as may be needed.

Access from the Trail to the beach is provided by means of pile supported walkways and stairs.

The access road and trail will be paved with Asphalt concrete with guard rail separating the two Travel ways.

10.5. Construction Quantities

The principal element of the Project is the construction of the secant pile wall. Project construction would involve the following key work quantities:

- Excavation of 171,000 cubic yards of material for installing the low profile wall, pile cap, and tiebacks;
- Removal of 19,000 cubic yards of rock, rubble, and quarry rock from the beach;
- Placement of 2,400 cubic yards of concrete for the pile cap;
- Placement of 16,000 cubic yards of concrete for the secant piles;

- Installing 22,400 linear feet of tie back anchors;
- Placement of 18,000 cubic yards of slope stabilization
- Placement of 40,000 cubic yards of sand for dune construction by re-using sand from excavation

10.6. Construction Equipment

An estimate of the major equipment that would be used for the secant pile wall construction include, but not limited to:

- 4 Drill Rigs for secant pile wall
- 2 Slurry mix plants (mobile).
- 1 drill rig for tie back installation
- 1 Mobile concrete pump
- 2 Cranes
- 1 Backhoe
- 1 Excavator
- 2 Dozer
- 1 Front End Loader (5 to 8 CY)

For the roadway construction, the following equipment will be required:

- 1 Motor Grader
- 2 Compactors
- 1 Asphalt Paving machine
- 2 Small Backhoes
- 2 Water trucks

The construction of the Access Road and Coastal Trail within the Right-of-Way Reservation can be included in the construction of the project, or it can be a separate project after the completion of wall construction.

11. Operations and Maintenance

11.1. Introduction

After construction of the proposed project, operations and maintenance will be required for the following elements of the project, each of which is described in the following text.

1. Beach and dunes, which are an integral part of the erosion control and wastewater infrastructure protection features
2. Public access features, which consist of a restroom facility, fixtures, trash enclosures, trails, signs, and lighting
3. Service road and parking lots

11.2. Periodic Beach and Dune Nourishment

Beach maintenance is expected to consist primarily of periodic beach nourishment in front of the low-profile wall to mitigate the impact of erosion on the wall and beach access by the public. Beach nourishment serves to protect the Lake Merced Tunnel, the Great Highway, and coastal bluff from the effects of storms by building a beach, which acts as a buffer. The need for periodic beach nourishment has long been recognized by the City, which has imported and placed sand on South Ocean Beach over many years as summarized in a memo from Moffatt & Nichol to SFPUC (2013). The Ocean Beach Master Plan (SPUR, 2012) endorsed the practice and recommended that the city pursue best practices for beach nourishment, including placement by the Army Corps of Engineers (Key Move 2.3). The subject was further explored in the Coastal Protection Measure and Management Strategy for Ocean Beach (SPUR, 2015) and the Alternatives Analysis Report Appendix (SFPUC, 2018). The principle difference among the various studies was in the frequency and quantity of required nourishment events, which intervals varied from 1 year to 30 years, and annualized quantities varied from 25,000-100,000 cubic yards. Additional considerations regarding nourishment included the source of the beach material (which also governs its grain sizes) and its transport and placement.

Although beach nourishment is one of the most commonly performed activities seen on the coast, predicting its effectiveness is a significant undertaking because of uncertainties in the frequencies of storms and the subsequent effects after sand is transported away from the nourished reach.

The Coastal Engineering Section 4.0 presents a study conducted for this CER that sheds further light on the required frequency and quantity of beach nourishment based on the low-profile wall concept presented in this report. In this study, a high-level desktop analysis was performed to approximate the quantity and frequency of beach nourishment required for the project under RCP8.5 Medium – High Risk Aversion SLR projection (OPC, 2018). Typically, beach width was used as the indicator for beach nourishment. Factors that affect beach width may include beach nourishment (+) and shoreline erosion or recession (-). The positive sign indicates an increase while the negative sign indicates a decrease in beach width.

The planform evolution of the beach profile can be estimated using the Pelnard-Considère equation (Pelnard-Considère 1956; Rosati et al 2002). This equation describes the shoreline evolution in terms of a one-line diffusion model. The basic model equation is:

$$\frac{\partial y}{\partial t} = G\frac{\partial^2 y}{\partial x^2}$$

where y is the shoreline position at a distance x alongshore and G is the longshore diffusivity:

$$G = \frac{KH_b^{2.5}\sqrt{g/\gamma}}{8(s-1)(1-p)(h_c+B)}.$$

In this equation, K is a sediment transport coefficient associated with median grain size (i.e. 0.25 mm to 0.35 mm per Moffatt & Nichol 1995; Barnard and Hanes 2006); H_b is the breaker wave height; g is the acceleration due to gravity; γ is the ratio of water depth to breaker wave height, typically about 0.78; s is the sediment specific gravity; p is the sediment porosity about 0.4; h_c is the closure depth (i.e. -35' MLLW per Moffatt & Nichol 1995); and B is the beach berm crest elevation. Overall, this is a diffusion model – meaning that the tendency is for the beach planform to flatten out. If the wave energy is constant along the shoreline, the model predicts a final condition in which the shoreline can be described as a straight line.

In addition, a long-term historical shoreline erosion rate of 2 feet per year was estimated for the project area (USACE 1996; USGS 2006). This rate of shoreline erosion is coupled with the loss due to sea-level rise, in which the Bruun Rule was applied (detailed in Section 2.8.1).

Figure 11-1 presents beach width variations for a compound beach nourishment scenario assessed in this study. The scenario assumes 125,000 CY of sand are placed along the entire project area every 5 years before Year 2060. After Year 2060, additional 40,000 CY (e.g. a total of 165,000 CY) of sand are required every 5 years to keep pace with the adopted RCP8.5 SLR projection. The annualized quantity varies from an initial 25,000 cubic yard to 33,000 beyond mid-century. The frequency of every 5 years is somewhat arbitrary at this point as it depends largely on sea level and storm condition; perhaps a better indication of the need to undertake a beach nourishment event is the width of the dry (above Mean High Water) beach. The model assumes that beach nourishment is called for when the dry beach width becomes 50 ft or less. The model also assumes that the dry beach width is at least 80 ft. upon completion of the project and the start of the periodic maintenance requirement.

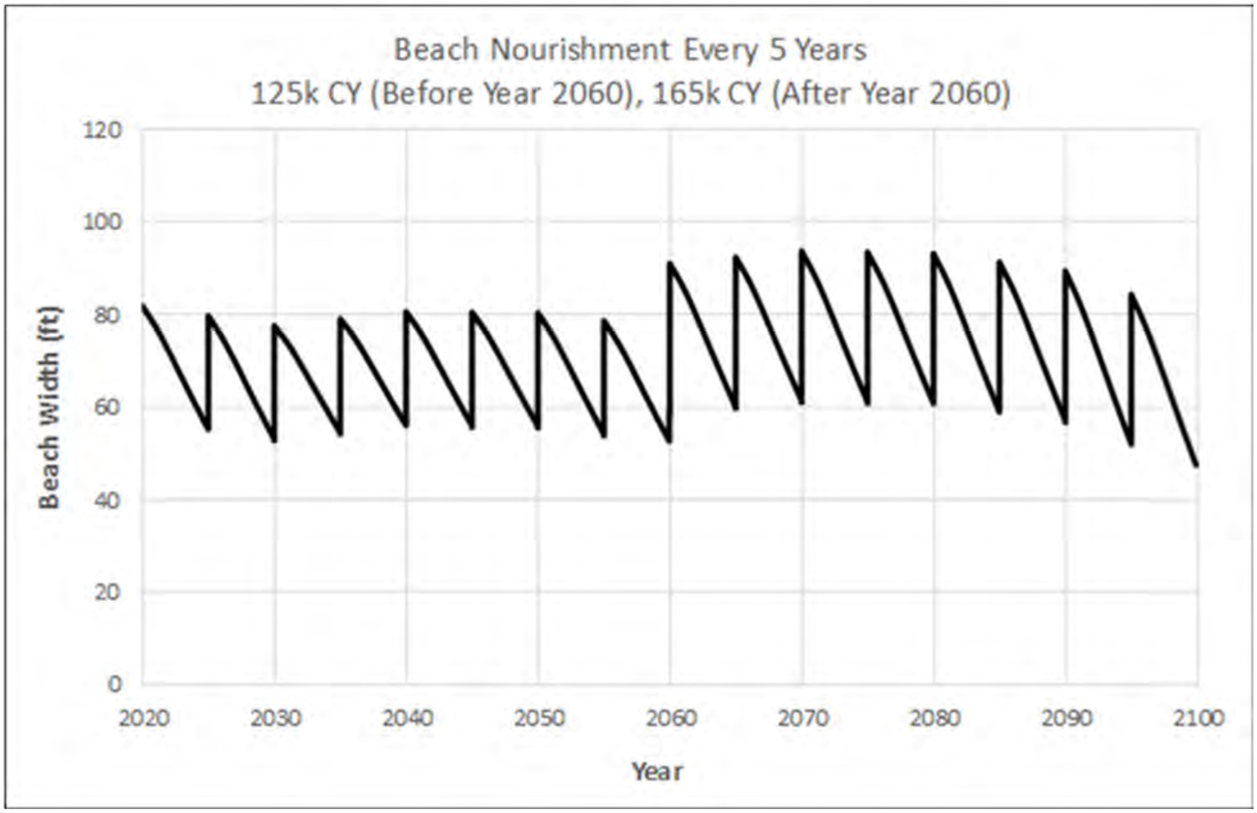


Figure 11-1: Beach Width Variation (RCP 8.5, Medium/High Risk Aversion Projection)

The dunes constructed as part of the project are a dynamic system that will grow when strong summer winds transport sand from the beach and erode in response to storm wave action during winter seasons. Monitoring and maintenance of the dunes, the sand fences, and the vegetation will be required on an annual basis.

Additional maintenance elements after extreme storm events may include repairs to the slope stabilization layer, which would consist of patching damaged areas if observed.

11.3. Public Access Features

The project envisions construction of several public access serving elements including a restroom, fixtures such as benches and signs, trash enclosures, an access trail, and lighting. Maintenance would consist of wind-blown sand management along the trail, similar to that conducted by Public Works along other areas of the Great Highway, and other activities similar to those that RPD provides at parks and open space areas within the City.

11.4. Service Road and Parking Lots

Maintenance of these features will consist of activities typical of other roadways and parking lots that are maintained by Public Works (periodic sealant, stormwater system management, striping, etc.)

12. Legal/Right-of-Way

The beach and bluffs along the project area are part of the National Park Service, Golden Gate National Recreation Area (GGNRA), while the lands east of that are part of the City and County of San Francisco (CCSF). A legal description of the property boundary is under development; in the interim, a draft of the property line provided by the City Surveyor is shown on Figure 12-1

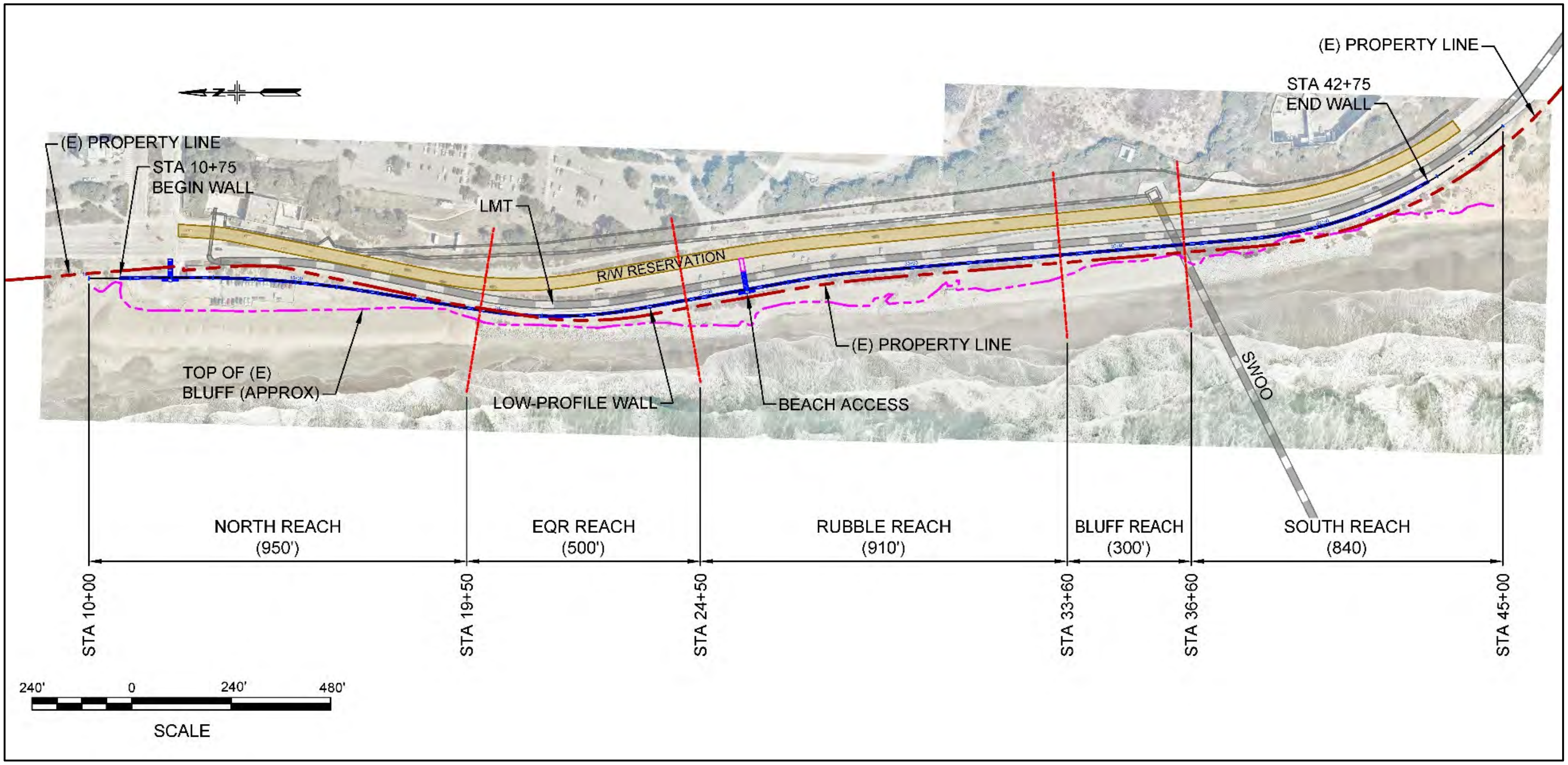


Figure 12-1: Interim Property Boundary Demarcation (GGNRA and CCSF)

13. Environmental Review

The San Francisco Public Utilities Commission (SFPUC) is responsible for managing critical infrastructure that the project has been designed to address and, therefore, the SFPUC is leading the project's design and environmental review processes. However, given the project also addresses facilities and lands under the control of other city and federal agencies, the project's implementation would be a collaborative, multi-agency initiative involving:

- SFPUC
- San Francisco Recreation and Parks (Rec and Parks)
- San Francisco Public Works (Public Works)
- San Francisco Municipal Transportation Agency (SFMTA)
- National Park Service (NPS) and
- Army Corps of Engineers (Corps)

The Corps project approval action will be for the large sand placement activities, both initially after project construction and on-going for beach nourishment. As a project partner and owner and manager of lands within the project area, NPS's project involvement would include a project approval action, such as issuing a Special Use Permit, as well as potential funding and management assistance for project elements. Accordingly, the Corps and NPS will be lead agencies for a separate federal environmental review process, including preparation of National Environmental Policy Act (NEPA) compliance documentation.

The core environmental review team consists of representatives from five main organizations:

- SFPUC, project Sponsor
- San Francisco Planning Department, Environmental Planning (EP) Division, CEQA Lead Agency
- National Park Service, NEPA Lead Agency
- Army Corps of Engineers, NEPA Lead Agency
- ESA+Orion, prime environmental consultant

The environmental team will conduct the environmental studies and prepare the environmental documentation required pursuant to the CEQA and NEPA including technical studies, permit applications, and the CEQA EIR and the NEPA EA or EIS (TBD). Separate CEQA and NEPA documentation will be prepared.

SFPUC Bureau of Environmental Management oversees the process for SFPUC projects for CEQA, NEPA and environmental permits. For more specific information, see Appendix C, CER CEQA Checklist.

CEQA Documentation

The environmental effects of the project from both construction and long-term operations and maintenance (including sand placement) will be analyzed in an Environmental Impact Report (EIR) pursuant to the California Environmental Quality Act (CEQA). The San Francisco Planning Department is the CEQA lead agency.

NEPA Documentation

Because there is federal agency involvement in the project (both the National Park Service and the Army Corps of Engineers) compliance with NEPA will be required.

Dredged sand placement (beach nourishment) would be conducted by the Army Corps of Engineers and placed on the beach within National Park Service jurisdiction. Depending on the timing of this work (i.e. either prior to or after the buried wall installation), this work would either be covered by separate CEQA and NEPA documents (see text below) or combined with the above referenced documents, to be determined in consultation with the Corps and NPS.

Environmental Permits

Permits anticipated for the project include:

- National Park Service Special Use Permit – for work within NPS jurisdiction, which includes work on the beach and bluff, but not work within Great Highway or intersections
- Army Corps of Engineers – Nationwide 404 permit for excavation work for rock revetment removal and for sand placement on the beach within Corps jurisdiction. The buried wall is not expected to be within the Corps' jurisdiction.
- US Fish and Wildlife Service – Section 7 permit for western snowy plover, salt marsh harvest mouse, San Francisco Garter Snake, spineflower.

- National Marine Fisheries Service - Section 7 permit for Steller Sea Lion or turtle species or abalone (more background information is needed on potential impacts associated with near shore species and USACE sand augmentation) and for NMFS designated critical habitat for the Leatherback Sea Turtles along the California Coast from Point Arena to Point Arguello.
- Marine Mammal Protection Act permit may be required for harassment to marine mammals including CA Sea Lion, Harbor porpoise, Gray Whale, Pacific Harbor Seal. Could require an incidental harassment authorization from NMFS in Washington DC or may be able to use existing Corps permit.
- California Department of Fish and Wildlife – 2081 permit for bank swallow for work along the bluff near the bank swallow colony.
- California Coastal Commission - Coastal Development Permit – current CCC permit requires long-term improvements complete application to be submitted no later than Dec 31, 2021.
- California State Lands Commission (through NPS) – TBD if needed for permanent installation of the buried sea wall
- State Historic Preservation Officer concurrence of no effect or coordination with lead federal agency.
- Regional Water Quality Control Board – Clean Water Act 401 and Waste Discharge Requirements for sand placement on the beach and recountouring of the bluff, also for storm drainage improvements
- State Water Resource Control Board – Clean Water Act 402 General Construction Permit for Stormwater.

14. Construction Duration and Schedule

The estimated duration for construction of this project.is about 44 months from Contractor Notice to Proceed. A detailed schedule is shown on the following pages.

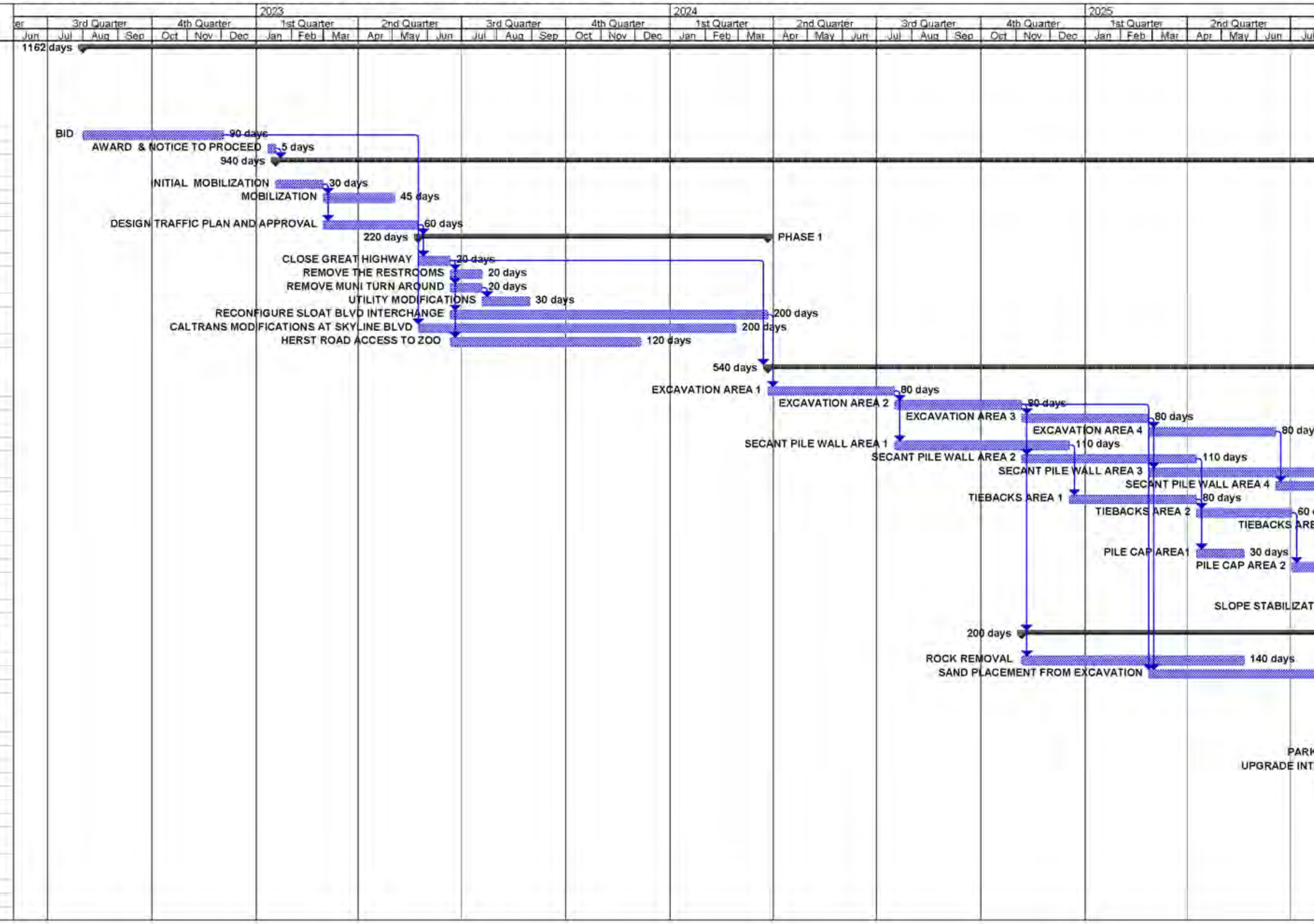
Construction could occur all year round, at least in different locations, and may only need to be suspended due to extreme weather conditions that may occur.

Construction of Sloat Blvd. can proceed independent of the Slurry wall construction and the tie-in to the Zoo access needs to be finalized to meet Zoo operations.

Construction of Skyline Blvd. intersection will require integration of SFPUC and CalTrans to agree on operations, and the construction can be independent of the Low-Profile Wall with temporary traffic management to provide access to the Zoo and for SFPUC service vehicles to the Westside Pump Station.

SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT

ID	Task Name	Duration	Start	Finish	Predecessors
1	**SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT**	**1162 days**	**Mon 8/1/22**	**Tue 1/12/27**	
2	BID	90 days	Mon 8/1/22	Fri 12/2/22	
3	AWARD & NOTICE TO PROCEED	5 days	Wed 1/11/23	Tue 1/17/23	
4	**CONSTRUCTION**	**940 days**	**Wed 1/18/23**	**Tue 8/25/26**	**3**
5	INITIAL MOBILIZATION	30 days	Wed 1/18/23	Tue 2/28/23	
6	MOBILIZATION	45 days	Wed 3/1/23	Tue 5/2/23	5
7					
8	DESIGN TRAFFIC PLAN AND APPROVAL	60 days	Wed 3/1/23	Tue 5/23/23	5
9	**PHASE 1**	**220 days**	**Wed 5/24/23**	**Tue 3/26/24**	**8**
10	CLOSE GREAT HIGHWAY	20 days	Wed 5/24/23	Tue 6/20/23	8
11	REMOVE THE RESTROOMS	20 days	Wed 6/21/23	Tue 7/18/23	10
12	REMOVE MUNI TURN AROUND	20 days	Wed 6/21/23	Tue 7/18/23	10
13	UTILITY MODIFICATIONS	30 days	Wed 7/19/23	Tue 8/29/23	12
14	RECONFIGURE SLOAT BLVD INTERCHANGE	200 days	Wed 6/21/23	Tue 3/26/24	10
15	CALTRANS MODIFICATIONS AT SKYLINE BLVD	200 days	Wed 5/24/23	Tue 2/27/24	2
16	HERST ROAD ACCESS TO ZOO	120 days	Wed 6/21/23	Tue 12/5/23	10
17					
18	**PHASE 2**	**540 days**	**Wed 3/27/24**	**Tue 4/21/26**	**10**
19	EXCAVATION AREA 1	80 days	Wed 3/27/24	Tue 7/16/24	14
20	EXCAVATION AREA 2	80 days	Wed 7/17/24	Tue 11/5/24	19
21	EXCAVATION AREA 3	80 days	Wed 11/6/24	Tue 2/25/25	20
22	EXCAVATION AREA 4	80 days	Wed 2/26/25	Tue 6/17/25	21
23	SECANT PILE WALL AREA 1	110 days	Wed 7/17/24	Tue 12/17/24	19
24	SECANT PILE WALL AREA 2	110 days	Wed 11/6/24	Tue 4/8/25	20
25	SECANT PILE WALL AREA 3	120 days	Wed 2/26/25	Tue 8/12/25	21
26	SECANT PILE WALL AREA 4	110 days	Wed 6/18/25	Tue 11/18/25	22
27	TIEBACKS AREA 1	80 days	Wed 12/18/24	Tue 4/8/25	23
28	TIEBACKS AREA 2	60 days	Wed 4/9/25	Tue 7/1/25	24
29	TIEBACKS AREA 3	60 days	Wed 8/13/25	Tue 11/4/25	25
30	TIEBACKS AREA 4	60 days	Wed 11/19/25	Tue 2/10/26	26
31	PILE CAP AREA1	30 days	Wed 4/9/25	Tue 5/20/25	27
32	PILE CAP AREA 2	30 days	Wed 7/2/25	Tue 8/12/25	28
33	PILE CAP AREA3	30 days	Wed 11/5/25	Tue 12/16/25	29
34	PILE CAP AREA 4	30 days	Wed 2/11/26	Tue 3/24/26	30
35	SLOPE STABILIZATION	180 days	Wed 8/13/25	Tue 4/21/26	32
36					
37	**PHASE 3**	**200 days**	**Wed 11/6/24**	**Tue 8/12/25**	**20**
38	ROCK REMOVAL	140 days	Wed 11/6/24	Tue 5/20/25	20
39	SAND PLACEMENT FROM EXCAVATION	120 days	Wed 2/26/25	Tue 8/12/25	20,21
40					
41	**PHASE 4**	**190 days**	**Wed 11/5/25**	**Tue 7/28/26**	**29**
42	TRAIL CONSTRUCTION	90 days	Wed 3/25/26	Tue 7/28/26	34
43	ACCESS ROAD CONSTRUCTION	90 days	Wed 3/25/26	Tue 7/28/26	34
44	PARKING LOT AT SKYLINE	150 days	Wed 11/5/25	Tue 6/2/26	29
45	UPGRADE INTERSECTION AT SKYLINE BLVD	150 days	Wed 12/17/25	Tue 7/14/26	33
46	PEDESTRIAN BEACH ACCESS	100 days	Wed 12/17/25	Tue 5/5/26	33
47	GENERAL PROJECT CLEANUP	60 days	Wed 4/22/26	Tue 7/14/26	35,39
48	PHASE 1 LANDSCAPING	90 days	Wed 12/17/25	Tue 4/21/26	33,39
49					
50	CLOSE OUT CONTRACT	90 days	Wed 4/22/26	Tue 8/25/26	35,39
51					
52	**ALTERNATE CONTRACT**	**100 days**	**Wed 8/26/26**	**Tue 1/12/27**	**50**
53	DUNES RESTORATION	100 days	Wed 8/26/26	Tue 1/12/27	50
54	BEACH REFURBISHING	100 days	Wed 8/26/26	Tue 1/12/27	50
55	LANDSCAPING	100 days	Wed 8/26/26	Tue 1/12/27	50



Project: PHASED CONSTRUCTION SCHEDULE A_190905 Date: Thu 9/5/19	Task	Rolled Up Task	External Tasks	Inactive Milestone	Manual Summary Rollup	Progress
	Split	Rolled Up Split	Project Summary	Inactive Summary	Manual Summary	Deadline
	Milestone	Rolled Up Milestone	External Milestone	Manual Task	Start-only	
	Summary	Rolled Up Progress	Inactive Task	Duration-only	Finish-only	

Page 1

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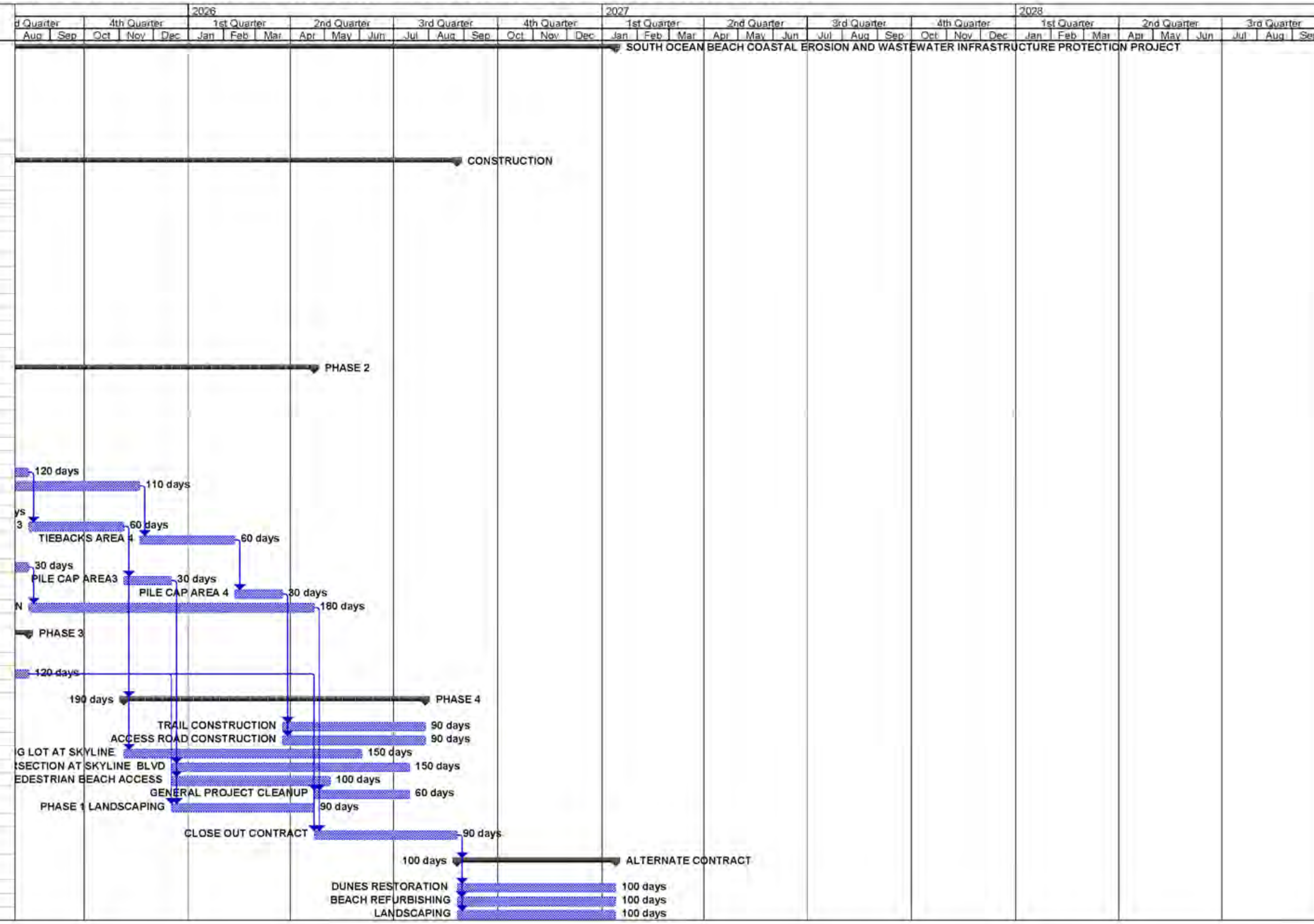
37	**PHASE 3**	**200 days**	**Wed 11/6/24**	**Tue 8/12/25**	**20**
38	ROCK REMOVAL	140 days	Wed 11/6/24	Tue 5/20/25	20
39	SAND PLACEMENT FROM EXCAVATION	120 days	Wed 2/26/25	Tue 8/12/25	20,21
40					
41	**PHASE 4**	**190 days**	**Wed 11/5/25**	**Tue 7/28/26**	**29**
42	TRAIL CONSTRUCTION	90 days	Wed 3/25/26	Tue 7/28/26	34
43	ACCESS ROAD CONSTRUCTION	90 days	Wed 3/25/26	Tue 7/28/26	34
44	PARKING LOT AT SKYLINE	150 days	Wed 11/5/25	Tue 6/2/26	29
45	UPGRADE INTERSECTION AT SKYLINE BLVD	150 days	Wed 12/17/25	Tue 7/14/26	33
46	PEDESTRIAN BEACH ACCESS	100 days	Wed 12/17/25	Tue 5/5/26	33
47	GENERAL PROJECT CLEANUP	60 days	Wed 4/22/26	Tue 7/14/26	35,39
48	PHASE 1 LANDSCAPING	90 days	Wed 12/17/25	Tue 4/21/26	33,39
49					
50	CLOSE OUT CONTRACT	90 days	Wed 4/22/26	Tue 8/25/26	35,39
51					
52	**ALTERNATE CONTRACT**	**100 days**	**Wed 8/26/26**	**Tue 1/12/27**	**50**
53	DUNES RESTORATION	100 days	Wed 8/26/26	Tue 1/12/27	50
54	BEACH REFURBISHING	100 days	Wed 8/26/26	Tue 1/12/27	50
55	LANDSCAPING	100 days	Wed 8/26/26	Tue 1/12/27	50



Project: PHASED CONSTRUCTION SCHEDULE A_190905
Date: Thu 9/5/19

Task	Rolled Up Task	External Tasks	Inactive Milestone	Manual Summary Rollup	Progress
Split	Rolled Up Split	Project Summary	Inactive Summary	Manual Summary	Deadline
Milestone	Rolled Up Milestone	External Milestone	Manual Task	Start-only	
Summary	Rolled Up Progress	Inactive Task	Duration-only	Finish-only	

Page 2

15. Cost Estimate

A construction cost estimate for the proposed project, broken out into three discrete work activities, is presented below.

NO.	DESCRIPTION	UNITS	QUANTITY	UNIT RATE	AMOUNT[1]
1. LMT PROTECTION (SFPUC)					
DIRECT COSTS					
1.1	**GENERAL**				
	MOBILIZATION	LS	1	$2,615,300	$2,615,300
	VALUE ENGINEERING	LS	1	$50,000	$50,000
	PARTNERING	LS	1	$30,000	$30,000
	PROJECT SIGNAGE	LS	3	$10,000	$30,000
	PERMITS (WATER, ETC)	LS	1	$50,000	$50,000
	CLOSE OFF GREAT HIGHWAY	LS	1	$30,000	$30,000
	SITE SURVEY	LS	1	$100,000	$100,000
	DEMOLITION				
	BATHROOMS, K-RAILS, FENCING, BARRIERS	LS	1	$200,000	$200,000
	OFFHAUL & DISPOSE	LS	1	$50,000	$50,000
	TEMPORARY FENCING	LF	1,000	$20	$20,000
1.2	**LOW PROFILE WALL & REVETMENT REMOVAL**				
	EXCAVATION	CY	152,000	$12	$1,824,000
	OFFHAUL & DISPOSE UNSUITABLE MATERIAL	CY	76,000	$25	$1,900,000
	EROSION PROTECTION	SY	35,000	$8	$280,000
	POLUTION CONTROL	LS	1	$100,000	$100,000
	PRIMARY PILES (25-FT)	LF	16,000	$300	$4,800,000
	SECONDARY STRUCTURE PILES (75-FT)	LF	44,800	$375	$16,800,000
	MONITORING & TESTING	EA	1,280	$500	$640,000
	PILE CAP FOR WALL	CY	2,400	$500	$1,200,000
	TIE BACKS	LF	22,400	$150	$3,360,000
	BRIDGE OVER SWOO PIPELINE	CY	60	$1,500	$90,000
	ABANDON OLD PEDESTRIAN TUNNELS	EA	2	$35,000	$70,000
	SLOPE STABILIZATION LAYER	CY	29,967	$300	$8,990,100
	CONSTRUCTION ACCESS RAMP TO BEACH	LS	1	$200,000	$200,000
	EXCAVATE ROCK AND OFFHAUL FROM BEACH	CY	20,000	$30	$600,000
	PLACE SUITABLE EXCAVATED SAND ON BEACH/BLUFF	CY	76,000	$10	$760,000
1.3	**MULTI-USE TRAIL AND SERVICE ROAD**				
	GRADING FOR SERVICE ROAD/TRAIL	CY	2,400	$10	$24,000
	FINAL GRADING	SY	800	$2	$1,600
	BASE	CY	1,200	$35	$42,000
	ASPHALT PAVING	TON	1,500	$45	$67,500
	FENCE BETWEEN SERVICE ROAD & TRAIL	LF	4,000	$50	$200,000
	GATES/CROSSINGS BETWEEN ROAD & TRAIL	EACH	4	$10,000	$40,000
	STRIPING	LF	4,116	$10	$41,160
	SUBTOTAL DIRECT COSTS				$45,205,660
INDIRECT COSTS					
	SALARY AND FIELD EXPENSES INDIRECT COSTS	%	39.58%		$17,891,496
	BONDS & INSURANCE	%	2.50%		$1,577,429
	DESIGN/CONSTRUCTION CONTINGENCY	%	30.00%		$19,402,376
	ESCALATION (3% per yr through 3/2025 = 5.5 yrs)[3]	%	27.07%		$22,763,055
	PROFIT	%	10.00%		$10,684,002
	SUBTOTAL INDIRECT COSTS				*$72,318,357*
	ADDITIONAL BUDGETARY ALLOWANCE ON TOTAL	%	10.00%		$11,752,402
	SUBTOTAL LMT PROTECTION (SFPUC)				**$129,276,419**

NO.	DESCRIPTION	UNITS	QUANTITY	UNIT RATE	AMOUNT[1]
2. INTERSECTION IMPROVEMENTS (SFMTA)					
DIRECT COSTS					
2.1	**SLOAT BLVD INTERSECTION**				
	REMOVAL OF EXISTING TRAFFIC SIGNALS AND LIGHTING.	LS	1	$50,000	$50,000
	REMOVAL OF PAVEMENT SECTION	SY	2,000	$100	$200,000
	RECONSTRUCT ROADWAY SECTION	SY	2,500	$350	$875,000
	STRIPING	LF	3,000	$10	$30,000
	MUNI LINE 25 REROUTE & BUSSTOP	LS	1	$250,000	$250,000
	ZOO ENTRANCE IMPROVEMENTS (Option 1)	LS	1	$300,000	$300,000
	NEW LIGHTING	EA	6	$20,000	$120,000
	TEMPORARY TRAFFIC CONTROL DURING	LS	1	$100,000	$100,000
	PG&E SERVICE AT SLOAT	LS	1	$45,000	$45,000
	PERMANENT TRAFFIC SIGNALS AT SLOAT	LS	1	$500,000	$500,000
	PLAZA/BIKE & PED TRAIL IMPROVEMENTS	LS	1	$250,000	$250,000
2.2	**SKYLINE BLVD INTERSECTION[2]**				
	TEMPORARY TRAFFIC CONTROL DURING	LS	1	$100,000	$100,000
	PERMANENT TRAFFIC SIGNALS AT SKYLINE (reprogram only	LS	1	$50,000	$50,000
	STRIPING & SIGNAGE (new Ped Xings)	LS	1	$125,000	$125,000
2.3	**WIDEN ZOO ROAD**	LF	200	$500	$100,000
	SUBTOTAL DIRECT COSTS				$3,095,000
INDIRECT COSTS					
	SALARY AND FIELD EXPENSES INDIRECT COSTS	%	39.58%		$1,224,939
	BONDS & INSURANCE	%	2.50%		$107,998
	DESIGN/CONSTRUCTION CONTINGENCY	%	30.00%		$1,328,381
	ESCALATION (3% per yr through 3/2025 = 5.5 yrs)[3]	%	27.07%		$1,558,470
	PROFIT	%	10.00%		$731,479
	SUBTOTAL INDIRECT COSTS				*$4,951,267*
	ADDITIONAL BUDGETARY ALLOWANCE ON TOTAL	%	10.00%		$804,627
	SUBTOTAL INTERSECTION IMPROVEMENTS (SFMTA)				**$8,850,894**

NO.	DESCRIPTION	UNITS	QUANTITY	UNIT RATE	AMOUNT[1]
3. PUBLIC ACCESS IMPROVEMENTS (RPD)					
DIRECT COSTS					
3.1	**IMPROVE PARKING LOT AT GREAT HWY & SKYLINE BLVD[2]**				
	REMOVAL OF PAVEMENT SECTION (for addl stalls)	SY	500	$200	$100,000
	RECONSTRUCT ROADWAY SECTION (addl stalls)	SY	500	$350	$175,000
	STRIPING	LF	3,000	$20	$60,000
	LIGHTING & SIGNAGE	EA	6	$20,000	$120,000
3.2	**ZOO ROAD ACCESS AND NEW PARKING**				
	ANIMAL ACCESS CONTROL GATES	EA	2	$100,000	$200,000
	PARKING LOT				
	GRADING & FILL	SF	8,600	$20	$172,000
	FINISH SLOPES	SY	3,000	$3	$9,000
	PARKING LOT SECTION	CY	8,600	$50	$430,000
	STRIPING	LF	13,500	$10	$135,000
	LIGHTING	EA	8	$20,000	$160,000
3.3	**PUBLIC ACCESS IMPROVEMENTS**				
	COMMERCIAL TOILET FACILITY	SF	2,000	$400	$800,000
	ACCESS WALKWAY & STAIRS (3 TOTAL)	SF	2,700	$250	$675,000
	PILE FOUNDATIONS FOR ACCESS	LF	3,000	$100	$300,000
	LIGHTING ALONG TRAIL	EACH	64	$5,000	$320,000
3.4	**DUNE RESTORATION**				
	DUNE PLANTINGS / SAND FENCING	SY	18,000	$50	$900,000
	MAINTENANCE DURING CONSTRUCTION	EA	1	$75,000	$75,000
	SUBTOTAL DIRECT COSTS				**$4,631,000**
INDIRECT COSTS					
	SALARY AND FIELD EXPENSES INDIRECT COSTS	%	39.58%		$1,832,857
	BONDS & INSURANCE	%	2.50%		$161,596
	DESIGN/CONSTRUCTION CONTINGENCY	%	30.00%		$1,987,636
	ESCALATION (3% per yr through 3/2025 = 5.5 yrs)[3]	%	27.07%		$2,331,914
	PROFIT	%	10.00%		$1,094,500
	SUBTOTAL INDIRECT COSTS				*$7,408,504*
	ADDITIONAL BUDGETARY ALLOWANCE ON TOTAL	%	10.00%		$1,203,950
	SUBTOTAL PUBLIC ACCESS IMPROVEMENTS (RPD)				**$13,243,454**

Notes
1 Costs are in 2019 dollars, based upon the CER Design (Moffatt & Nichol, Aug 2019)
2 Assumes that projects funded by FLAP Grant and Caltrans have been constructed
3 PUC rates: 6% per yr for first 2 yrs, 4% per yr for next 3.5 yrs

TOTAL PROJECT COSTS			
1	LMT PROTECTION (SFPUC)	$129,276,419	85.4%
2	INTERSECTION IMPROVEMENTS (SFMTA)	$8,850,894	5.8%
3	PUBLIC ACCESS IMPROVEMENTS (RPD)	$13,243,454	8.7%
	TOTAL	**$151,370,767**	**100%**

16. Specification List

DIVISION 00 – PROCUREMENT AND CONTRACTING REQUIREMENTS

INTRODUCTORY INFORMATION

00 01 02 ADVERTISEMENT FOR BIDS/INVITATION TO BID

00 01 03 KEY CONTACTS AND DETAILS

00 01 07 SEALS PAGE

00 01 10 TABLE OF CONTENTS

00 01 15 LIST OF DRAWING SHEETS

BIDDING REQUIREMENTS

00 21 13 INSTRUCTIONS TO BIDDERS

00 21 14 QUESTIONS ON BID DOCUMENTS

00 21 15 REQUEST FOR TRADE EXEMPTION FORM

00 21 16 RELEASE AND WAIVER

00 31 00 AVAILABLE PROJECT INFORMATION

00 40 13 BIDDING FORMS CHECKLIST

00 41 00 BID FORM

00 41 10 SCHEDULE OF BID PRICES

00 43 13 BID BOND

00 43 20 ACKNOWLEDGMENT OF RECEIPT OF ADDENDA

00 43 36 PROPOSED SUBCONTRACTORS FORM

00 43 37 PROPOSED SUBCONTRACTORS FORM FOR ALTERNATE WORK

00 45 13/PQ BIDDER’S QUALIFICATIONS

00 45 19 NON-COLLUSION AFFIDAVIT

00 45 57 CITYBUILD/FIRST SOURCE REFERRAL PROGRAM CERTIFICATION

00 45 57/PLA CERTIFICATION FORM FOR WSIP PLA LOCAL AREA APPRENTICESHIP AND EMPLOYMENT OPPORTUNITIES PROGRAM

00 45 58 INITIAL EMPLOYMENT PROJECTION

00 45 59 AGREEMENT TO HIRE APPRENTICES FROM PARTICIPATING REFERRALS AGENCIES

00 45 60 HIGHEST PREVAILING WAGE RATE CERTIFICATION

00 45 70 CERTIFICATE OF BIDDER REGARDING NONDISCRIMINATION IN CONTRACTS AND BENEFITS

00 45 71 S.F. ADMINISTRATIVE CODE CHAPTERS 12B & 12C DECLARATION: NONDISCRIMINATION IN CONTRACTS AND BENEFITS (CMD-12B-101)

00 45 72 CHAPTER 12B EQUAL BENEFITS DOCUMENTATION GUIDE

00 45 82 CERTIFICATION OF BIDDER REGARDING DEBARMENT AND SUSPENSION

00 45 85 BUSINESS TAX REGISTRATION DECLARATION

CONSTRUCTION CONTRACTS

CONTRACTING REQUIREMENTS

00 73 00/APB APPENDIX B: PERMITS AND AGREEMENTS TO BE OBTAINED BY THE CONTRACTOR

00 73 02 CONTRACT TIME, INCENTIVES, AND LIQUIDATED DAMAGES

00 73 10 DISPUTE RESOLUTION ADVISOR SPECIFICATION

00 73 10/A DISPUTE RESOLUTION ADVISOR THREE-PARTY AGREEMENT

00 73 12 DISPUTE REVIEW BOARD SPECIFICATION

00 73 12/A DISPUTE REVIEW BOARD THREE-PARTY AGREEMENT

00 73 16 INSURANCE REQUIREMENTS

00 73 17 PDF DOCUMENT LIABILITY WAIVER AND RELEASE

00 73 18 PROJECT LABOR AGREEMENT

00 73 19 HEALTH AND SAFETY REQUIREMENTS

00 73 20 EXISTING UTILITY FACILITIES

00 73 21 UTILITY CROSSINGS SPECIFICATIONS

00 73 25 ARCHAEOLOGICAL CONDITIONS

00 73 30 LOCAL HIRING REQUIREMENTS

00 73 31 CITYBUILD/FIRST SOURCE REFERRAL PROGRAM

00 73 63 SECURITY REQUIREMENTS - PHOTO ID/ACCESS CARD REQUEST FORM

00 73 73 STATUTORY AND OTHER REQUIREMENTS

00 73 73/APA APPENDIX A: NOISE CONTROL ORDINANCE

00 73 73/APB APPENDIX B: EXCAVATION, GENERAL REQUIREMENTS

00 73 73/APC APPENDIX C: INDUSTRIAL WASTE ORDINANCE #19-92

00 73 73/APD APPENDIX D: INDUSTRIAL WASTE DISCHARGE LIMITS INTO CITY'S SEWERAGE SYSTEM

00 73 73/APE APPENDIX E: GUIDELINES FOR PLACEMENT OF BARRICADES AT CONSTRUCTION SITE

00 73 83 FORMAL PARTNERING SPECIFICATION

00 73 83/A FORMAL PARTNERING FACILITATOR THREE-PARTY AGREEMENT

00 73 84 INFORMAL PARTNERING SPECIFICATION

DIVISION 01 – GENERAL REQUIREMENTS

01 11 00 SUMMARY OF WORK

01 14 00 WORK RESTRICTIONS

01 20 00 PRICE AND PAYMENT PROCEDURES

01 24 13 VALUE ENGINEERING

01 25 13 PRODUCT SUBSTITUTION PROCEDURES

01 29 73 SCHEDULE OF VALUES

01 31 00 PROJECT MANAGEMENT AND COORDINATION

01 31 19 PROJECT MEETINGS

01 32 16 CONSTRUCTION PROGRESS SCHEDULE

01 32 33 PHOTOGRAPHIC DOCUMENTATION
01 32 41 SURVEYING
01 33 00 SUBMITTAL PROCEDURES
01 35 43 ENVIRONMENTAL PROCEDURES
01 35 43.19 CONTAMINATED SOILS IN EXCAVATION
01 41 00 REGULATORY REQUIREMENTS
01 41 15 EXCAVATION PERMIT REQUIREMENTS (inside S.F.)
01 42 00 REFERENCES
01 45 00 QUALITY CONTROL
01 50 00 TEMPORARY FACILITIES AND CONTROLS
01 52 13 FIELD OFFICES
01 55 26 TRAFFIC CONTROL
01 56 55 TEMPORARY NOISE AND VIBRATION CONTROLS
01 57 23 STORMWATER POLLUTION PREVENTION, EROSION/SEDIMENT CONTROL
01 74 19 CONSTRUCTION WASTE MANAGEMENT AND DISPOSAL
01 77 00 CLOSEOUT PROCEDURES
01 78 36 WARRANTIES
01 78 39 PROJECT RECORD DOCUMENTS

DIVISION 02 – EXISTING CONDITIONS

02 21 13 SITE SURVEYS AND SITE PREPARATION
02 41 00 DEMOLITION AND DECONSTRUCTION

DIVISION 03 – CONCRETE

03 11 00 CONCRETE FORMING
03 20 00 CONCRETE REINFORCING
03 30 00 CAST-IN-PLACE CONCRETE
03 60 00 GROUTING

DIVISION 05 – METALS

05 12 00 STRUCTURAL STEEL
05 50 00 METAL FABRICATIONS

DIVISION 06 THRU 30 (NOT USED)

DIVISION 31 – EARTHWORK

31 00 00 EARTHWORK

31 11 00 CLEARING AND GRUBBING
31 32 00 SOIL STABILIZATION
31 62 13 CAST-IN-PLACE CONCRETE PILES
31 68 13 SOIL AND ROCK ANCHORS

DIVISION 32 – EXTERIOR IMPROVEMENTS
32 11 19 AGGREGATE BASE AND SUBBASE
32 12 16 ASPHALT PAVING

DIVISION 33 THRU 46 (NOT USED)

ACCESS ROAD AND COASTAL TRAIL

(Under Development)

LANDSCAPING AND SIGNAGE

(Under Development)

17. Drawing List

GENERAL

OBP-G0-00 COVER SHEET
OBP-G0-01 DRAWING LIST
OBP-G0-02 SITE PLAN AND VICINITY MAP
OBP-G0-03 GENERAL NOTES, ABBREVIATIONS, AND LEGEND
OBP-G1-00 SITE TOPOGRAPHIC SURVEY PLAN 1
OBP-G1-01 SITE TOPOGRAPHIC SURVEY PLAN 2
OBP-G1-02 SITE TOPOGRAPHIC SURVEY PLAN 3
OBP-G1-03 SITE TOPOGRAPHIC SURVEY PLAN 4

CIVIL

OBP-C0-00 GENERAL DEMOLITION PLAN
OBP-C0-01 DEMOLITION PLAN 1
OBP-C0-02 DEMOLITION PLAN 2
OBP-C0-03 DEMOLITION PLAN 3
OBP-C0-04 DEMOLITION PLAN 4
OBP-C0-05 DEMOLITION DETAILS 1
OBP-C0-06 DEMOLITION DETAILS 2
OBP-C1-00 GENERAL SITE PLAN
OBP-C1-01 PILE WALL PLAN AND PROFILE 1
OBP-C1-02 PILE WALL PLAN AND PROFILE 2
OBP-C1-03 PILE WALL PLAN AND PROFILE 3
OBP-C1-04 PILE WALL PLAN AND PROFILE 4
OBP-C1-05 SITE GRADING PLAN 1
OBP-C1-06 SITE GRADING PLAN 2
OBP-C1-07 SITE GRADING PLAN 3
OBP-C1-08 SITE GRADING PLAN 4
OBP-C1-09 SITE GRADING PLAN 5
OBP-C1-10 SITE GRADING PLAN 6
OBP-C2-01 PILE WALL TYPICAL SECTIONS 1
OBP-C2-02 PILE WALL TYPICAL SECTIONS 2
OBP-C3-01 PILE WALL SECTIONS 1 – STATION____ TO STATION____
OBP-C3-02 PILE WALL SECTIONS 2 – STATION ___TO STATION ____
OBP-C3-03 PILE WALL SECTIONS 3- STATION ____TO STATION ____
OBP-C3-04 PILE WALL SECTIONS 4 – STATION ____TO STATION ____
OBP-C3-05 PILE WALL SECTIONS 5 – STATION ____ TO STATION ____
OBPC3-06 PILE WALL SECTIONS 6 – STATION ____ TO STATION ____
OBP-C3-07 PILE WALL SECTIONS 7 – STATION ____ TO STATION ____
OBP-C3-08 PILE WALL SECTIONS 8 – STATION ____ TO STATION ____
OBP-C4-01 PILE WALL AT SOUTHWEST OCEAN OUTFALL (SWOO) DETAIL 1

OBP-C4-02 PILE WALL AT SOUTHWEST OCEAN OUTFALL (SWOO) DETAIL 2
OBP-C5-01 MISCELLANEOUS SECTIONS AND DETAILS 1
OBP-C5-02 MISCELLANEOUS SECTIONS AND DETAILS 2

UTILITIES

OBP-U1-01 UTILITY DETAILS 1
OBP-U1-02 UTILITY DETAILS 2
OBP-U1-03 UTILITY DETAILS 3
OBP-U1-04 UTILITY DETAILS 4
OBP-U1-05 UTILITY DETAILS 5

TRAFFIC AND INTERSECTIONS

OBP-T0-00 TRAFFIC CONTROL PLAN AND GENERAL NOTES
OBP-T0-01 TRAFFIC CONTROL DETAILS 1
OBP-T0-02 TRAFFIC COTROL DETAILS 2
OBP-T0-03 TRAFFIC CONTROL DETAILS 3
OBP-T1-01 STRIPING PLAN 1
OBP-T1-02 STRIPING PLAN 2
OBP-T1-03 STRIPING PLAN 3

STRUCTURAL

OBP-S0-01 GENERAL STRUCTURAL NOTES AND TYPICAL DETAILS
OBP-S1-01 TYPICAL PILE WALL PLAN AND ELEVATION
OBP-S1-02 TYPICAL PILE WALL SECTIONS 1
OBP-S1-03 TYPICAL PILE WALL SECTIONS 2
OBP-S1-04 TYPICAL PILE WALL SECTIONS 3
OBP-S1-05 TYPICAL PILE WALL SECTIONS 4
OBP-S2-01 PILE WALL DETAILS 1
OBP-S2-02 PILE WALL DETAILS 2
OBP-S2-03 PILE WALL DETAILS 3
OBP-S2-04 PILE WALL DETAILS 4
OBP-S2-05 PILE WALL TIEBACK ANCHOR DETAILS 1
OBP-S2-06 PILE WALL TIEBACK ANCHOR DETAILS 2
OBP-S2-07 PILE WALL TIEBACK ANCHOR DETAILS 3
OBP-S2-08 PILE WALL TIEBACK ANCHOR DETAILS 4
OBP-S3-01 PILE WALLNOTH END DDETAILS 1
OBP-S3-02 PILE WALL NORTH END DETAILS 2
OBP-S3-03 PILE WALL SOUTH END DETAILS 1
OBP-S3-04 PILE WALL SOUTH END DETAILS 2

OBP-S4-01 PILE WALL AT SWOO DETAILS 2
OBP-S4-02 PILE WALL AT SWOO DETAILS 2
OBP-S4-03 PILE WALL AT SWOO DETAILS 3
OBP-S4-04 PILE WALL AT SWOO DETAILS 4
OBP-S5-01 MISCELLANEOUS DETAILS 1
OBP-S5-02 MISCELLANEOUS DETAILS 2
OBP-S6-01 BEACH ACCESS STAIRWAY 1
SBR-S6-02 BEACH ACCESS STAIRWAY 2
SBR-S6-03 BEACH ACCESS STAIRWAY 3
SBR-S6-04 BEACH ACCESS STAIRWAY 4
SBR-S6-05 BEACH ACCESS STAIRWAY 5

ACCESS ROAD AND COASTAL TRAIL

(Under Development)

LANDSCAPING AND SIGNAGE

(Under Development)

REFERENCE DRAWINGS

TO BE DETERMINED

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Appendix A:
Coastal Engineering Analysis

- Coastal Engineering Analysis -

South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection Project

PREPARED FOR:

San Francisco
Water Power Sewer
Services of the San Francisco Public Utilities Commission

MAY 2019

PREPARED BY:

moffatt & nichol

Document Verification

Client	SFPUC
Project name	Engineering Services for South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection
Document title	South Ocean Beach Coastal Engineering Analysis
Document subtitle	
Status	Report
Date	May 6, 2019
Project number	10419
File reference	

Revision	Description	Issued by	Date	Checked
A	Progress Draft	M. Jorgensen	May 6, 2019	R. Dornhelm

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Table of Contents

List of Figures

List of Tables

Glossary

AAR	Alternatives Analysis Report
CCC	California Coastal Commission
CEQA	California Environmental Quality Act
CER	Conceptual Engineering Report
CSD	Combined Sewer Discharge
CY	Cubic Yards
ENSO	El Niño Southern Oscillation
EQR	Emergency Quarrystone Revetment
EVA	Extreme-Value Analysis
LMO	Lake Merced Overflow
LMT	Lake Merced Transport and Storage Tunnel
M&N	Moffatt & Nichol
MG	Million Gallons
MGD	Million Gallons per Day
MWL	Mean Water Level
NOAA	National Oceanographic and Atmospheric Administration
NTS	Not To Scale
OBMP	Ocean Beach Master Plan
ONI	Oceanic Niño Index
OSP	Oceanside Water Pollution Control Plant
PDO	Pacific Decadal Oscillation
PTI	Post-Tensioning Institute
SFPUC	San Francisco Public Utilities Commission
SPT	Standard Penetration Test
STA	Station
SWOO	Southwest Ocean Outfall
VLM	Vertical Land Motion
WPS	Westside Pump Station
WST	Westside Transport/Storage Box
WWE	{SFPUC} Wastewater Enterprise

Executive Summary

This appendix to the South Ocean Beach Conceptual Engineering Report (CER) summarizes the coastal engineering analysis conducted as part of the the conceptual engineering for the Wastewater Infrastructure Protection low-profile wall preferred alternative recommended in the Alternatives Analysis Report (AAR).

1. Introduction

The wastewater infrastructure within the South Ocean Beach area is threatened by chronic coastal erosion of the beach and bluffs, caused by wave action and episodic coastal storms. Infrastructure, such as the LMT, which is closest to the beach is in jeopardy of structural instability and eventual structural failure without engineered protection.

Over the years, federal, state, and local agencies have adopted erosion mitigation measures, aimed at protecting the existing shoreline and beach. These efforts have included depositing sand along the bluffs and/or off shore areas and the construction of engineered rock revetment (under emergency permit order).

Efforts in recent years have focused on the development of the Ocean Beach Master Plan (OBMP), which outlines coastal protection strategies along Ocean Beach until 2050. The OBMP recommends management and protection measures for the existing essential wastewater infrastructure at Ocean Beach (including the LMT) in conjunction with increasing local access to the beach, improving aesthetics, and improving the beach's ecological functions.

2. Coastal Evaluation

2.1. Historical Background

Beach and dune fill activities started as early as the 1870's when dune stabilization and road improvements affected the shoreline position and shape, *M&N (1995)*. Significant beach and dune fill occurred in the period from 1900 to 1929 when the O'Shaughnessy Seawall was constructed. Between the years 1900 and 1956, a total known volume of 2.35 million cubic yards (CY) of sand was placed as beach and dune fill. Since 1956, over one million cubic yards of sand was placed, primarily south of Lincoln Way. Additional sand may have been dumped on the beach and dunes in the late 1940's and early 1950's when nearby residential development peaked, requiring removal of sand dunes from lots. About 100,000 CY of sand was mined between 1963 and 1967 (mining started in 1953). Since completion of the Great Highway in 1929, significant beach and dune nourishment has taken place, while sand mining rates were relatively low. The net volume change to the beach and dunes by man since 1929 is estimated to be an increase of about 1.3 million cubic yards.

Recent history for the Ocean Beach area from *M&N (2005b)* is summarized in Table 2-1.

Table 2-1: Ocean Beach Recent History and Events.

Timeframe	Development
1850 to 1900	Pre-Great Highway construction period.
1927-1929	Completion of the O'Shaughnessy Seawall and construction of the improved Great Highway. 1.26 million CY of sand placed along the length of Ocean Beach, creating a dune 200 feet wide with a top elevation of 30-32 feet MLLW.
1932-1934	San Francisco Bar Channel deepened to -45 feet MLLW. 140,000 CY placed as beach fill.
1942-1943	A portion of San Francisco Bar Channel deepened to -50 feet MLLW.
1959	San Francisco Bar Channel deepened to -50 feet MLLW for full 2000 feet width; 3.84 million CY of sand dredged.
1963	14,500 CY of sand mining on beach.
1964	22,500 CY of sand mining on beach.
1965	25,500 CY of sand mining on beach.
1966	6,900 CY of sand mining on beach.
1967	30,000 CY of sand mining on beach. Sand mining at Ocean Beach terminated.
Early 1970's	Fill placed south of Sloat Boulevard for the extension of the Great Highway
1972	On the southern end of Ocean Beach, ice plant and grasses planted on the dune and concrete blocks and rubble placed in front of the dunes for erosion mitigation.
1972-1975	San Francisco Bar Channel deepened to -55 feet MLLW; 5.8 million CY of sand dredged.
1980-1981	Westside Sewage Transport Box construction started; 600,000 CY of excavated sand placed on the beach as nourishment.
1982-1983	Heavy El Niño season eroded up to 70% of the 1980-1981 sand placement; part of the dune system eroded up to 60 feet; in places the beach was eroded to hardpan.

Timeframe	Development
1984	The Great Highway undercut and damaged by storms.
1985	Sand fencing erected and dune grass planted for dune stabilization south of Lincoln Way.
1986-1993	Sand excavated for a new Great Highway Seawall between Noriega and Santiago Streets; 100,000 CY of sand placed on the beach between 1986 and 1989, and 250,000 CY between 1989 and 1992.
1994	35,000 CY of wind deposited sand covering the new seawall between, including 25,000 CY placed in front of the South Lot south of Sloat Boulevard.
1994-1995	High tides and waves during this El Niño season caused 30-40 feet of bluff retreat between the Sloat Lot and the South Lot (Moffatt & Nichol, 2003), with somewhat less retreat elsewhere.
1995	295,000 CY dredged from the Bar Channel placed at the SF-08 ocean disposal site.
1996	1,009,000 CY dredged from the Bar Channel placed at the SF-08 ocean disposal site.
1996-1997	Precipitation runoff and wave action formed numerous erosion gullies in the bluff face. In March 1997, one storm formed a gully extending to the beach that eroded the bluff to within 15 feet of the Great Highway in the area between the two parking lots.
1997	480,800 CY dredged from the Bar Channel placed at the SF-08 ocean disposal site.
Fall 1997	Initial placement of (minimal) toe protection. Two rows of quarrystone placed by the City at the toe of the bluff between the Sloat Lot and the South Lot, as temporary bluff protection.
1997/1998	Construction of the Emergency Quarrystone Revetment (EQR).
1997/1998	During this El Niño season, loss of sand from the beach south of Sloat Blvd. resulted in extensive erosion of the bluffs. In some areas, beach elevations were lowered 10-15 feet compared to their summer/early fall elevations. The bluff edge retreated up to 30 feet in the unprotected areas at the south end of Sloat Lot. The bluff edge protected by the EQR retreated 2-6 feet in localized areas between the two parking lots. Along the South Lot, bluff edges retreated 10-16 feet and there was an overall over-steepening of the bluff slope, making it likely that future wave undercutting would result in more extensive bluff erosion. High tides and waves eroded a large mass of sand from the beach in front of the temporary quarry stone protection.
1998, Feb.	Additional quarrystone placed on top of the temporary toe protection by the City.
1998	393,800 CY dredged from the Bar Channel and placed at SF-08 ocean disposal site.
1998-1999	The bluff edge retreated approximately 50 feet in places along the section of beach from the south end of South Lot to Funston Cliffs. Bluff slopes have been oversteepened along the entire reach and are more susceptible to slope failure and wave undercutting. Bluff slopes above the toe are 40 to 70 degrees or more. The slope of the face of the bluff south of South Lot was nearly vertical in places following the erosion in early 1999.
1999	270,000 CY dredged from the Bar Channel and placed at SF-08 disposal site.
1999, Oct.	Approximately 20,000 cubic yards of sand placed by the City along a 370-foot long reach of the bluffs south of South Lot to form a temporary sand barrier.
2000	667,000 CY dredged from the Bar Channel and placed at SF-08 disposal site.

Timeframe	Development
2001, Jan.	12,000 CY of sand placed by the City at the temporary sand barrier south of South Lot in response to a loss of almost half of the sand between October 1999 and April 2000.
2000-2001	All of the sand placed in January 2001, plus about 17 feet of the original sand barrier, eroded away. The bluff south of the barrier eroded by 7 to 13 feet.
2001	78,000 CY dredged from the Bar Channel and placed at SF-08 ocean disposal site.
2002	300,000 CY dredged from the Bar Channel and placed at SF-08 ocean disposal site.
2003	Dredging from the Bar Channel placed at SF-08 ocean disposal site. 23,000 CY of material from North Ocean Beach placed on the sand barrier near the SWOO Outfall.
2004	Dredging from the Bar Channel placed at SF-08 ocean disposal site. 15,000 CY of material from North Ocean Beach placed on the sand barrier.
2005	Dredging from the Bar Channel placed offshore of Ocean Beach near Sloat, at an approximate depth of 40 feet MLLW.
2006	Dredging from the Bar Channel placed offshore of Ocean Beach near Sloat, at an approximate depth of 40 feet MLLW.
2007	Dredging from the Bar Channel placed offshore of Ocean Beach near Sloat, at an approximate depth of 40 feet MLLW.
2003-2007	10,000 to 15,000 CY of material from North Ocean Beach placed annually by the City at the temporary sand barrier.
2012, Jan.	Construction of sandbag revetment.
2013	Construction of 2nd Emergency Quarrystone Revetment at erosion hotspot south of South Lot.

2.2. Area Geology and Morphology

The portion of the LMT alignment located within the project area passes through dune sands, Colma Formation, and artificial fill. Bluffs along South Ocean Beach are in the Colma Formation, interspersed with artificial fill, riprap shore protection and rubble along the sections of Great Highway and shoreline parking areas that have historically been managed and only partially protected.

Sand on Ocean Beach originates from several different sources, including sediment from bluff erosion, sand that migrates to the beach from the San Francisco Bar, and sand from other sources imported for beach nourishment.

2.2.1. Bluff Material

The bluff material along the project area is defined as the Colma Formation, which consists of moderately cemented to uncemented sand deposits with varying amounts of clay and silt. The Colma Formation varies in thickness from about 25 feet to 40 feet, and is overlain by a few feet of recent dune sand and artificial fill.

A characterization of the bluff material is summarized in Figure 2-2 based on borings obtained for construction of the SWOO. Boring locations are indicated in Figure 2-1.

The boring profiles shown In Figure 2-2 are organized left to right, indicating boring taken: 1) in the bluff, 2) on the beach, and 3) in the surf zone.

The color coding and abbreviation for classification of material types is provided in the table below the figure to the left. The table on the right indicates the color coding utilized to indicate the strength of the material based on SPT N values obtained during the geotechnical exploration program. SPT N values (blow counts) are indicated on the right-hand side of each boring profile.

The surf zone borings show that a hardpan of very dense material exists below an approximately 6 feet thick layer of mobile sand on the seabed. Based on this information it can be inferred that the low beach profiles surveyed after strong El Niño episodes are like to reflect the hardpan under the beach or very near to it.

Figure 2-1: SWOO Boring Locations.

The bluff borings show the bluff material (Colma Formation) as being very dense, but with the possibility of zones of uncemented very loose, and loose to medium dense material.

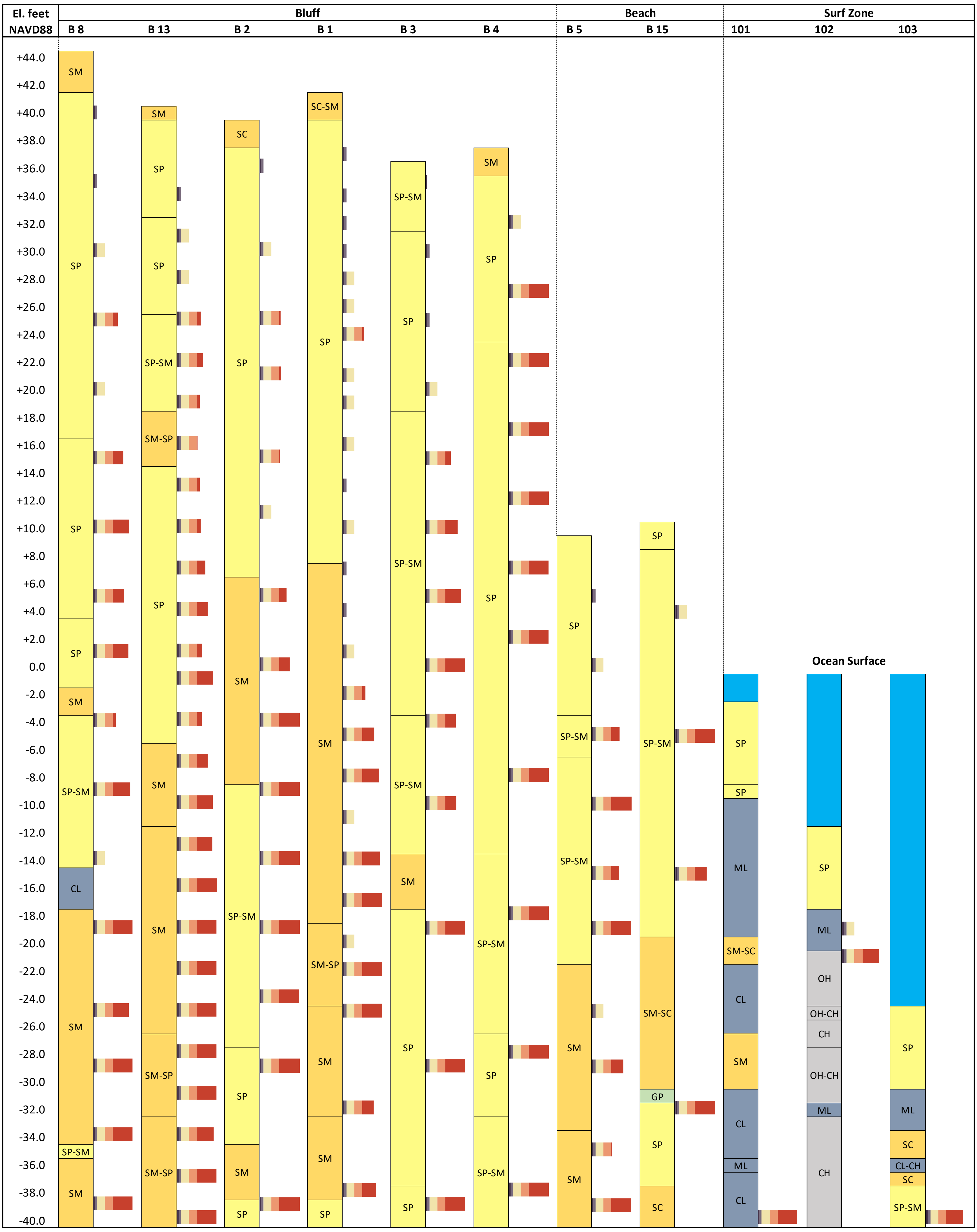


Figure 2-2: Bluff Material Characterization and Relative Density.

Classification		ID	Description
Gravels	Clean	GW	Well graded gravels
		GP	Poorly graded gravels
	With Fines	GM	Silty gravels
		GC	Clayey gravels
Sands	Clean	SW	Well graded sands
		SP	Poorly graded sands
	With Fines	SM	Silty sands
		SC	Clayey sands
Silts and Clays LL 50% or less		ML	Inorganic silts
		CL	inorganic clays
		OL	Organic silts or clays
Silts and Clays LL greater than 50%		MH	Inorganic silts
		CH	inorganic clays
		OH	Organic silts or clays
Highly Organic Soils		Pt	Peat

Relative Density	Legend	SPT N Value
Very Loose		0 - 4
Loose		5 - 10
Medium		11 - 30
Dense		30 - 50
Very Dense		> 50

2.2.2. Beach Material

Median grain sizes for Ocean Beach from *M&N (1995)* are summarized in Table 2-2. See Figure 2-4 for allocation of reaches.

Table 2-2: Median Grain Sizes for Ocean Beach, *M&N (1995)*.

Location	Area	Grain Size (mm)	Date	Comments
Reach 3	North of San Francisco Zoo.	1.07 (Wawona) 0.40 (Vicente) 1.57 (Wawona) 0.25 (Vicente) 0.32 (Wawona) 0.38 (Sloat)	Feb-79 Dec-70 Feb-79 Feb-85 Feb-85 Feb-85	Gavin (1979a) Ecker (1980) Noble (1985) (Grab samples at MSL)
Reach 2	San Francisco Zoo	0.312 to 0.316 0.53 (Center of bathhouse) 0.23 (Dunes) 0.33 (Beach) 0.45 (MLLW)	 Feb-79 Nov-93 Nov-93 Nov-93	Trask (1958) Gavin (1979a) USACE (1993)
Reach 1	South of San Francisco Zoo.	0.180 to 0.300 0.33 (North end of bluff) 0.41 0.64	 Feb-79 Dec-70 Dec-79	WWC (1979) (30' of water) Gavin (1979a) Ecker (1980)

Figure 2-3 shows grain size distributions for Ocean Beach Colma Formation (green curves) and beach sand (yellow curves). The Colma Formation material has a larger percentage of fine to very find sand and silt than the beach sand. This shows that 40-60% of the material in the Colma Formation is washed out when the bluff erodes as this fraction of the material is too fine to be retained on the beach. Only the coarser fraction of the Colma Formation material contributes to nourish the beach. The blue shading in the background of the figure indicates the range of grainsizes obtained from beach grab samples (Table 2-2). The majority of these samples are representative of *Medium Sand*, with a few samples of *fine* and *coarse* to *very coarse sand*.

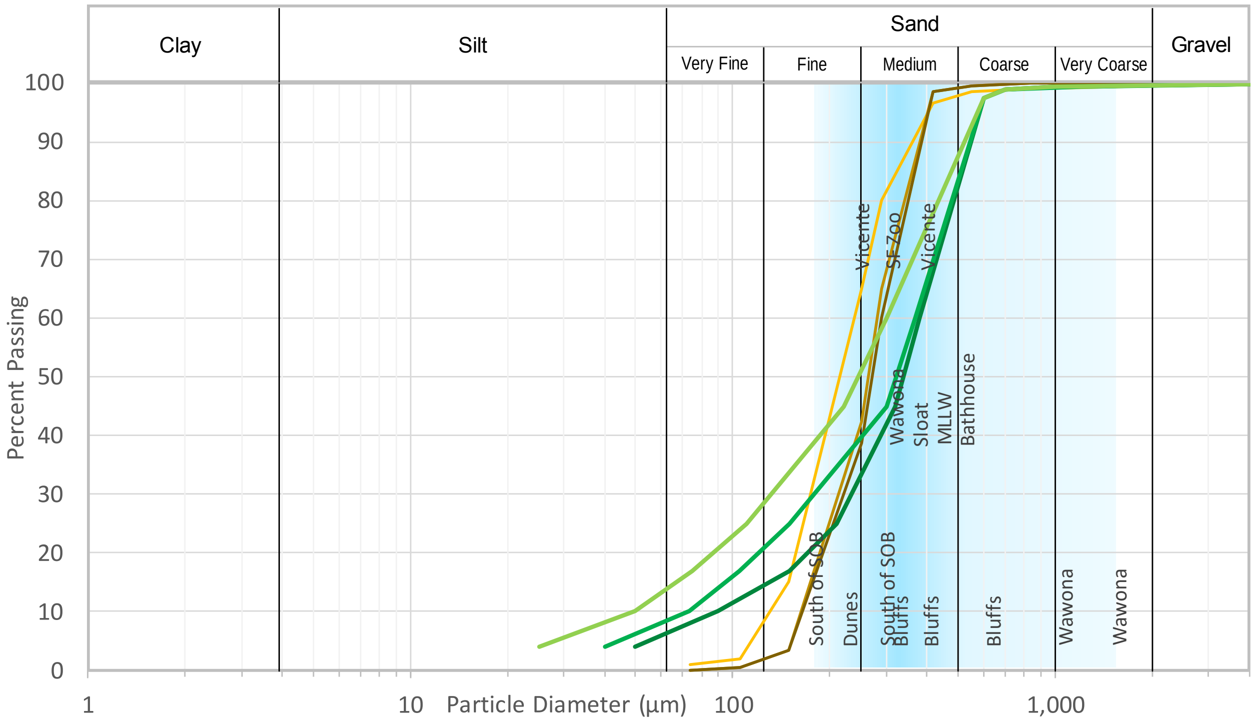


Figure 2-3: Grainsize distribution for beach sand (yellow) and Colma Formation (green).

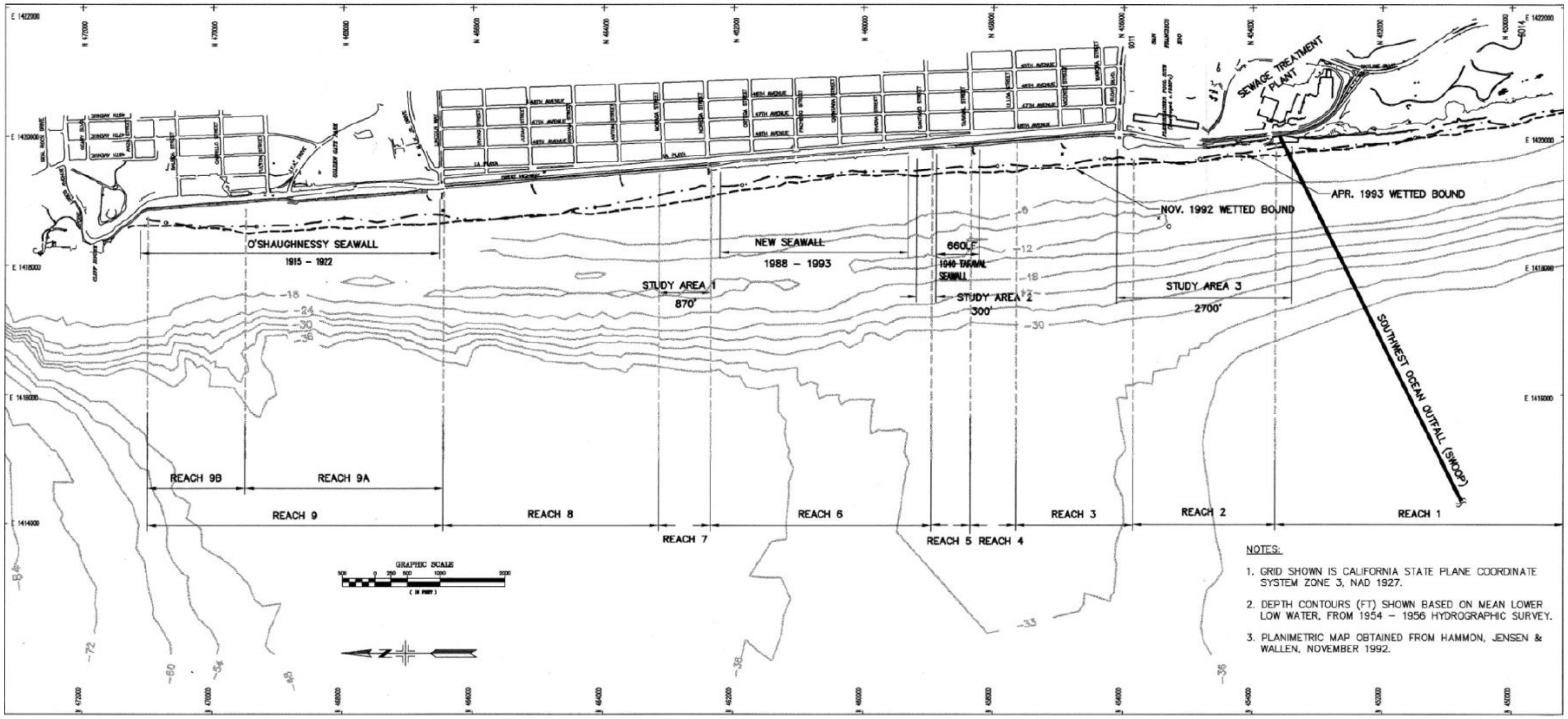


Figure 2-4: Ocean Beach Vicinity Map and Reaches, *M&N (2005a)*.

2.2.3. San Francisco Bar Material

Figure 2-5 shows the distribution of characteristic sediment grain sizes in the region of the San Francisco Bar offshore from the Golden Gate and Ocean Beach. The figure was compiled based on sediment samples acquired by *USGS (2007)* offshore, and by *CHS (1999)* within the bay. Deposits of medium to coarse sand[1] exist over at large area around the Golden Gate where the tidal currents are the strongest. Fine sand is deposited over the San Francisco Bar, and very fine sand is deposited in the deeper water offshore.

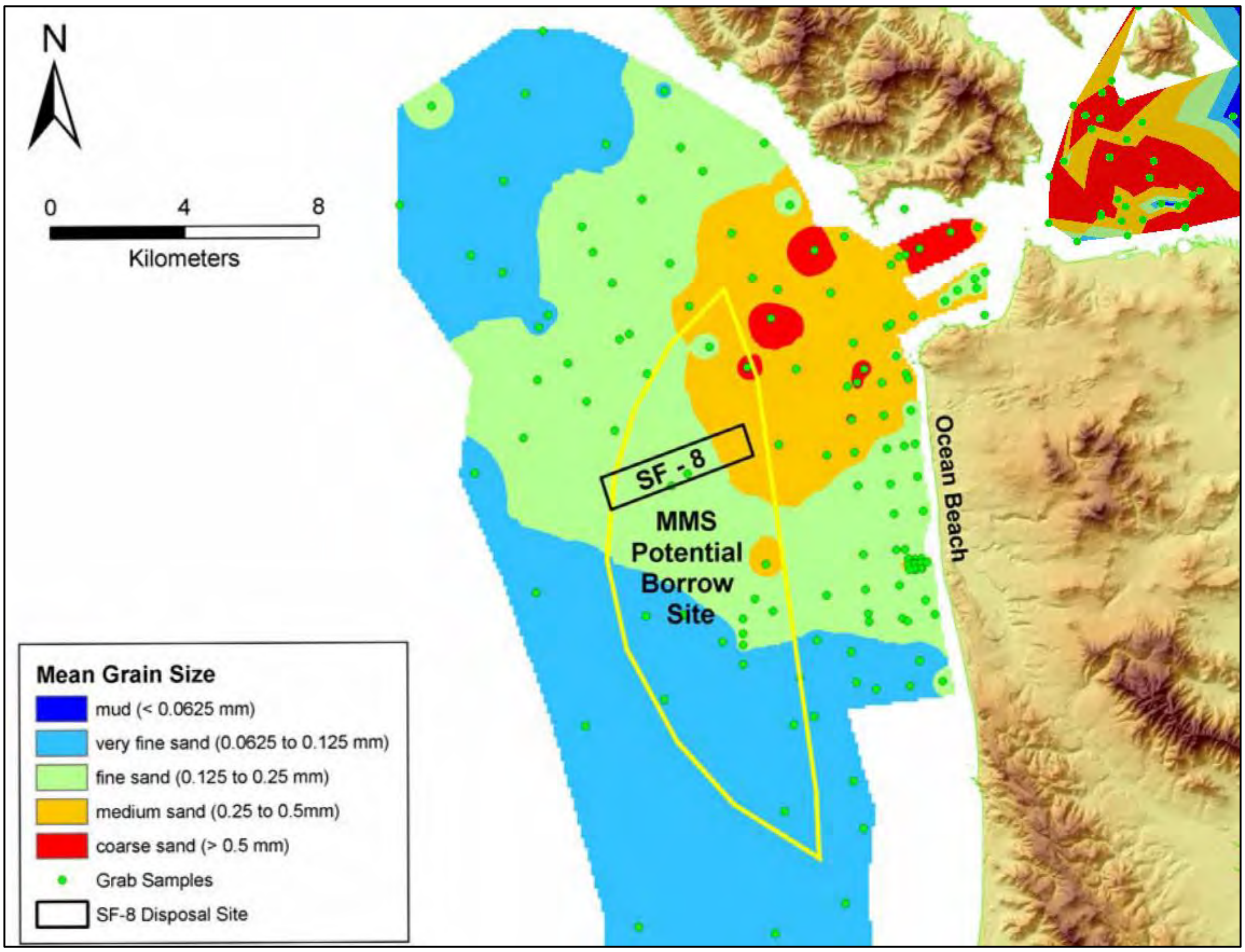


Figure 2-5: Distribution of sediment grain sizes at the Golden Gate.

[1] Classification per the Wentworth Grading Scale, Appendix A, Figure A-1.

2.3. Sediment Transport Patterns

Sediment transport was studied in *M&N (1995)*. This study developed a characterization of sediment in the shore zone along Ocean Beach and offshore on the San Francisco Bar. Sediment transport analysis was utilized to evaluate shoreline changes, changes in beach volume, and corresponding bathymetric changes along the San Francisco Bar based on hydrographic surveys from 1855, 1873, 1884, 1900, and 1954-56.

Both wave-driven littoral sediment transport and wind-induced sand transport was studied. Figure 2-6 summarizes the sediment transport patterns at Ocean Beach. The study established that sediment transported out through the Golden Gate on tidal ebb currents, and dispersed along the San Francisco Bar, is subject to a sediment transport mechanism which brings the sediment back to shore in the area of San Francisco Zoo. From this point the sediment is spread both northwards and southwards.

The past sediment transport studies, *M&N (1995, 2005a)*, concluded the following:

- The San Francisco Bar morphology is integrally linked to Ocean Beach shoreline location, as evidenced from ongoing shoreline recession/beach nourishment practices when the bar navigation channel dredging project dumped material offshore of the Bar. Since ca. 1971 dredged material is being placed on the south lobe of the Bar (SF-08) and shoreline recession, along with the need for beach nourishment in the North Ocean Beach area in particular has decreased substantially.

- The long-term shoreline position in the reach south of the San Francisco Zoo (Reaches 1 & 2) is less influenced by the Bar, and more by cross shore transport and by the bluffs which provide a source of sand to the beaches. A long-term erosional trend was found for these reaches. Alongshore transport also exists in response to storms, but the movement is transitory and moves either north or south. Material moving southwards from Ocean Beach tends to not remain in the Ocean Beach reaches.

- The reach between Sloat and the southerly extent of the South Lot (Reach 2 in study) is subject to reversals in transport direction based on wave climate, and the shoreline is fluctuating about a mean position. Aerial photo analysis between 1938 and 1992 confirmed this finding.

- Net transport over the south lobe of the Bar is towards Ocean Beach, net transport over Fourfathom Bank north of the Golden Gate is along the Bar Channel. Transport between San Francisco Bay and the Bar is seasonal and storm influenced but is an important component of the overall sediment budget. Reduction in the tidal prism of San Francisco Bay due to land reclamation and diminishing sediment supply from the Sierras has caused a radial shrinking of the Bar towards the Gate between the late 1800's and 1950's. The flood tidal channel along Ocean Beach (South Channel) became shallower due to the larger tidal exchange through the dredged bar channel.

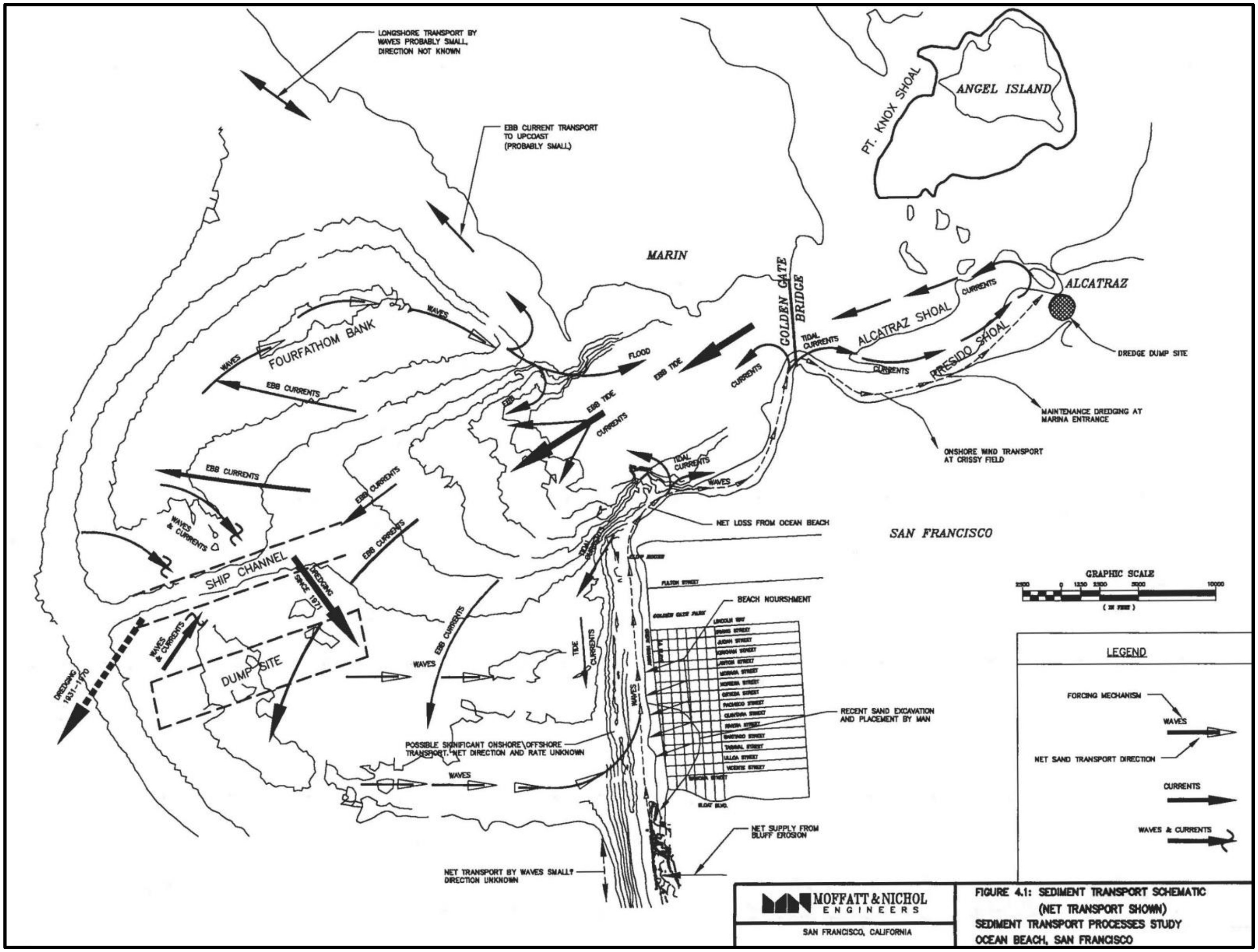


Figure 2-6: Ocean Beach Sediment Transport Patterns, *M&N (2005a)*.

2.4. Bluff Retreat

2.4.1. Short-Term Bluff Recession Rates

The USGS conducted a comprehensive coastal processes study at Ocean Beach from 2004 to 2006, *USGS (2007)*, which concluded the following:

- Single storm events can cause an average shoreline retreat of over 30 feet.
- Shoreline retreat can exceed 65 feet during severe winters, with localized retreat over 230 feet.
- Very strong El Niño conditions such as in the winter of 1997-98 can double the average shoreline retreat.

Figure 2-7 shows shoreline retreat determined by USGS for the 1997-98 El Niño event from *USGS (2007)*. The yellow box in the figure indicates the USGS beach profile transects within the South Ocean Beach project extent. A portion of the shoreline from Profile no. 88 to 115 were categorized as an erosion hot spot, i.e. an area particularly prone to erosion.

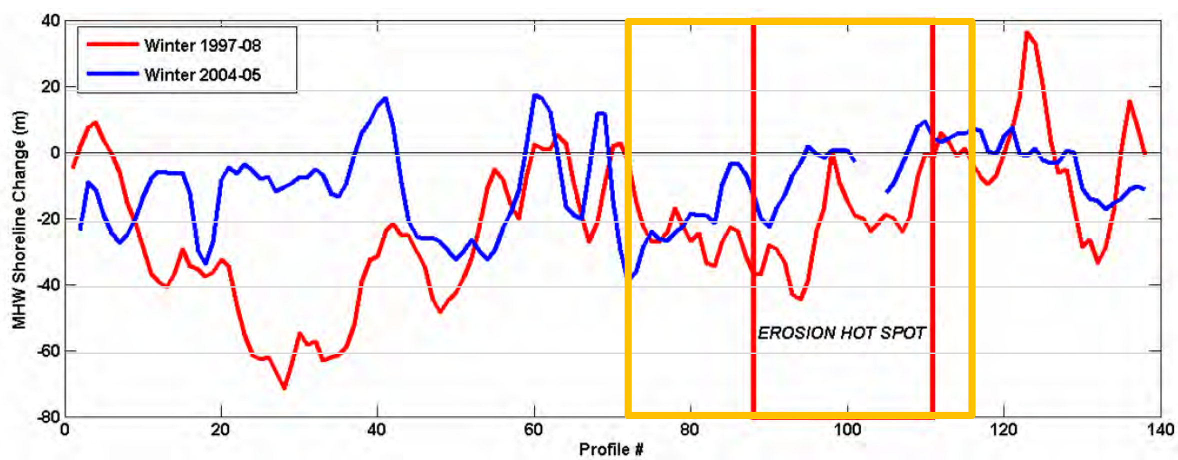


Figure 2-7: Shoreline retreat during 1997-98 El Niño from *USGS (2007)*.

In connection with emergency repairs along the Great Highway in response to erosion during the 2009-10 winter, the recurrence and magnitude of episodic bluff failures was studied. Figure 2-8 summarizes findings from *M&N (2010)* compared with findings from earlier studies. The results indicate that bluff failures on the order of 10 feet can occur every 5-8 years on average; 20 feet of bluff erosion every 8-17 years on average; and 40 feet of bluff erosion every 25-33 years on average.

In addition, Prof. Sitar of University of California together with USGS conducted a detailed study on recession of bluffs composed of weakly cemented and moderately cemented material (Merced Formation), *JOG (2008)*. The study utilized LiDAR surveys to identify episodic bluff failures due to wave action and precipitation runoff. The findings are also summarized in Figure 2-8. As seen in the figure, bluff retreat rates associates with failures in the moderately cemented bluffs are generally consistent with the findings in *M&N (2010)*. Dr. Sitar's data is situated at the lower end of the curve because the data spanned a shorter duration, between 2002 and 2006. Bluff retreat in weakly cemented material (provided for comparison) exhibits higher recession rates as this material is more erodible.

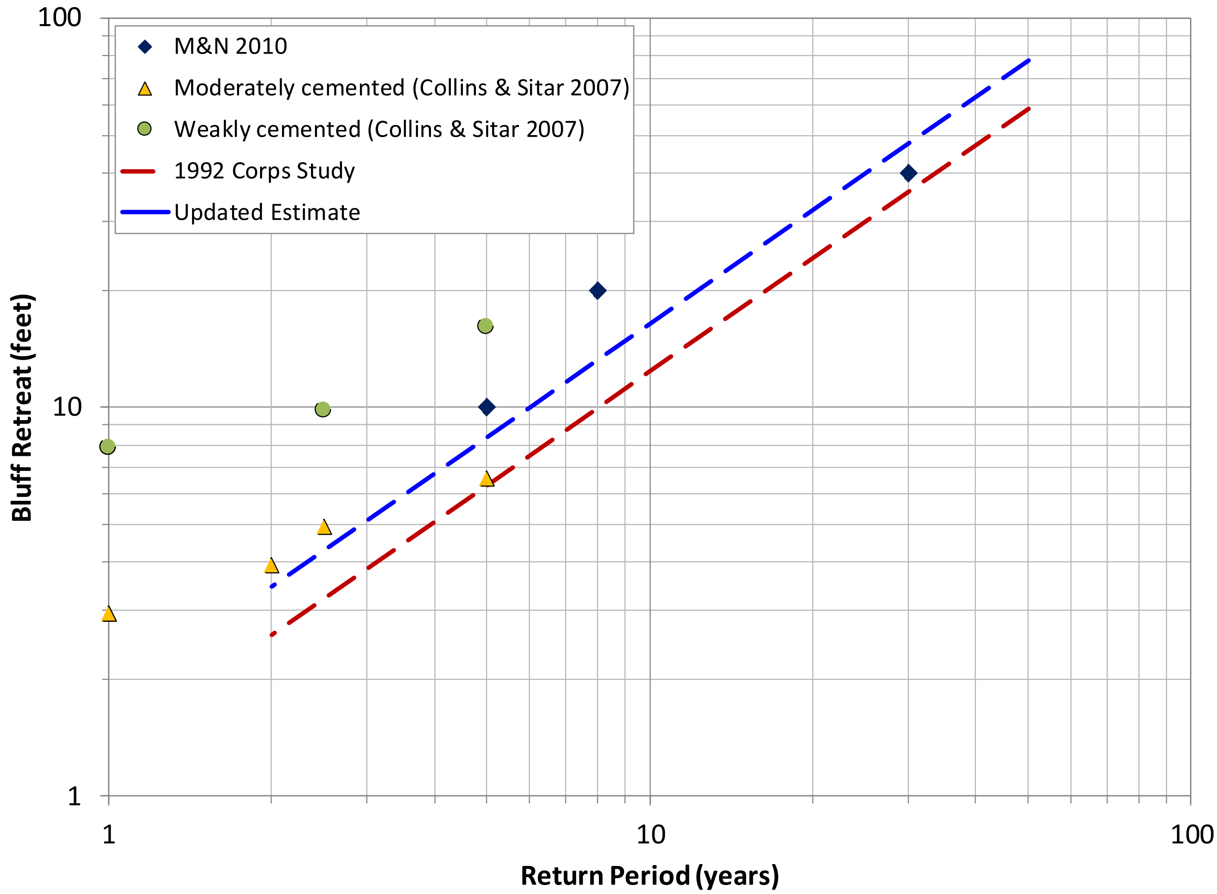


Figure 2-8: Estimated Return Period Bluff Retreat Values.

2.4.2. Long-Term Bluff Recession Rates

Shoreline mapping was conducted in *M&N (1994)*. The analysis determined the location of the toe of the bluff for years: 1938, 1948, 1959, 1970, 1971, 1978, 1980, 1985, 1986, 1992, and 1993.

Historical aerial imagery South Ocean Beach for the years from 1938 to 2019 was inspected and the distance to the bluff edge was mapped at approximately 60-foot intervals. The average rate of bluff retreat was then estimated based on the slope of a linear trend through the data. Figure 2-9 shows examples of how the linear trend was determined for the project reaches. The linear trends for each dataset are indicated by solid lines.

In the case of the dataset represented by the yellow dots, the trend is very near linear, in particular over the years from 1965 to 2018. The dataset captured in the blue dots show a higher degree of variability, possibly indicating periods where the rate of bluff retreat could be lower (flatter trend) or higher (steeper trend) than the average trend indicated. This variability shows that bluff erosion is intermittent and highly episodic. However, a portion of this variability may also be attributed with uncertainties in establishing the individual data point. Some of these uncertainties can include: distortions in the photographic imagery; inaccuracies in the geo-referencing of images; and inaccuracies associated with locating the edge of the bluff, which at some locations is soft. In some cases vegetation has been taken as an indicator of the approximate edge of bluff. In areas where the shoreline has been managed, the bluff edge has been delineated as the edge of hardscape and/or manmade structures.

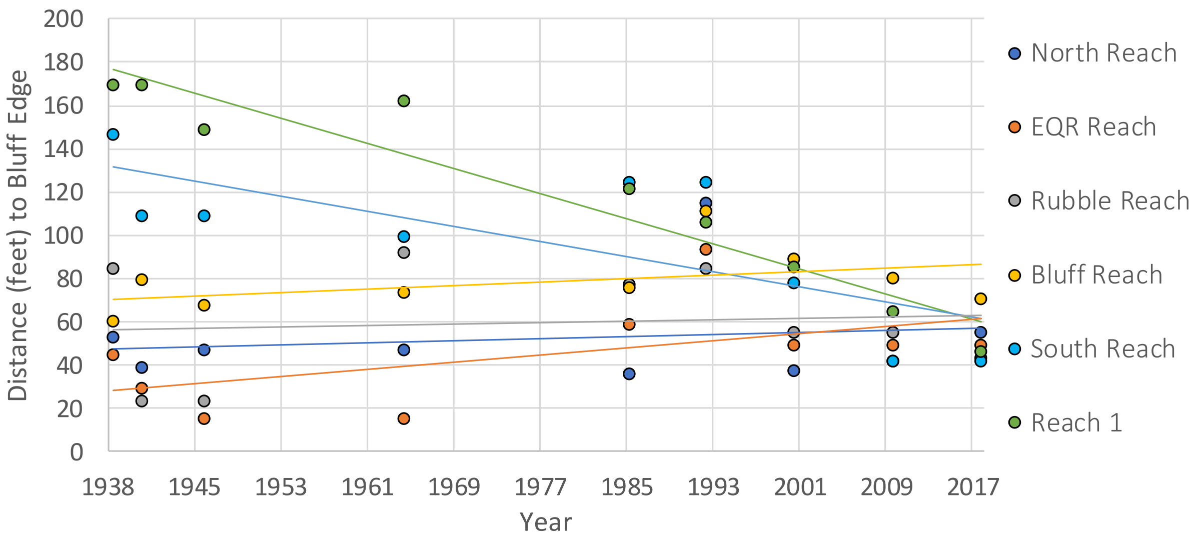


Figure 2-9: Estimation of Bluff Retreat with Linear Trend.

Figure 2-10 shows an example of how the bluff edge was delineated based on imagery from 1938 and 2019.

Figure 2-10: Example Delineation of 1938 Bluff Edge (left) and 2019 Bluff Edge (right).

Figure 2-11 summarizes bluff retreat rates along South Ocean Beach. These rates are determined based on a linear trend of data for the location of the bluff edge over the years from 1938 to 2019. Along the central and northern part of the project area where the shoreline has been maintained since 1938, the rate of retreat is near zero or slightly positive (blue bars in Figure 2-9) due to armoring, intentional or from accumulation of fill debris. This indicates a stable shoreline enabled by manmade shore protective structures.

Transitioning to the southern part of the project area where the bluff is mostly unprotected, the rate of retreat increases progressively. The colored bars indicate the rate of retreat ranging from 0.5 feet per year (light yellow) to 2.4 feet per year (purple).

These findings are consistent with the shoreline change rates determined in *M&N (2005a)*, which established the following recession rates:

- 0.5 to 2.6 feet per year in Reach 1 (unprotected bluffs south of the project area)
- 1.2 feet per year of recession to 0.7 feet per year of advance in Reach 2 (the South Ocean Beach project area)
- 0.9 to 1.8 feet per year of advance in Reach 3 (north of the project area).

Figure 2-11: South Ocean Beach Bluff Retreat Rates.

2.5. Erosion Patterns

Erosion mechanisms are discussed in detail in *M&N (2005b)* and summarized in the following.

2.5.1. Bluff Erosion

Figure 2-12 reproduced from *JOG (2008)* shows the mechanism of bluff failure due to wave action. Wave runup and breaking waves erode the toe of the bluffs, leading to over-steepening and making the bluff susceptible to slope failure.

During the winter when the beach profile is at its lowest and waves generated by storms are larger, the frequency of bluff toe erosion increases dramatically. Bluff toe erosion is exacerbated when water levels are high due to tides, storm surge and/or El Niño effects.

In areas where the bluff crest is low, gully erosion can also occur due to runoff of water from precipitation and wave overtopping.

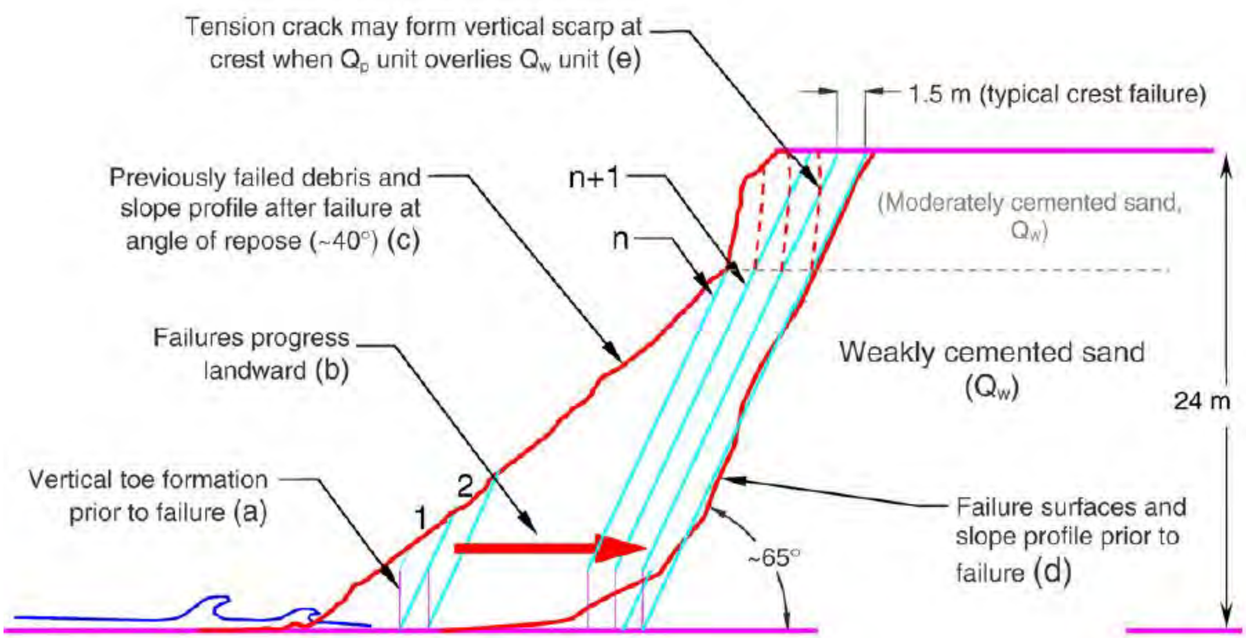


Figure 2-12: Geomorphic Model of Wave Action-Induced Bluff Failure, *JOG (2008).*

2.5.2. Beach Erosion

Wave action brings beach material into suspension and is active across the shore and along the shore. Only a limited fraction of sandy material eroded from bluffs contributes to beach nourishment. The majority of the bluff material is fine and swiftly removed by wave-driven sediment transport.

The presence of shore armoring such as vertical walls and rock revetment tends to result in lowering of the beach level and narrowing of the beach. This effect has been noted in front of the EQR structure.

The yellow lines in Figure 2-13 shows transect locations for representative beach and bluff profiles within the project area. Profiles north of the project area (Reach 3) and south of the project area (Reach 1) are also included for discussion. The LMT alignment is indicated by the light blue curve.

An evaluation of beach and bluff profiles captured in LiDAR topographic surveys is provided in the following for Figure 2-14 to Figure 2-20 at the transect locations shown in Figure 2-13.

Each figure shows beach and bluff surface elevations captured in LiDAR data from 1997 to 2016. Elevation data is referenced to NAVD88. Horizontal distances are in feet referenced to an arbitrary datum. The LMT outline is indicated for reference, seen as the oval shape on the right-hand side of the figures.

In each figure, LiDAR beach and bluff profiles are shown in yellow, orange, and brown colors. The presentation focuses on demonstrating the variability of the profiles, rather than each distinct profile. The highest and lowest envelope of the profiles is indicated by the dashed black lines. The estimated 100-year water levels, present (2019) and with 1.9 feet of sea-level rise by 2050, and 6.9 feet of sea-level rise by 2100 are indicated by the horizontal blue lines.

Figure 2-14 shows beach and bluff profiles for Reach 3. This reach is representative of the northernmost extent of the project area and the shoreline to the north. The profiles indicate that the seasonal variation of the beach profile is approximately 6 feet in the vertical, and there has been a trend of progressive shoreline recession. The bluff face is located between 300-350 feet where the profile turns steeper. The distance from the back beach to the LMT is approximately 110 feet.

Only the 2014 survey shows a beach profile with a berm, indicated by the horizontal extent of beach from 150 to 250 feet at an elevation of around +10 feet NAVD88. The absence of this berm in the majority of the profiles indicates that the beach profile variation is strongly dominated by longshore sediment transport (as opposed to cross-shore transport).

Figure 2-15 for the North Reach shows a similar profile variation. It can be noted that the lowest beach profiles occurred in 2010 and 2016, due to erosion associated with strong El Niño conditions in these years. The distance from the back beach to the LMT is approximately 130 feet.

Profiles for the EQR Reach are shown in Figure 2-16. In these profiles, the steep portion of the profile from 260 to 290 feet is the rock placed for the EQR. This protection is stable and therefore shows as fixed across all of the profiles. The beach profile variation in the vertical is approximately 6-10 feet. This beach profile variation at this location and further south along Ocean Beach are dominated by longshore sediment transport. The distance from the back beach to the LMT is approximately 70 feet.

Figure 2-17 shows beach profiles for the Rubble Reach. The typical beach profile variation in the vertical is around 6 feet. The apparent advance of the bluff face between 2010 and 2014 is due to shore protection placed to combat bluff erosion. The distance from the back beach to the LMT is approximately 100 feet.

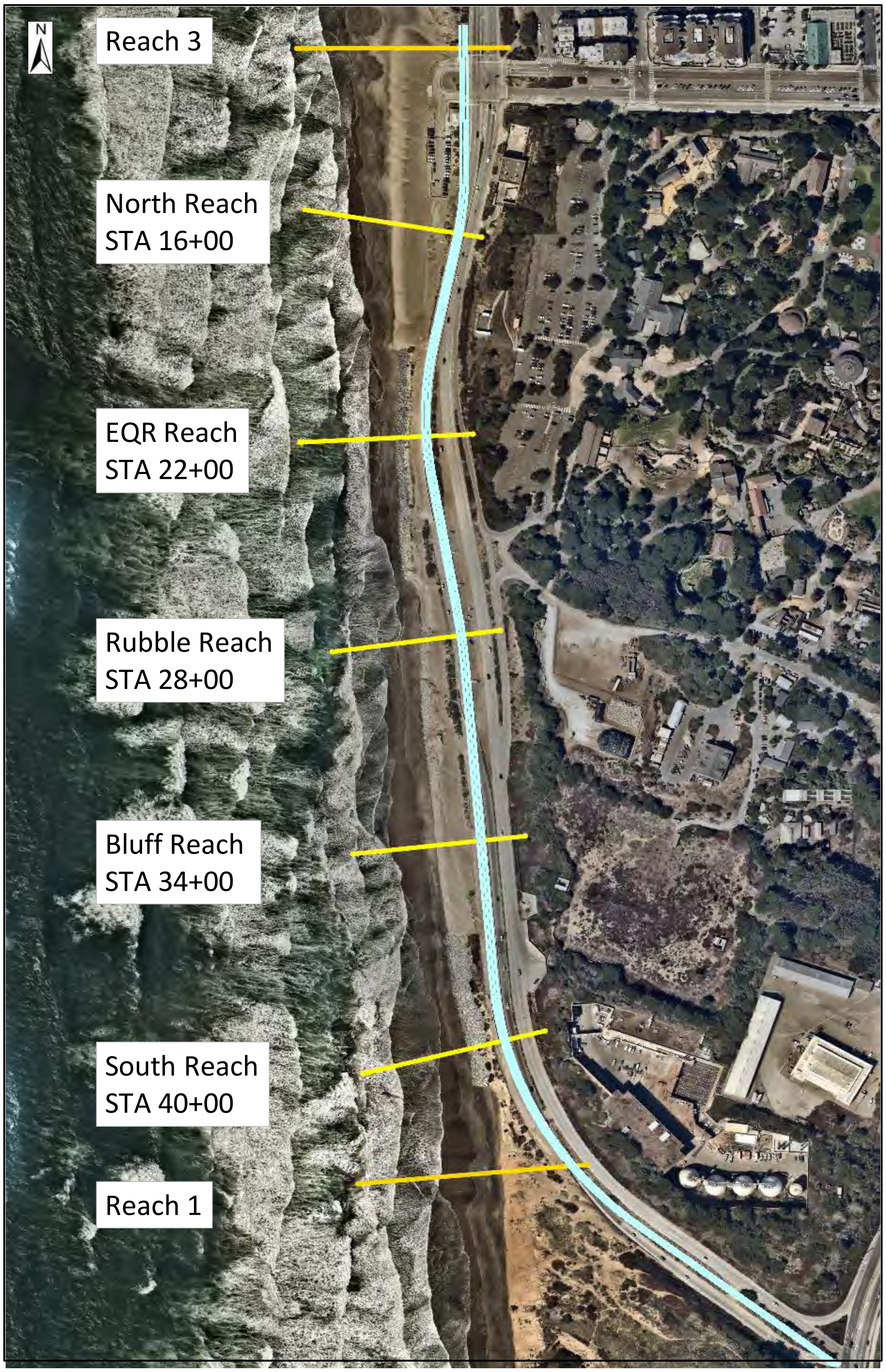


Figure 2-13: Beach and Bluff Profile Transect Locations.

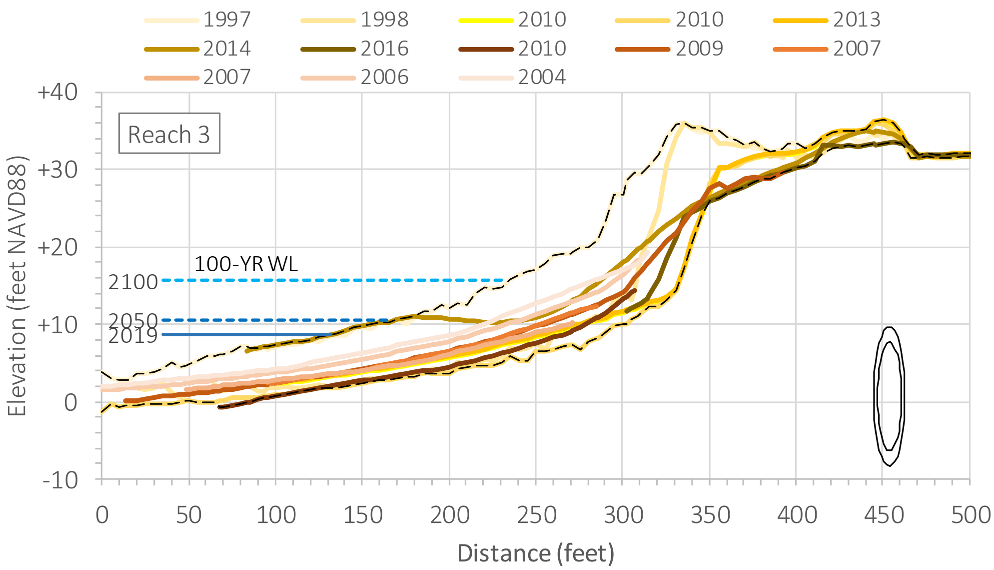


Figure 2-14: Reach 3 (North of Project Area) Beach and Bluff Profiles Captured in LiDAR Topographical Surveys.

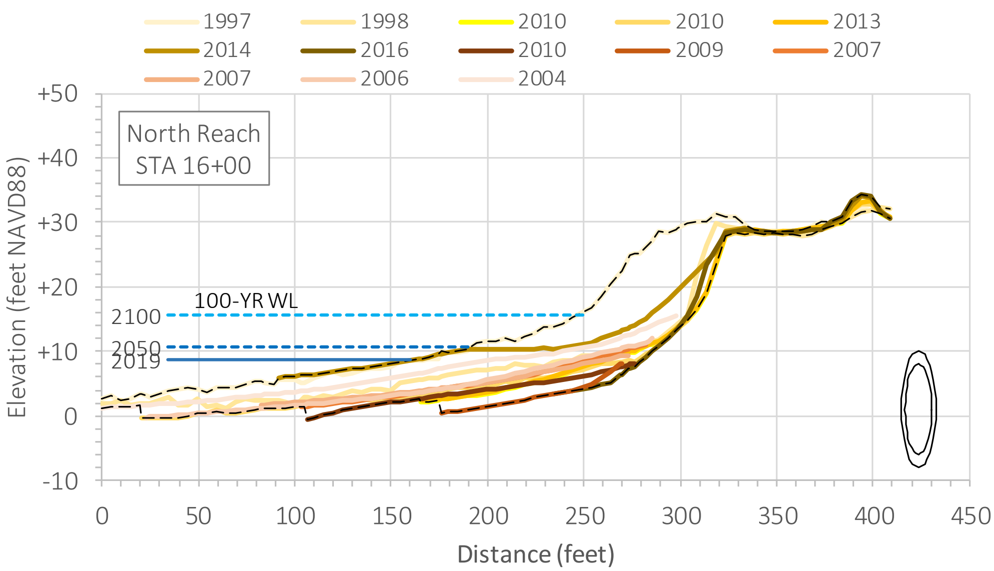


Figure 2-15: North Reach Beach and Bluff Profiles Captured in LiDAR Topographical Surveys.

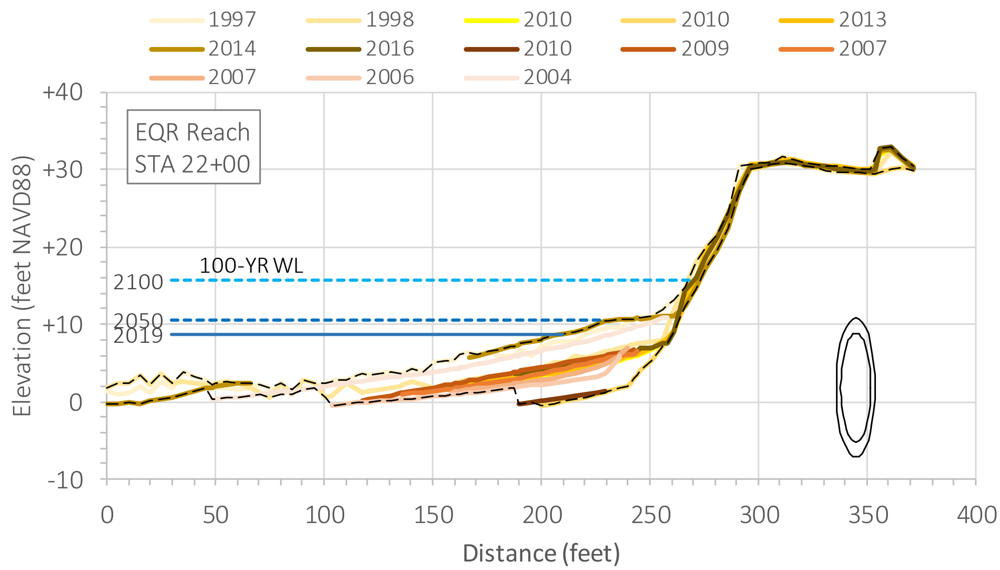


Figure 2-16: EQR Reach Beach and Bluff Profiles Captured in LiDAR Topographical Surveys.

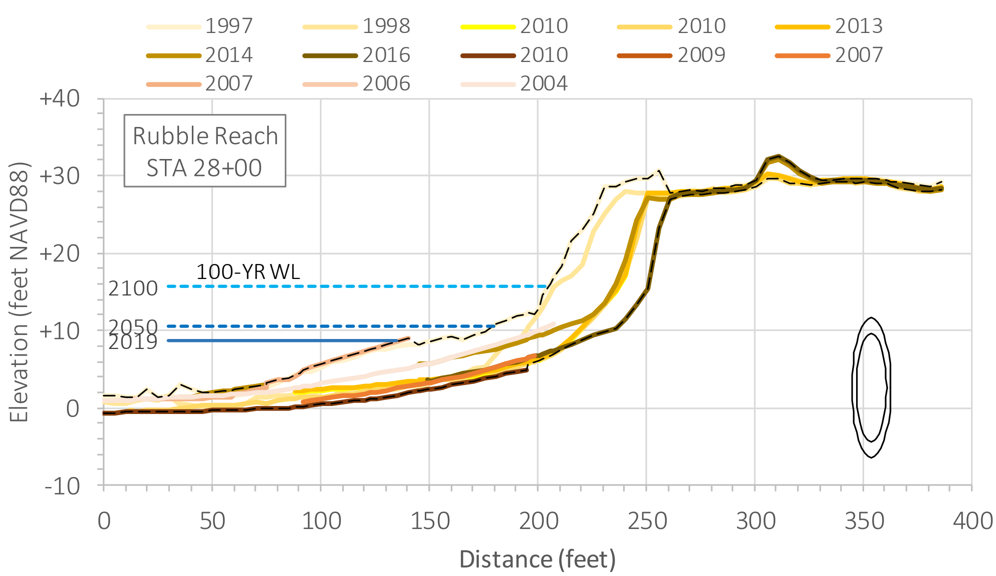


Figure 2-17: Rubble Reach Beach and Bluff Profiles Captured in LiDAR Topographical Surveys.

Figure 2-18 shows the beach profile variation at the Bluff Reach. The vertical variation of the beach profile is in this case limited to 4-6 feet. Note that the toe of the bluff is at around 250 feet. The distance to the LMT is approximately 85 feet.

Beach profile variation for the South Reach is shown in Figure 2-19. This location experienced substantial erosion during the 2009-10 El Niño where the toe of the eroded bluff came within approximately 50 feet of the LMT. Emergency rock protection was subsequently put in place at this location.

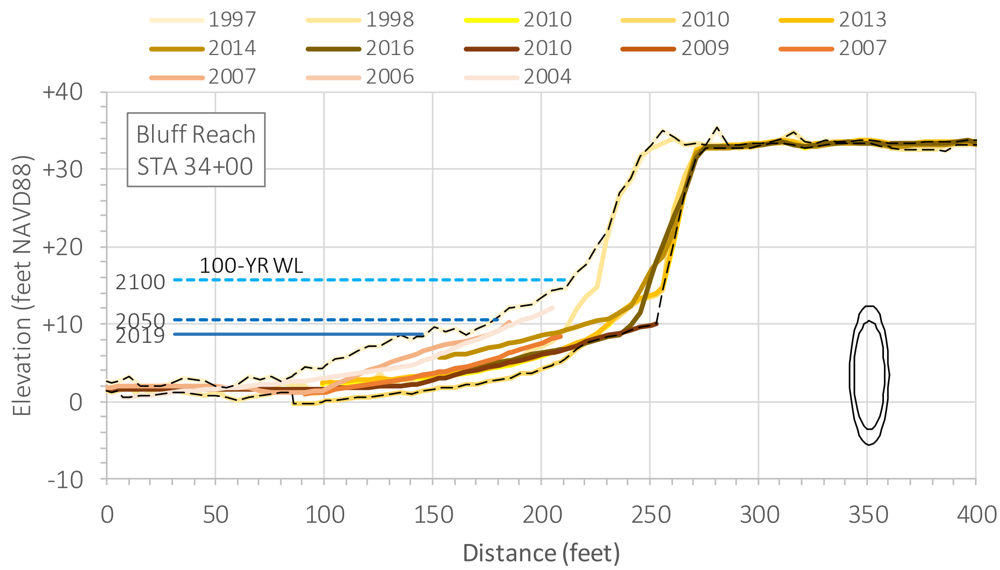


Figure 2-18: Bluff Reach Beach and Bluff Profiles Captured in LiDAR Topographical Surveys.

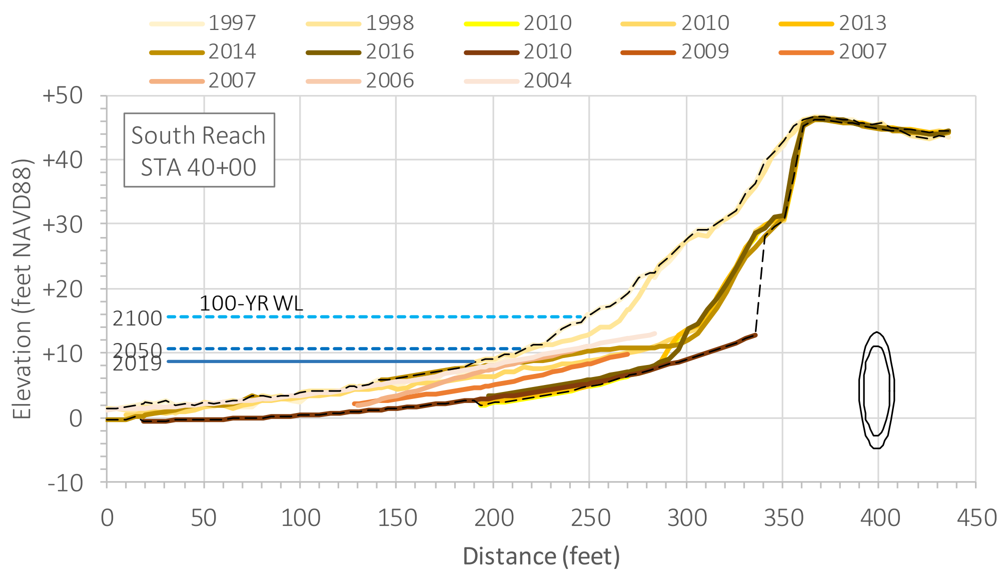


Figure 2-19: South Reach Beach and Bluff Profiles Captured in LiDAR Topographical Surveys.

Figure 2-20 is representative of Reach 1 (see Figure 2-4 for Reach definitions from earlier studies), which is located south of the project area. This reach is of interest because the shoreline and bluff is unprotected and therefore better reflect the natural shoreline morphological processes.

The LiDAR topography shows that considerable bluff erosion has taken place since 1997. The profiles showing the lowest beach elevations are associated with strong El Niño conditions in 2010 and 2016.

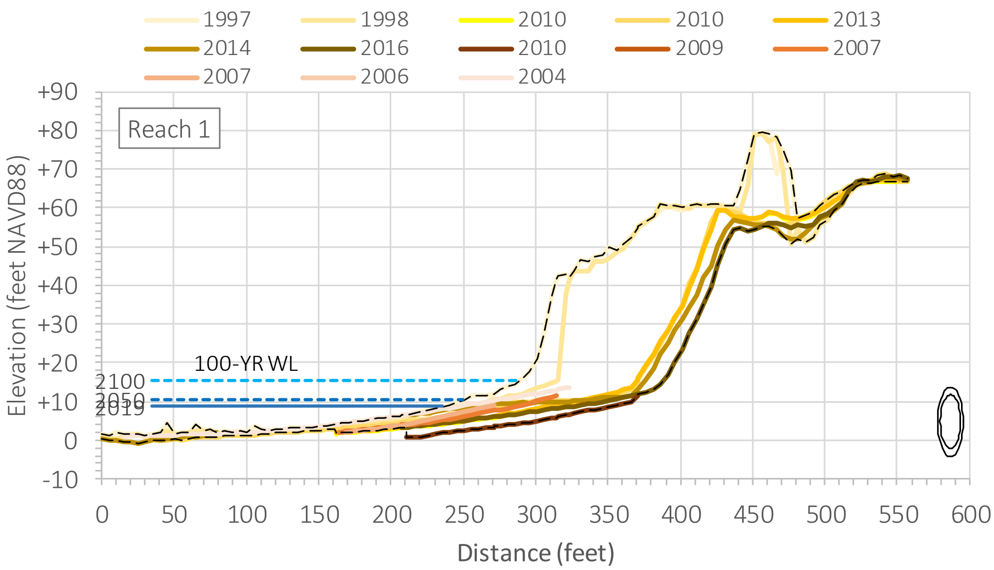


Figure 2-20: Reach 1 (South of Project Area) Beach and Bluff Profiles Captured in LiDAR Topographical Surveys.

2.5.3. Numerical Modeling of Beach Profile Change

Long-term bluff and beach erosion, and beach recovery rates can be estimated with the help of numerical modeling. According to previous studies by M&N, (*M&N, 1995; 2005a*) the long-term shoreline changes at South Ocean Beach were found to be less influenced by the San Francisco Bar, and more by cross shore transport and by the bluffs which provide a source of sand to the beaches. Although longshore transport exists, the material moves briefly either north or south. Material arriving from the northern part of Ocean Beach does not remain in reaches in South Ocean Beach.

As described previously in Section 2.5.1, bluff erosion occurs when wave runup and breaking waves erode the toe of the bluffs, leading to over-steepening and making the bluff susceptible to slope failure (*JOG, 2008*). Considering the nature of processes influencing the sediment transport in the South Ocean Beach reaches, numerical models that evaluate changes along various cross-shore profiles would be adequate.

XBeach is a numerical model for wave propagation, long waves and mean flow, sediment transport and morphological changes of the nearshore area, beaches, dunes and backbarriers. It is a public-domain model that has been developed with funding from the United States Army Corps of Engineers (USACE), Rijkswaterstaat (the Dutch Ministry of Infrastructure and Environment) and the European Union, supported by a consortium of UNESCO-IHE, Deltares, Delft University of Technology and the University of Miami.

Input to the XBeach model include the elevation along a profile and sediment characteristics, such as density, porosity, and median diameter (D_{50}). The model also has the capability to include multiple sediment layers with different characteristics, and simulate structures such as seawalls, defined as impermeable non-erodible features. Long-term bed level changes can be calculated with the help of morphological acceleration factor, which speeds up the morphological time scale relative to the hydrodynamic time scale.

Model results are provided in the following. Figure 2-21 shows changes in the beach profile due to larger fall and winter waves, and Figure 2-22 shows beach recovery during benign spring and summer wave conditions.

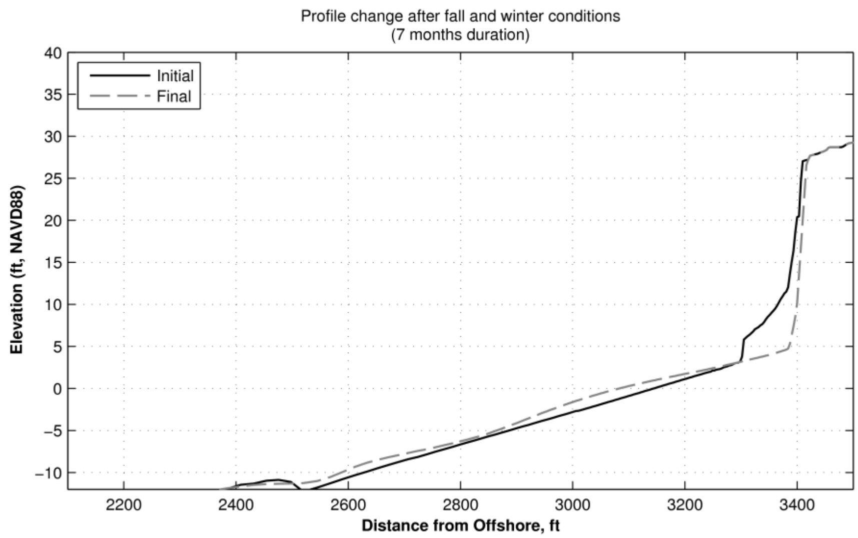


Figure 2-21: XBeach Simulation of Beach Profile Change during Fall and Winter Conditions

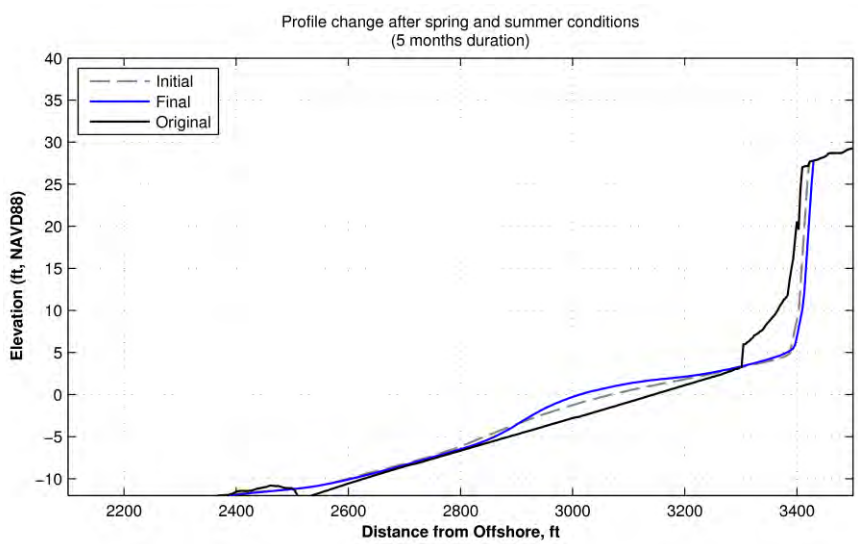


Figure 2-22: XBeach Simulation of Beach Profile Change during Spring and Summer Conditions

2.6. El Niño Effects

Past observations have revealed that bluff erosion is closely correlated with winter storm and high tides in El Niño years. *M&N (2003, 2005a)* summarizes erosion rates as follows. See Figure 2-4 for definition of reaches.

Table 2-3: Bluff Retreat Observed During El Niño Episodes.

Year	Bluff Retreat (feet)
Winter of 1994/1995	30 to 40 between the Sloat lot and the north part of the South lot.
Winter of 1997/1998	High tides and waves eroded a large mass of sand from the beach in front of the Emergency Quarrystone Revetment (EQR). Beach elevations were lowered 10-15 feet. The bluff edge retreated up to 30 feet in the unprotected areas at the south end of the Sloat Lot. The bluff edge protected by the EQR retreated only 2-6 feet in localized areas between the two parking lots. Along the South Lot, bluff edges retreated 0-16 feet.
Winter of 1998/1999	The bluff edge retreated approximately 50 feet in places along the section of beach from the south end of South Lot to Funston Cliffs.
Summer of 2000	20 feet in Reach 1 and 3.
Winter of 2000/2001	The bluff south of the sand barrier placed in 1999 (replenished in 2001) eroded back by 7 to 13 feet.
Winter of 2003/2004	10 feet in Reach 1 and 3.
Spring of 2010	20 feet in Reach 1 and 40 feet in Reach 3.

2.7. Climate Cycles

The two primary climate cycles that govern climate patterns on the Pacific Coast are the El Niño Southern Oscillation (ENSO) and the Pacific Decadal Oscillation (PDO).

2.7.1. El Niño Southern Oscillation

The El Niño Southern Oscillation (ENSO) reflects irregular variations of the sea surface temperature in the Eastern Pacific. The warming phase is termed El Niño while the cooling phase is named La Niña.

Since 1950, the oceanographic community has used the Oceanic Niño Index (ONI) to characterize ENSO ocean temperatures (Figure 2-23). When warming of the ocean exceeds +0.5˚C El Niño conditions prevail. If the ocean temperature cools below -0.5˚C La Niña conditions are present. Within the range of+/-0.5˚C, conditions are termed ENSO-neutral. The ENSO cycle affects temperatures and rainfall worldwide.

El Niño and La Niña cycles typically last 9 to 12 months. They often commence in June or August and reach their peak during December through April, and subsequently, decay over May through July of the following year. Their periodicity is irregular, occurring every 3 to 5 years on average.

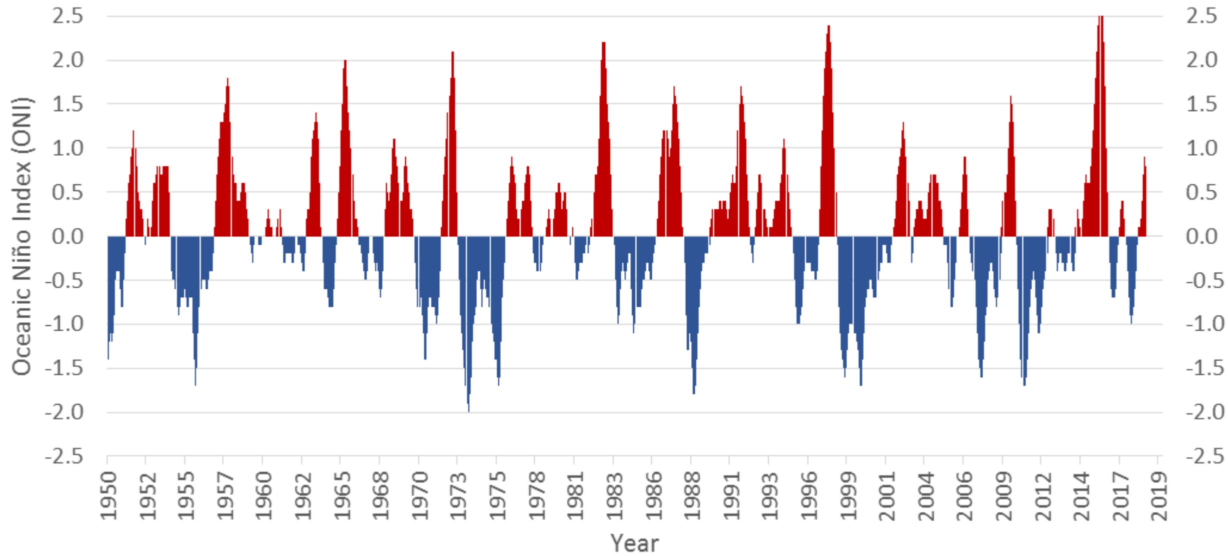


Figure 2-23: ENSO variation (1950-2017).

Table 2-4 groups years categorizes as having very strong, strong, moderate, or weak El Niño or La Niña conditions.

Table 2-4: Years with Very Strong, Strong, Moderate and Weak El Niño / La Niña Conditions.

El Niño				La Niña		
Very Strong	**Strong**	**Moderate**	**Weak**	**Weak**	**Moderate**	**Strong**
1972-73	1957-58	1951-52	1952-53	1954-55	1955-56	1973-74
1982-83	1965-66	1963-64	1953-54	1964-65	1970-71	1975-76
1997-98	1972-73	1968-69	1958-59	1971-72	1995-96	1988-89
2015-16	1987-88	1986-87	1969-70	1974-75	2011-12	1999-00
	1991-92	1994-95	1976-77	1983-84		2007-08
		2002-03	1977-78	1984-85		2010-11
		2009-10	1979-80	2000-01		
			2004-05	2005-06		
			2006-07	2008-09		
			2014-15	2016-17		
			2018-19	2017-18		

Extreme-Value Analysis (EVA) was utilized to assess return periods for the recurrence of El Niño and La Niña events. The data is summarized in Figure 2-24 and Figure 2-25. The results show that very strong El Niño conditions occur every 14 years on average (Oceanic Niño Index > 2.0). Return periods for Strong, Moderate, Weak, and Neutral conditions are indicated along the horizontal axis in Figure 2-24.

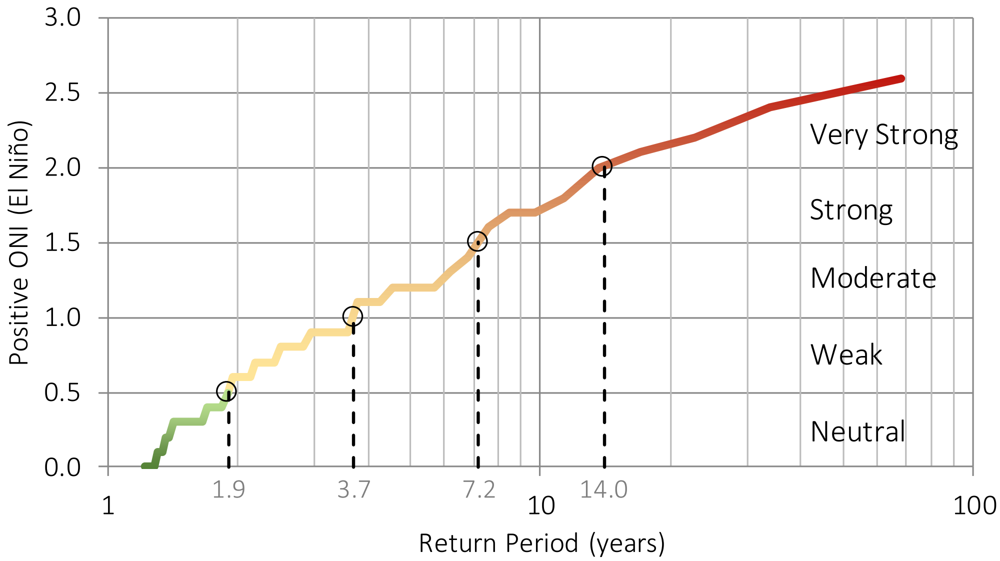


Figure 2-24: Recurrence interval of El Niño Conditions.

Recurrence intervals for La Niña conditions are shown in Figure 2-25, where Strong La Niña conditions are estimated to occur every 8 years on average (Oceanic Niño Index < -1.5). Very Strong La Niña conditions have so far not been observed.

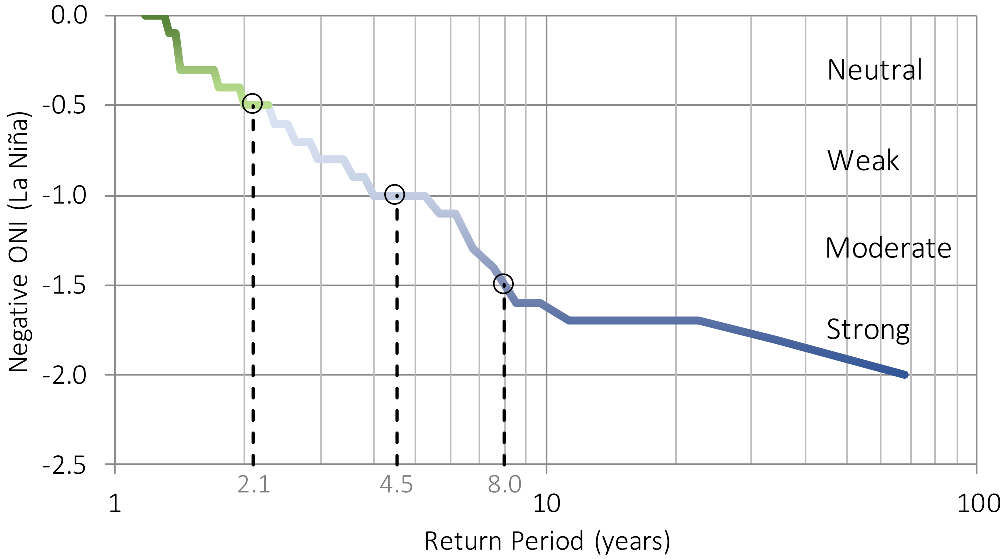


Figure 2-25: Recurrence Interval of La Niña Conditions.

2.7.2. Pacific Decadal Oscillation

Figure 2-26 shows the variation of the Pacific Decadal Oscillation (PDO), which is another climate cycle that produces ocean warming and cooling trends over decades, as opposed to ENSO variations which unfold over months to years.

The data from 1950 to 1976 show a cooling trend (blue), followed by a warming phase from 1976 to 2005. A brief cooling phase occurred from 2005 to 2014, after which another warming phase has commenced. A comparison of Figure 2-23 and Figure 2-26 reveals that variations of the PDO over the short term are influenced by the ENSO directly. Thus, it seems that when these two oscillations are out of phase, they may to some extent moderate ocean cooling and warming, and when they are in phase, combine to produce increased warming or cooling.

Warming of the ocean causes it to expand, increasing the water level above normal. The effects that may combine to intensify shoreline erosion include El Niño conditions, typically reaching a peak in the winter months where storms are prevalent, which in combination with a warming phase of the PDO can lead to above-normal shoreline erosion.

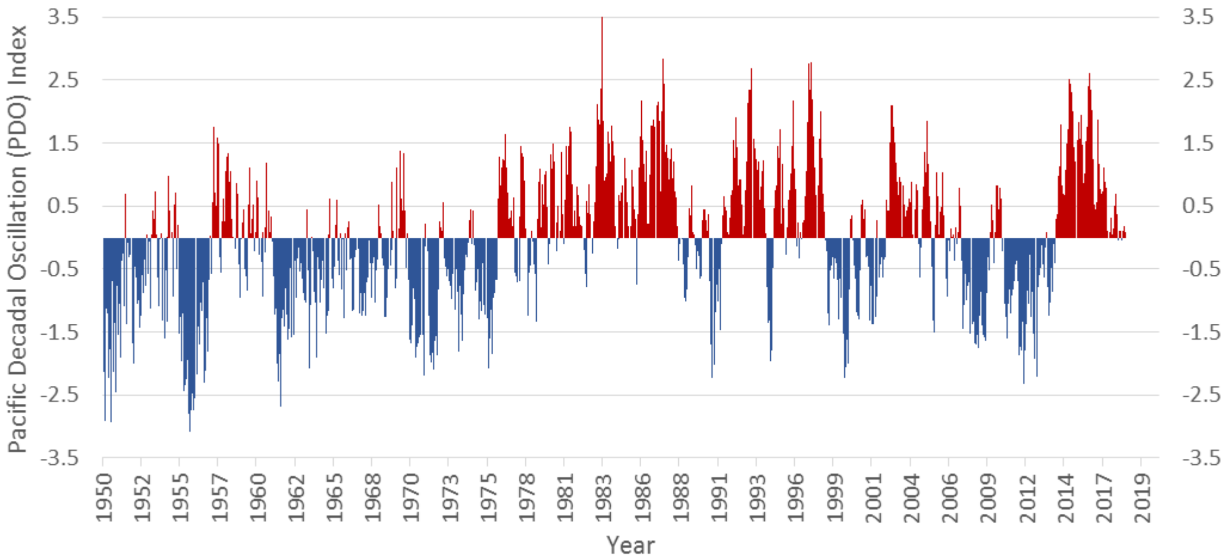


Figure 2-26: PDO variation (1950-2019).

Figure 2-27 shows the variation of tides at NOAA Station 9414290, San Francisco, indicated by the light blue shading, with elevations referenced to NAVD88. The dark blue line indicates the variation of the Mean Water Level (MWL) obtained through tidal filtering, i.e. removal of the tidal variation, leaving the mean. A composite of the Oceanic Niño Index and Pacific Decadal Oscillation Index (ONI-PDO) is superimposed on the figure for comparison (NTS).

It can be observed that several instances of increases of the MWL coincide with peaks in the ONI-PDO variation. A similar trend can be seen for ocean cooling, i.e. lower MWL coinciding with lower ONI-PDO, although the cooling cycles are not as obvious as the warming cycles.

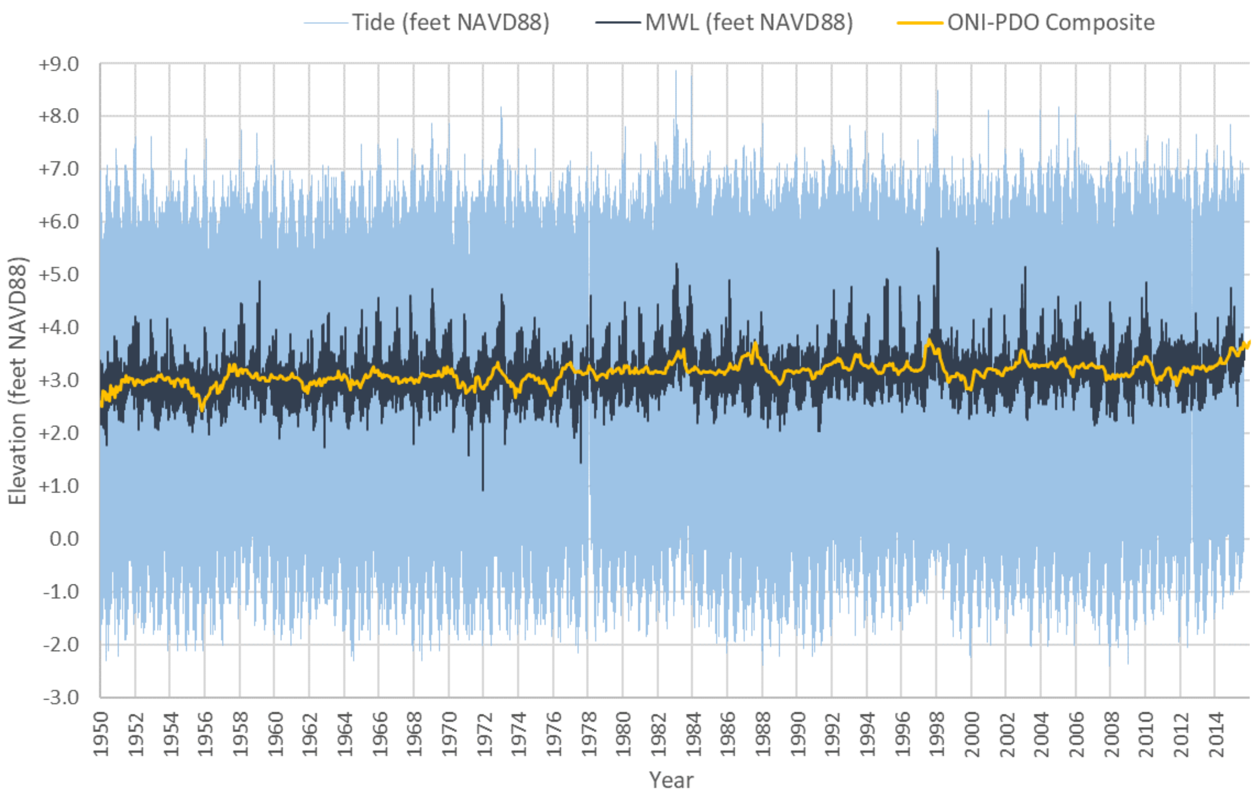


Figure 2-27: Tidal variation, mean water level, and Oceanic Niño – Pacific Decadal Oscillation Index.

The maximum MWL increase recorded at San Francisco is 2.6 feet, while the largest decrease of the MWL is -2.0 feet. Periods of elevated or lowered ocean levels can be on the order of months, while the peak highs and lows occur on a scale of days to weeks.

2.8. Sea-Level Rise

2.8.1. Changing Shorelines

There are several different ways sea-level rise can affect shorelines. Along shorelines subject to wave action, the typical response of the shoreline to sea-level rise is to recede inland. This happens as the shoreline profile rebalances itself around the new higher mean sea level. This effect was described in 1962 by Per Bruun and is known as the *Bruun Rule*. In order to maintain the same beach slope, upland material is eroded and shifted to the below water portion of the profile. In undeveloped areas the effect may be pronounced recession of the shoreline. If there is an insufficient supply of sediment available to raise the shoreline profile in tow with sea-level rise, the result can be accelerated erosion and deepening of the coastal waters. This in turn allows larger waves to impact the shore which further exacerbates erosion. These effects are the reason why shorelines often experience a higher degree of erosion during strong El Niño episodes occurring over the winter months. The El Niño conditions cause the ocean level to be higher which manifests as a temporary sea-level rise.

2.8.2. Planning for Sea-Level Rise

Current guidance for California recommends evaluation of SLR impacts using a scenario-based analysis. This method is founded on the approach by the Intergovernmental Panel on Climate Change (IPCC) to understand how SLR and other drivers interact to threaten health, safety, and resources of coastal communities. Comprehensive SLR guidance for California was first developed by the National Research Council, *NRC (2012).* The guidance relied on the best available science at the time to identify a range of sea-level rise scenarios including high, low, and intermediate projections, taking into account regional factors such as El Niño and extreme storm events that affect ocean levels, precipitation, and storm surge. This approach allows planners to understand the full range of possible impacts that can be reasonably expected based on the best available science, and build an understanding of the overall risk posed by potential future SLR.

The best available science and most recent guidance is provided in *OPC (2018)* and has been adopted for this vulnerability assessment. Table 2-5 summarizes SLR scenarios adopted from *OPC (2018)* for time horizons out to 2150. The columns outlined in dark blue reflects the OPC guidance for risk levels, which include low risk aversion, medium to high risk aversion, and extreme risk aversion. The SLR scenario adopted for this analysis is the *Medium – High Risk Aversion* scenario, assuming high greenhouse gas (GHG) emissions.

Figure 2-28 depicts the *OPC (2018)* SLR projections from Table 2-5 in graphical form, where the *Medium to High Risk Aversion* scenario with high emissions is indicated by the red line. The projected sea-level rise for planning horizons 2030, 2050, and 2100 is indicated by the red circles. Present day conditions for 2019 is indicated by the black circle (E) denoting existing conditions.

The horizontal blue bands provide an indication of when the adopted levels of sea-level rise could be experienced under the other *OPC (2018)* scenarios, e.g. low risk aversion under a low or high GHG emissions scenario or extreme risk aversion indicated by the purple line for the H++ scenario.

Table 2-5: Sea-Level Rise Projections for San Francisco Bay Area, *OPC (2018)*.

		Probabilistic Projections (in feet) (based on Kopp et al. 2014)						H++ scenario (Sweet et al. 2017) *Single scenario
		MEDIAN	LIKELY RANGE			1-IN-20 CHANCE	1-IN-200 CHANCE	
		50% probability sea-level rise meets or exceeds...	66% probability sea-level rise is between...			5% probability sea-level rise meets or exceeds...	0.5% probability sea-level rise meets or exceeds...	
					Low Risk Aversion		Medium - High Risk Aversion	Extreme Risk Aversion
High emissions	2030	0.4	0.3	-	0.5	0.6	0.8	1.0
	2040	0.6	0.5	-	0.8	1.0	1.3	1.8
	2050	0.9	0.6	-	1.1	1.4	1.9	2.7
Low emissions	2060	1.0	0.6	-	1.3	1.6	2.4	
High emissions	2060	1.1	0.8	-	1.5	1.8	2.6	3.9
Low emissions	2070	1.1	0.8	-	1.5	1.9	3.1	
High emissions	2070	1.4	1.0	-	1.9	2.4	3.5	5.2
Low emissions	2080	1.3	0.9	-	1.8	2.3	3.9	
High emissions	2080	1.7	1.2	-	2.4	3.0	4.5	6.6
Low emissions	2090	1.4	1.0	-	2.1	2.8	4.7	
High emissions	2090	2.1	1.4	-	2.9	3.6	5.6	8.3
Low emissions	2100	1.6	1.0	-	2.4	3.2	5.7	
High emissions	2100	2.5	1.6	-	3.4	4.4	6.9	10.2
Low emissions	2110*	1.7	1.2	-	2.5	3.4	6.3	
High emissions	2110*	2.6	1.9	-	3.5	4.5	7.3	11.9
Low emissions	2120	1.9	1.2	-	2.8	3.9	7.4	
High emissions	2120	3	2.2	-	4.1	5.2	8.6	14.2
Low emissions	2130	2.1	1.3	-	3.1	4.4	8.5	
High emissions	2130	3.3	2.4	-	4.6	6.0	10.0	16.6
Low emissions	2140	2.2	1.3	-	3.4	4.9	9.7	
High emissions	2140	3.7	2.6	-	5.2	6.8	11.4	19.1
Low emissions	2150	2.4	1.3	-	3.8	5.5	11.0	
High emissions	2150	4.1	2.8	-	5.8	5.7	13.0	21.9

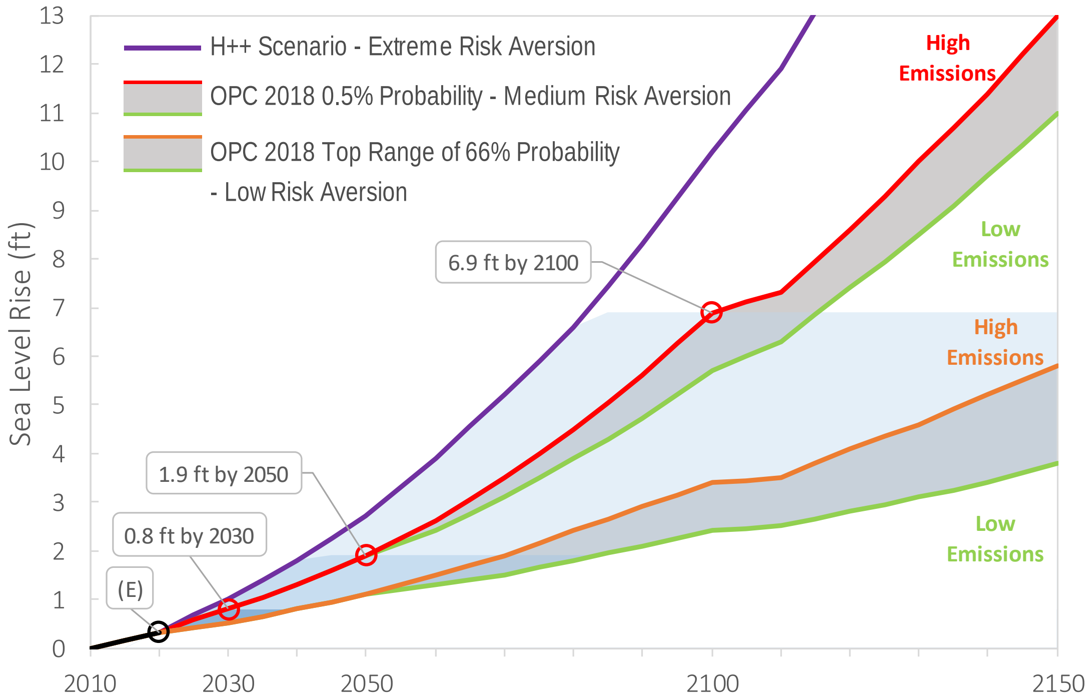


Figure 2-28: OPC Sea-Level Rise Projections.

2.9. Sea-Level Rise Scenarios

Coastal erosion is projected to increase with sea-level rise. Additional factors that can exacerbate coastal erosion events include high tides, storm surge, El Niño effects, and elevated groundwater tables. These elements can increase the severity and frequency of coastal erosion and bluff recession.

- Tides occur regularly with about two high tides and two low tides each day. The highest tides (spring tides) occur twice a month during the full moon and the new moon. Around December and January when a new or full moon occurs at the same time as the moon is at its closest to the earth, the tides run higher. These higher perigean spring tides are commonly known as King Tides.

- Storm surge can occur as a combination of wind shear over the water and low atmospheric pressure.

- El Niño (and La Niña) are cycles of warming and cooling of the ocean, typically lasting 9 to 12 months. They often commence in June or August and reach their peak during December through April, and subsequently decay over May through July of the following year. Their periodicity is irregular, occurring every 3 to 5 years on average. The warming associated with El Niño produces a rise of the ocean level, which can be on the order of 6 to 13 inches. The period of elevated (or lowered) ocean levels can be on the order of months, while the peak highs and lows occur on a scale of days to weeks.

- Elevated Groundwater Tables. Sea-level rise can cause seawater intrusion into coastal aquifer systems and can raise shallow groundwater tables. These can short circuit levee systems and contribute to inland flooding and/or impacts to buried infrastructure.

The historically highest water levels recorded around the Bay Area occurred in January of 1983 and were due to a combination of King Tides and rise of the ocean level due to a pronounced El Niño episode. Based on the tide station at San Francisco Golden Gate (NOAA Station 9414290) the estimated water level at South Ocean Beach would have been around +8.82 feet MLLW.

Table 2-6 provides a breakdown of tidal datums and extreme water levels for existing conditions, and estimated water levels with SLR projected for 2030, 2050, and 2100. The sea-level rise projection reflects the *Medium to High Risk Aversion* OPC Scenario, assuming *High Emissions*.

Table 2-6: Tidal and Extreme Water Level Datums, SLR Scenarios.

Condition [2]	Existing	Sea Level Rise (feet) by [1] 2030	2050	2100
		0.8	1.9	6.9
	Water Level (feet NAVD88)			
1% Annual Chance Storm	+8.7	+9.5	+10.6	+15.6
King Tides	+7.2	+8.0	+9.1	+14.1
MHHW	+5.9	+6.7	+7.8	+12.8
MHW (Shoreline)	+5.3	+6.1	+7.2	+12.2
MTL	+3.3	+4.1	+5.2	+10.2
MSL	+3.2	+4.0	+5.1	+10.1
MLW	+1.2	+2.0	+3.1	+8.1
MLLW	+0.1	+0.9	+2.0	+7.0

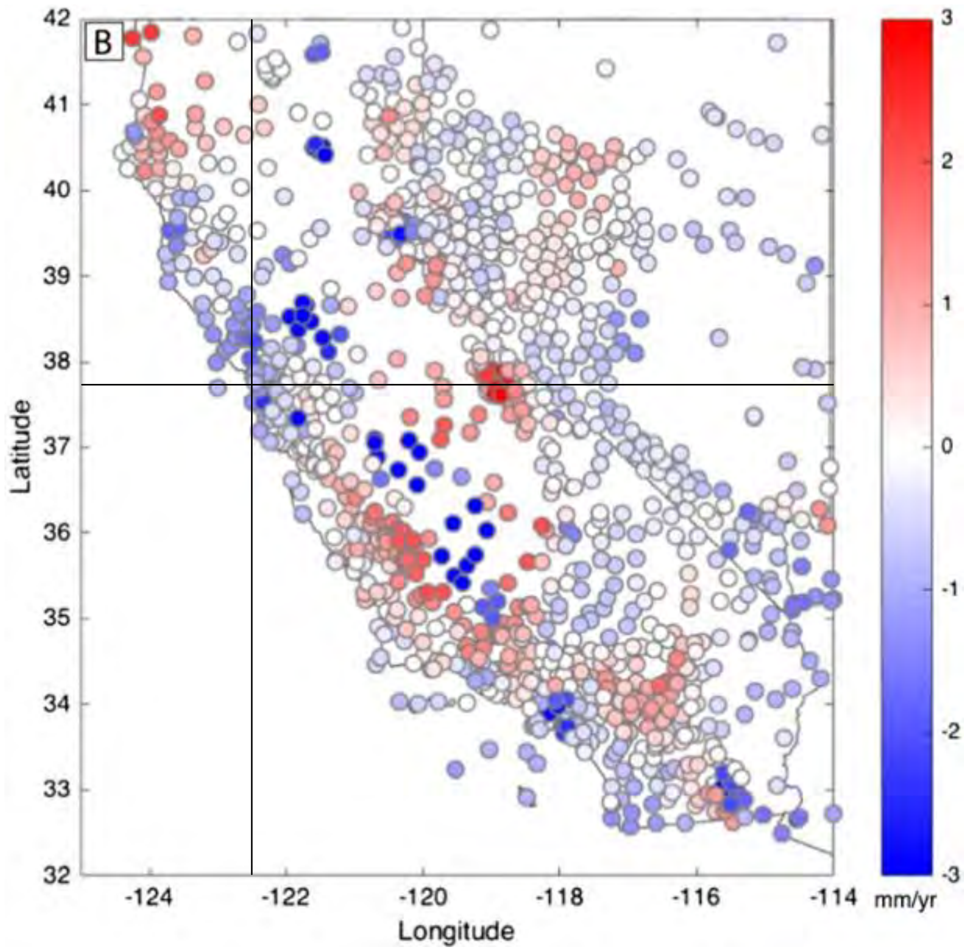
[1] State of California Sea-Level Rise Guidance, OPC (2018) Update.

2.9.1. Trends in Local Relative Sea Level

Local relative sea-level rise reflects the chance in sea-level due to climate change and vertical movement of the landmass. Vertical land motion (VLM) can occur due to tectonic activity, isostatic rebound which is adjustment of the earth due to compression from the ice masses during the last ice age, and due to subsidence.

Figure 2-29 shows estimates of vertical land motion (VLM) for California and Nevada from *JGR (2016)*. GPS imaging was employed to track vertical land motion data over a period of five years, accounting for groundwater withdrawal, elastic bedrock uplift and tectonic uplift. Red colors indicate uplift and blue colors indicate subsidence. The intersection of the black horizontal and vertical lines reflects the location of Ocean Beach is subsiding by 0.5 mm per year. At this rate the land will sink by 1.6 inches by 2100.

The vertical land motion in this case adds to the relative sea level rise at Ocean Beach, but the effect is limited as the projected rise in ocean level is an order of magnitude larger than the VLM.



-114
-125

81
113.4
11.34
4.464567

Figure 2-29: Rates of Vertical Land Motion, CA and NV, *JGR (2016)*.

2.10. Coastal Engineering Design

2.10.1. Tidal Datums and Extreme Water Levels

Table 2-7 summarizes tidal datums and extreme water levels for NOAA Station 9414290 – San Francisco referenced to North American Vertical Datum of 1988 (NAVD88).

Table 2-7: Tidal Datums and Extreme Water Levels.

Datum	El. Feet NAVD88	Comment
1% HWL	+8.69	1% Annual Chance (100-year) High Water Level
10% HWL	+8.20	10% Annual Chance (10-year) High Water Level
50% HWL	+7.68	50% Annual Chance (2-year) High Water Level
99% HWL	+7.05	99% Annual Chance (annual maximum) high water level
MHHW	+5.92	Mean Higher High Water
MHW	+5.31	Mean High Water
MTL	+3.26	Mean Tide Level
MSL	+3.20	Mean Sea Level
NGVD29	+2.72	National Geodetic Vertical Datum of 1929
MLW	+1.22	Mean Low Water
MLLW	+0.08	Mean Lower Low Water
NAVD88	0.00	North American Vertical Datum of 1988
99% LWL	-1.05	99% Annual Chance (annual minimum) low water level
50% LWL	-1.67	50% Annual Chance (2-year) Low Water Level
10% LWL	-2.03	10% Annual Chance (10-year) Low Water Level
1% LWL	-2.30	1% Annual Chance (100-year) Low Water Level

The Still Water Elevation (SWEL) most often used for coastal engineering design is the 1% HWL 100-year High Water Level indicated in Table 2-7.

2.10.2. Design High Water Level

The design water level on the beach is augmented by wave-driven processes, which include a setdown of the water due to waves shoaling (prior to breaking), a setup of the water level due to wave breaking in the surf zone, a setup of the water level due to surf beat, and any contribution from sea-level rise and vertical land motion as described in sections 2.8 and 2.9.1.

Wave-induced water level changes are described in the following.

The setdown, η_b, can be estimated via:

$$\eta_b = -\frac{1}{8}\frac{H_b^2\frac{2\pi}{L}}{sinh\left(\frac{4\pi h_b}{L}\right)}$$

Where H_b is the breaking wave height, h_b is the water depth at wave breaking, and L is the wave length. The wave setdown is on the order of 3-5% of the breaking wave height.

The wave setup in the surf zone, η_s, can be estimated as:

$$\eta_s = 0.27 H_{0s} (\xi_0)^{0.4}$$

Where H_{0s} is the deep-water significant wave height, and ξ_0 is the Iribarren number, relating wave breaking by beach slope α, deep-water significant wave height H_0 and deep-water wave length L_0 as follows:

$$\xi_0 = \frac{tan(\alpha)}{\sqrt{\frac{H_0}{L_0}}}$$

The wave-setup is slightly larger than the wave setdown in terms of magnitude, on the order of 6-7% of the breaking wave height along the beach and 10-11% of the offshore incident wave height.

Surf beat, ζ_{rms}, also produces a small increase of the water level across the surf zone, which can be estimated as:

$$\zeta_{rms} = \frac{0.01 H_0'}{\sqrt{\frac{H_0'}{L_0}\left(1 + \frac{h}{H_0'}\right)}}$$

Where h is the local water depth. The water level increase due to surf beat is on the order of 6% of the deep-water wave height.

The contribution of the above effects to the design water level is provided in Table 2-8.

Table 2-8: Design High Water Level.

Water Level and Wave Effects	Contribution to Design Water Level
Still Water Elevation	+8.69 feet NAVD88
Wave setdown	-1.61 feet
Wave setup	3.00 feet
Surf Beat	1.69 feet
Total (without sea-level rise)	+11.77 feet NAVD88

2.10.3. Wave Action

Figure 2-30 summarizes wave exposure along South Ocean Beach. Swell waves generated by distant storms in the North Pacific arrive from westerly and northwesterly directions (red arc), but the Point Reyes promontory at Drakes Bay moderates these waves. Secondary swell arrives from southwesterly directions (green arc). Local wind waves generated by storms passing over the region (yellow arc) can occur over the sector from south to north.

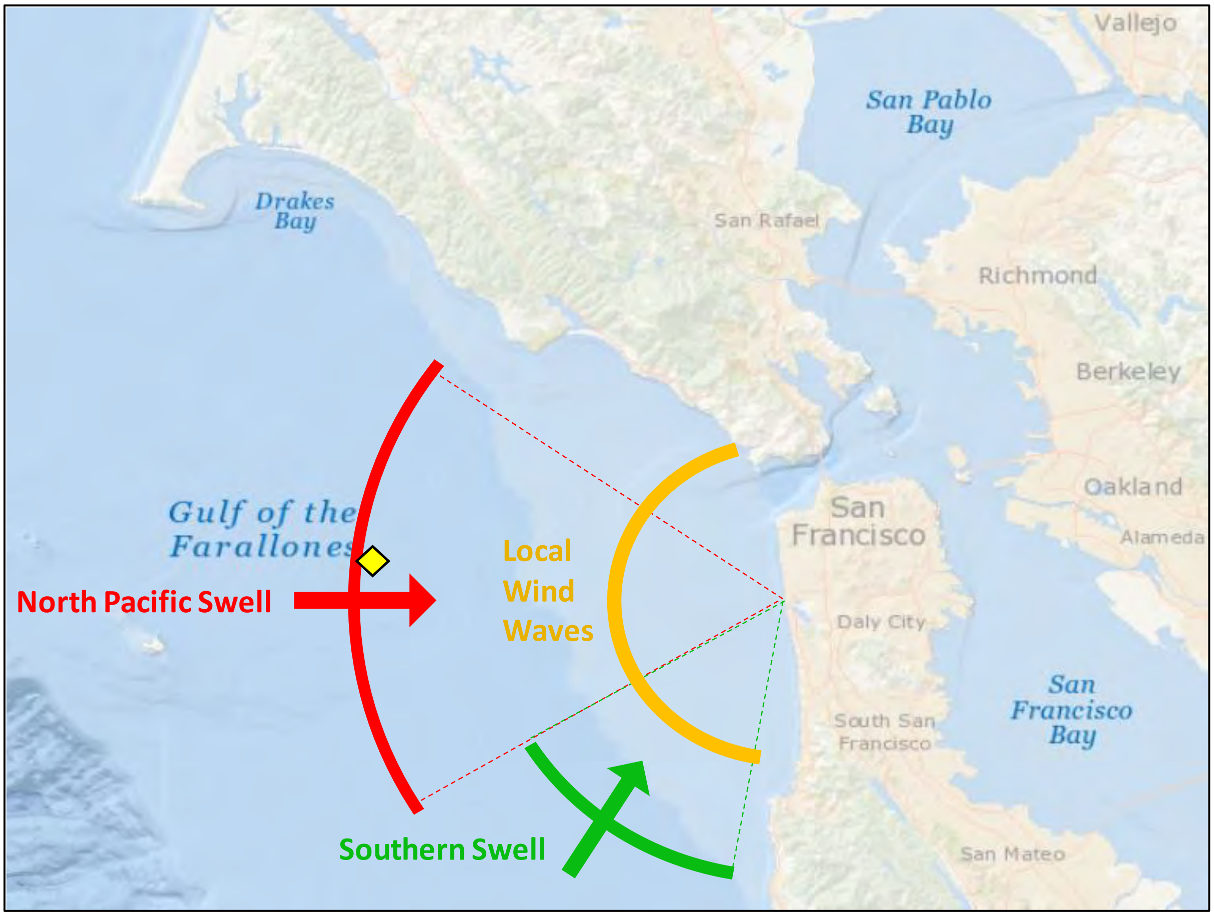


Figure 2-30: South Ocean Beach Wave Exposure.

The yellow diamond indicates NDBC wave buoy 46026 (LLNR 357), San Francisco. The wave buoy is located at: 37.755°N 122.839°W, approximately 18 nm west of San Francisco. Wave data on record captures a period of 37 years, from 1982 through 2018.

Table 2-9, Table 2-10, and Table 2-11 summarize the offshore annual average significant wave height distribution based on wave data from NDBC wave buoy 46026. Table 2-9 presents the data divided into percent occurrence of wave height by compass direction (from), and Table 2-10 by wave period and compass direction (from).

Table 2-9: Annual Average Distribution of Significant Wave Heights by Direction, NDCB Station 46026.

Significant Wave Height (ft)		Percentage of Occurrence Direction (From)																	
From	**To**	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**N**	**All**
0	2	0.00	-	-	-	-	-	0.00	0.01	0.04	0.06	0.05	0.03	0.04	0.03	0.02	0.00	-	0.279
2	4	0.00	0.02	0.01	0.01	0.01	0.02	0.04	0.23	1.49	3.13	2.15	1.25	3.30	5.82	2.94	0.17	0.01	20.587
4	6	0.01	0.01	0.01	0.00	0.01	0.01	0.02	0.12	0.68	1.24	0.85	1.26	6.50	14.66	7.55	0.25	0.01	33.165
6	8	0.00	0.00	-	-	-	0.00	0.00	0.03	0.24	0.26	0.19	0.55	5.45	13.12	4.62	0.06	-	24.522
8	10	-	-	-	-	-	-	-	0.01	0.12	0.12	0.11	0.29	3.32	7.33	1.88	0.01	-	13.192
10	12	0.00	-	-	-	-	-	-	0.00	0.07	0.07	0.04	0.12	1.47	3.10	0.70	0.00	-	5.573
12	14	-	-	-	-	-	-	-	0.01	0.07	0.05	0.01	0.05	0.57	0.92	0.22	0.00	-	1.889
14	16	-	-	-	-	-	-	-	-	0.03	0.02	0.00	0.04	0.17	0.24	0.03	-	-	0.527
16	18	-	-	-	-	-	-	-	0.00	0.01	0.01	0.00	0.02	0.06	0.07	0.00	-	-	0.176
18	20	-	-	-	-	-	-	-	-	0.00	0.00	-	0.00	0.03	0.01	-	-	-	0.048
20	22	-	-	-	-	-	-	-	-	0.00	-	0.00	0.00	0.01	0.01	-	-	-	0.028
22	24	-	-	-	-	-	-	-	-	-	-	-	-	0.00	0.00	-	-	-	0.006
24	26	-	-	-	-	-	-	-	-	-	-	-	-	0.01	-	-	-	-	0.005
26	28	-	-	-	-	-	-	-	-	-	-	-	-	0.00	0.00	-	-	-	0.003
28	30	-	-	-	-	-	-	-	-	-	-	-	-	-	0.00	-	-	-	0.001
Total		**0.01**	**0.02**	**0.01**	**0.01**	**0.02**	**0.02**	**0.06**	**0.40**	**2.75**	**4.97**	**3.41**	**3.61**	**20.92**	**45.32**	**17.96**	**0.49**	**0.01**	**100.00**

Predominant wave heights range from 2 to 10 feet (significant wave height), with incidence from westerly to northwesterly directions (91% of all waves). Waves are very rarely lower than 2 feet in height (0.28% of all waves), and infrequently exceed 10 feet in height (8.25% of all waves). The largest wave on record was 28.2 feet, incident from west-northwest.

Table 2-10 shows the annual average distribution of peak wave periods by significant wave height. The data cover waves with periods from 1 to 23 seconds. Local storms passing over the region produce waves with periods less than 8 seconds, while waves with longer wave periods characterize swell originating from distant storm systems in the Pacific Basin.

Table 2-10: Distribution of Significant Wave Height and Peak Wave Periods, NDCB Station 46026.

Significant Wave Height (ft)		Percentage of Occurrence Peak Wave Period (seconds)												
From	**To**	**1**	**3**	**5**	**7**	**9**	**11**	**13**	**15**	**17**	**19**	**21**	**23**	**All**
0	2	-	-	-	0.02	0.02	0.02	0.15	0.05	0.03	-	-	-	0.279
2	4	-	0.05	0.87	1.64	3.69	3.64	4.78	2.86	2.71	0.25	0.10	0.01	20.587
4	6	-	0.00	1.21	4.99	6.36	8.20	6.42	2.15	3.16	0.53	0.13	0.02	33.165
6	8	-	-	0.25	3.78	4.39	5.48	6.87	1.52	1.71	0.40	0.10	0.03	24.522
8	10	-	-	0.02	1.43	2.15	2.37	4.33	1.26	1.32	0.23	0.07	0.01	13.192
10	12	-	-	0.00	0.29	0.92	0.96	1.77	0.74	0.74	0.11	0.03	0.01	5.573
12	14	-	-	-	0.05	0.29	0.38	0.50	0.25	0.36	0.05	0.01	0.00	1.889
14	16	-	-	-	0.01	0.05	0.09	0.13	0.07	0.15	0.02	0.00	-	0.527
16	18	-	-	-	-	0.02	0.04	0.04	0.02	0.07	0.01	-	-	0.176
18	20	-	-	-	-	0.01	0.00	0.00	0.01	0.03	0.01	-	-	0.048
20	22	-	-	-	-	0.00	0.00	0.00	0.00	0.02	0.01	-	-	0.028
22	24	-	-	-	-	-	-	-	-	0.01	0.00	-	-	0.006
24	26	-	-	-	-	-	-	-	-	0.00	0.00	-	-	0.005
26	28	-	-	-	-	-	-	-	-	-	0.00	-	-	0.003
28	30	-	-	-	-	-	-	-	-	0.00	-	-	-	0.001
Total		**0.00**	**0.05**	**2.34**	**12.22**	**17.89**	**21.18**	**24.98**	**8.92**	**10.31**	**1.61**	**0.43**	**0.07**	**100.00**

Table 2-11 shows the distribution of wave heights over the months of the year. Inspection of the wave data reveals that the most severe wave conditions typically occur in January.

Table 2-11: Distribution of Waves by Month, NDCB Station 46026.

Significant Wave Height (ft)		Percentage of Occurrence by Month												
From	To	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	All
0	2	-	0.00	0.02	-	0.03	0.00	0.01	0.12	0.01	0.00	0.08	0.01	0.279
2	4	0.91	0.40	0.73	0.91	2.00	1.90	3.59	3.67	2.57	1.94	1.35	0.62	20.587
4	6	2.13	1.66	2.23	2.41	3.04	3.13	3.51	3.01	3.97	3.48	2.40	2.22	33.165
6	8	2.06	1.94	2.43	2.20	2.24	2.46	1.76	0.81	1.67	2.14	2.55	2.25	24.522
8	10	1.43	1.58	1.65	1.42	1.11	1.11	0.31	0.14	0.34	1.06	1.55	1.48	13.192
10	12	0.58	0.72	0.87	0.73	0.40	0.29	0.02	0.04	0.05	0.51	0.68	0.68	5.573
12	14	0.29	0.35	0.25	0.25	0.09	0.07	-	0.01	0.00	0.09	0.23	0.25	1.889
14	16	0.11	0.09	0.08	0.06	0.01	-	-	0.00	-	0.00	0.06	0.11	0.527
16	18	0.04	0.03	0.02	0.02	-	-	-	-	-	-	0.02	0.05	0.176
18	20	0.02	0.01	0.00	0.01	-	-	-	-	-	-	-	0.02	0.048
20	22	0.01	0.01	-	-	-	-	-	-	-	-	-	0.01	0.028
22	24	0.00	0.00	-	-	-	-	-	-	-	-	-	0.00	0.006
24	26	0.01	-	-	-	-	-	-	-	-	-	-	-	0.005
26	28	0.00	-	-	-	-	-	-	-	-	-	-	-	0.003
28	30	0.00	-	-	-	-	-	-	-	-	-	-	-	0.001
Total		**7.58**	**6.80**	**8.28**	**8.01**	**8.92**	**8.96**	**9.20**	**7.79**	**8.60**	**9.22**	**8.92**	**7.72**	**100.00**

Table 2-12 provides estimates of offshore significant wave height extremes based on extreme-value analysis (EVA) of the wave data from NDBC Station 46026.

Table 2-12: Significant wave height extremes, NDBC Station 46026

Return Period (years)	Offshore Significant Wave Height (feet)	90% Confidence Interval	
		Lower Bound	Upper Bound
5	22.9	20.9	24.8
10	25.4	22.9	28.0
50	31.1	27.2	35.0
100	33.5	29.0	38.0

Wave transformation by refraction and shoaling occur over the complex bathymetry around the San Francisco Bar, but note that the larger storm waves become depth-limited and will break and reform in the fairly wide surf zone at South Ocean Beach. The governing design wave for the low-profile wall alternative is therefore the maximum breaking wave supported by the design water depth at the wall.

2.10.4. Scour Elevations

Potential scour at the toe of the wall is assessed in the following. Using the method of Fowler (1992), the maximum scour depth can be estimated as:

$$S_{max} = H_0 \sqrt{\frac{22.72 \cdot d_s}{L_0} + 0.25}$$

Where H_0 is the zero moment wave height, L_0 is the deep water wave length, and d_s is the pre-scour water depth at the wall. This method estimates a toe scour elevation of approximately +1.4 feet NAVD88.

XBeach simulation of beach profile variation and scour at the toe of a seawall (Figure 2-31) confirms this result.

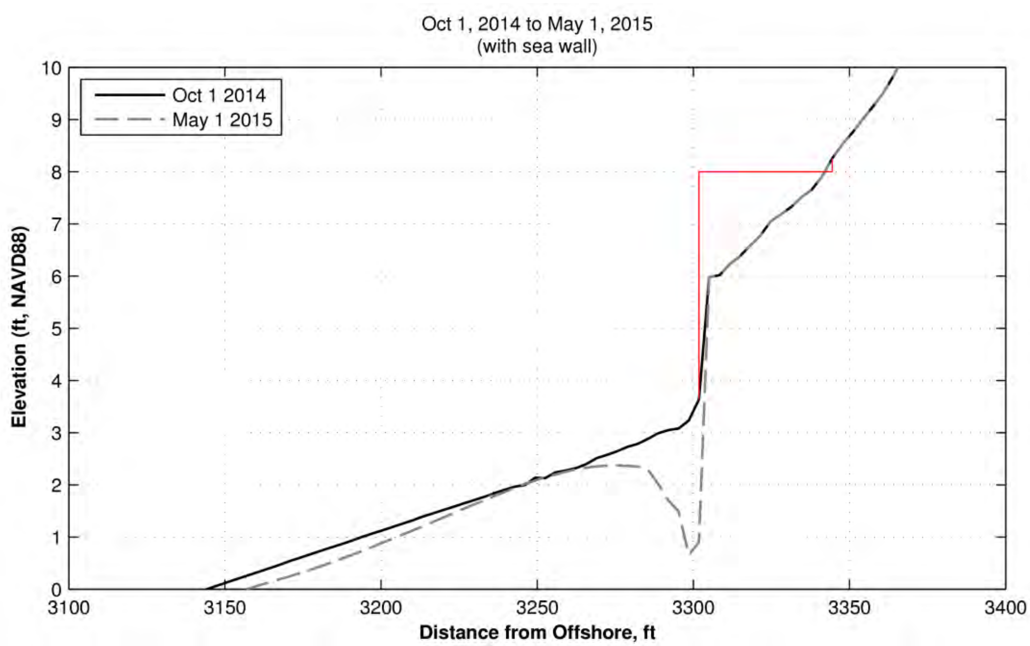


Figure 2-31: Toe scour elevation determined from XBeach modeling.

2.10.5. Wave Runup

Estimated elevations of wave runup on the slope above the crest of the wall are summarized in Table 2-13. The first column of elevations identifies the wall crest elevation, which transitions over the reaches between STA 12+20 and STA 42+60. The subsequent columns indicate the wave runup elevations for no sea-level rise followed by sea-level rise in increments of two feet.

Table 2-13: Wave Runup Elevations for Project Reach Segments.

<table>
<tr><th rowspan="3">Station</th><th rowspan="3">Segment</th><th colspan="5">Elevation (feet NAVD88)</th></tr>
<tr><th rowspan="2">Wall Crest</th><th colspan="4">Wave Runup at Crest</th></tr>
<tr><th>No SLR</th><th>0.8’ SLR</th><th>1.9’ SLR</th><th>6.9’ SLR</th></tr>
<tr><td>12+20</td><td rowspan="2">North Reach</td><td>+14.50</td><td>+21.5</td><td>+22.5</td><td>+24.0</td><td>+30.4</td></tr>
<tr><td rowspan="2">19+45</td><td rowspan="2">+15.50</td><td rowspan="2">+21.3</td><td rowspan="2">+22.2</td><td rowspan="2">+23.6</td><td rowspan="2">+30.2</td></tr>
<tr><td rowspan="2">EQR Reach</td></tr>
<tr><td rowspan="2">24+55</td><td rowspan="2">+16.10</td><td rowspan="2">+21.2</td><td rowspan="2">+22.1</td><td rowspan="2">+23.5</td><td rowspan="2">+30.0</td></tr>
<tr><td rowspan="2">Rubble Reach</td></tr>
<tr><td rowspan="2">33+70</td><td rowspan="2">+17.10</td><td rowspan="2">+21.0</td><td rowspan="2">+21.9</td><td rowspan="2">+23.2</td><td rowspan="2">+29.7</td></tr>
<tr><td rowspan="2">Bluff Reach</td></tr>
<tr><td rowspan="2">36+65</td><td rowspan="2">+17.75</td><td rowspan="2">+21.0</td><td rowspan="2">+21.9</td><td rowspan="2">+23.1</td><td rowspan="2">+29.5</td></tr>
<tr><td rowspan="2">South Reach</td></tr>
<tr><td>42+60</td><td>+18.50</td><td>+21.1</td><td>+21.8</td><td>+23.0</td><td>+29.4</td></tr>
</table>

2.10.6. Wave Loads

Wave loads on the low-profile wall were estimated based on *ASCE (2016).*

Figure 2-33 shows how ASCE 7-16 defines the breaking wave load on a wall as the sum of a hydrostatic pressure and a dynamic pressure component.

This approach assumes that waves incident at the wall are depth-limited and breaking. In this case, 70% of the wave crest is located above the stillwater level. The maximum depth-limited breaking wave height, H_b, is given by:

$$H_b = 0.78d_{100}$$

Where d_{100} is the 100-year stillwater depth. The crest elevation of the incident wave is consequently: $\eta_c = SWEL + 70\%H_b = 0.7 \cdot 0.78d_{100} = 0.55d_s$.

The ASCE 7-16 approach additionally assumes that the wall is fully reflective so that the incident wave height is reflected without any reduction in wave height. By superposition, the wave height at the wall is therefore the sum of the incident wave and the reflected wave (the wave height at the wall effectively doubles), i.e. $2 \cdot 0.55d_s \cong 1.2d_s$.

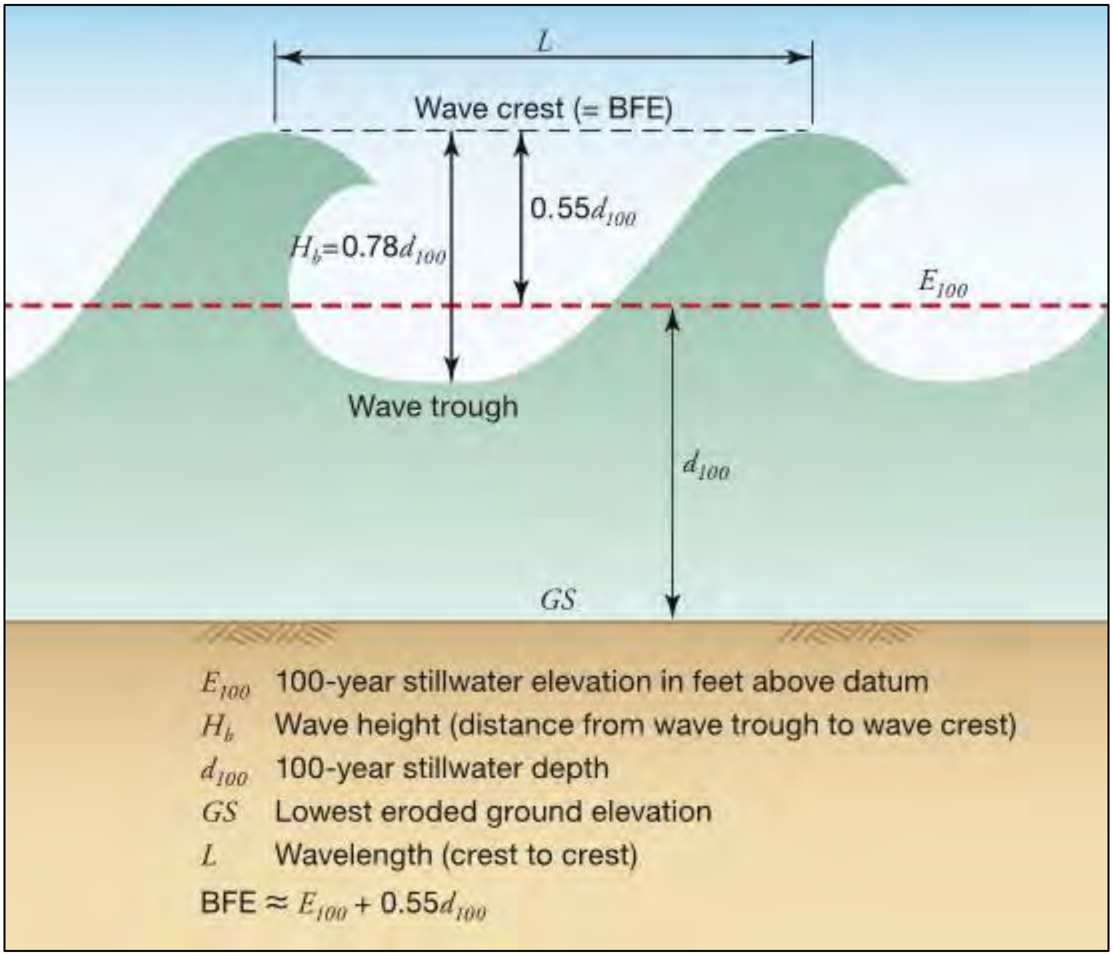


Figure 2-32: Definition of BFE for Breaking Wave Conditions, *FEMA (2011)*.

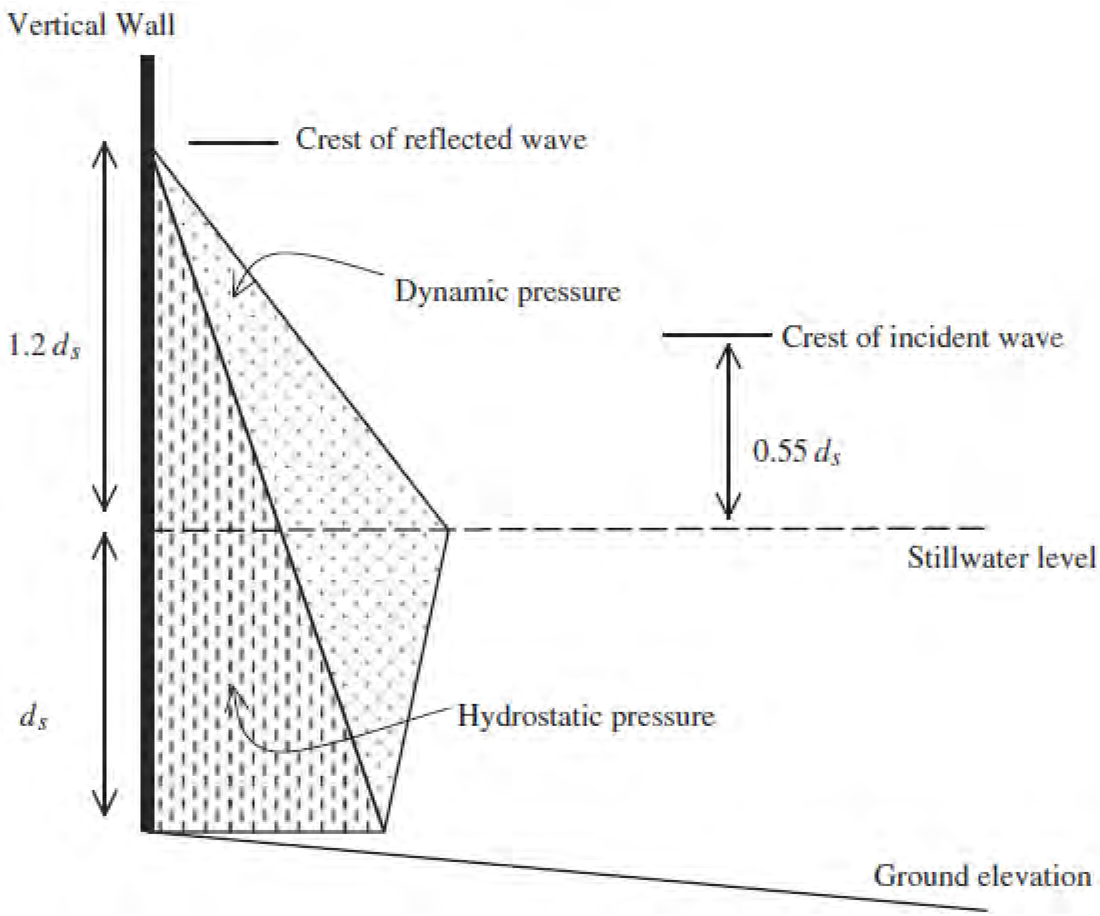


Figure 2-33: Normally Incident Wave Breaking Pressures, *ASCE (2016).*

The maximum combined dynamic and static pressure, P_{max}, is given by:

$$P_{max} = C_p \gamma_w d_s + 1.2 \gamma_w d_s$$

Where: C_p is a dynamic pressure coefficient, γ_w is the unit weight of water, and d_s is the still water depth at the base of the wall. The estimated maximum pressure is: P_{max} = 10.5 psi.

The breaking wave force per unit length of wall, F_t, is given by:

$$F_t = 1.1 C_p \gamma_w d_s^2 + 2.4 \gamma_w d_s^2$$

The estimated breaking wave force is: F_t = 19,0 kip/ft.

2.10.7. Scour at Wall Crest

An analysis was conducted to assess the spatial extent of wave overtopping past the crest of the low-profile wall with respect to sea-level rise, and the potential for scouring behind the wall if the crest is not protected.

The results of the analysis are summarized in Figure 2-34, which shows that substantial scour behind the wall could develop if the slope at the crest is not protected. For the scenarios with 1.9’ to 6.9’ of sea-level rise (SLR), it is estimated that the ground level behind the wall could erode down to approximately El. 0.0 feet NAVD88 and expose the LMT. Progressive erosion would be noted from present day to 0.8’ of SLR. It is therefore imperative that the slope above the crest of the wall be protected to prevent loss of cover material over the LMT and potential undermining of the coastal trail at the crown of the slope. The estimated spatial extent of wave overtopping is about 15 feet for 0.8’ of SLR, 30 feet for 1.9’ of SLR, and 45 feet for 6.9’ of SLR.

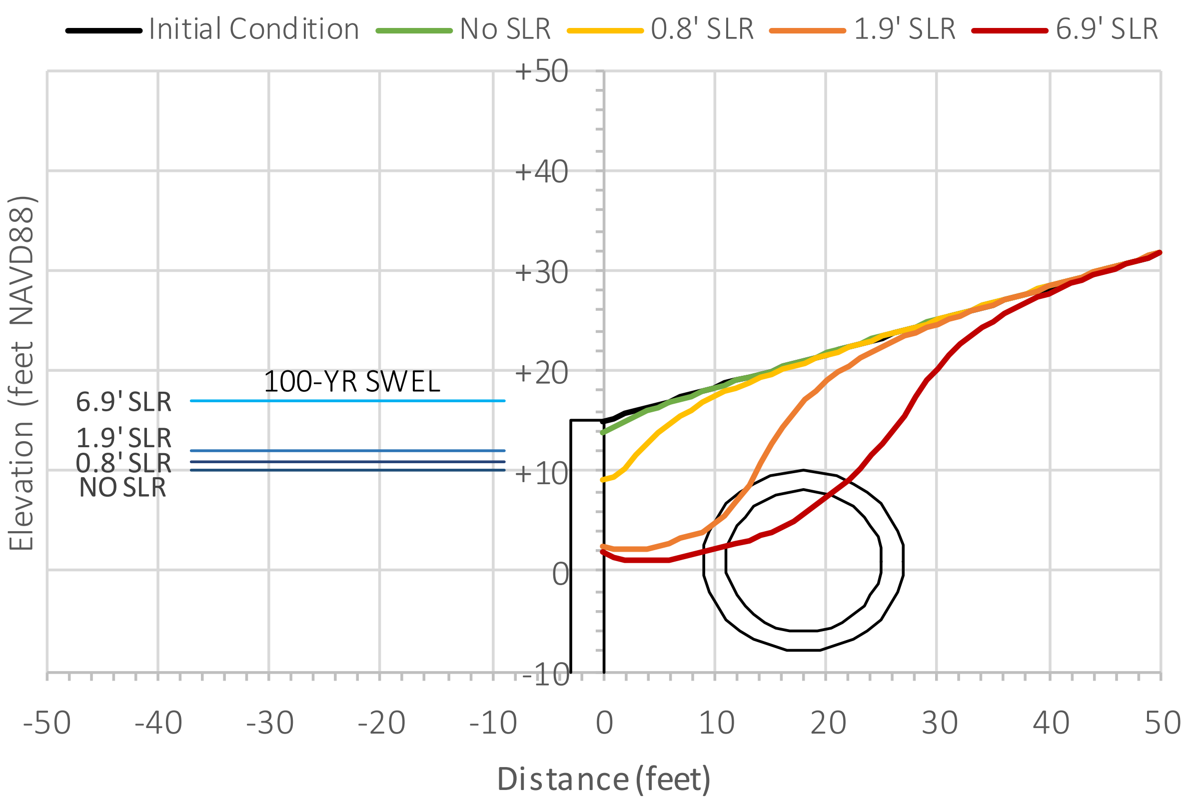


Figure 2-34: Assessment of Potential Scour of Slope above Wall Crest (No Slope Protection).

2.10.8. Beach Nourishment

Beach nourishment serves to protect the Lake Merced Tunnel, the Great Highway, and coastal bluff from the effects of storms by building a beach, which acts as a buffer. Although beach nourishment is one of the most commonly-performed activities seen on the coast, predicting its effectiveness is a significant undertaking because of uncertainties in the frequencies of storms and the subsequent effects after sand is transported away from the nourished reach.

In this study, a high-level desktop analysis was performed to approximate the quantity and frequency of beach nourishment required for the project under RCP8.5 *Medium – High Risk Aversion* SLR projection *(OPC, 2018*). Typically, beach width was used as the indicator for beach nourishment. Factors that affect beach width may include beach nourishment (+) and shoreline erosion or recession (-). The positive sign indicates an increase while the negative sign indicates a decrease in beach width.

The planform evolution of the beach profile can be estimated using the Pelnard-Considère equation (Pelnard-Considère 1956; Rosati et al 2002). This equation describes the shoreline evolution in terms of a one-line diffusion model. The basic model equation is:

$$\frac{\partial y}{\partial t} = G\frac{\partial^2 y}{\partial x^2}$$

where *y* is the shoreline position at a distance *x* alongshore and *G* is the longshore diffusivity:

$$G = \frac{K{H_b}^{2.5}\sqrt{g/\gamma}}{8(s-1)(1-p)(h_c+B)}.$$

In this equation, *K* is a sediment transport coefficient associated with median grain size (i.e. 0.25 mm to 0.35 mm per Moffatt & Nichol 1995; Barnard and Hanes 2006); H_b is the breaker wave height; *g* is the acceleration due to gravity; γ is the ratio of water depth to breaker wave height, typically about 0.78; *s* is the sediment specific gravity; *p* is the sediment porosity about 0.4; h_c is the closure depth (i.e. -35' MLLW per Moffatt & Nichol 1995); and *B* is the beach berm crest elevation. Overall, this is a diffusion model – meaning that the tendency is for the beach planform to flatten out. If the wave energy is constant along the shoreline, the model predicts a final condition in which the shoreline can be described as a straight line.

In addition, a long-term historical shoreline erosion rate of 2 feet per year was estimated for the project area (USACE 1996; USGS 2006). This rate of shoreline erosion is coupled with the loss due to sea-level rise, in which the Bruun Rule was applied (detailed in Section 2.8.1).

Figure 2-35 presents beach width variations for a compound beach nourishment scenario assessed in the study. The scenario assumes 125,000 CY of sand are placed along the entire project area every 5 years before Year 2060. After Year 2060, additional 40,000 CY (e.g. a total of 165,000 CY) of sand are required every 5 years to keep pace with the adopted RCP8.5 SLR projection.

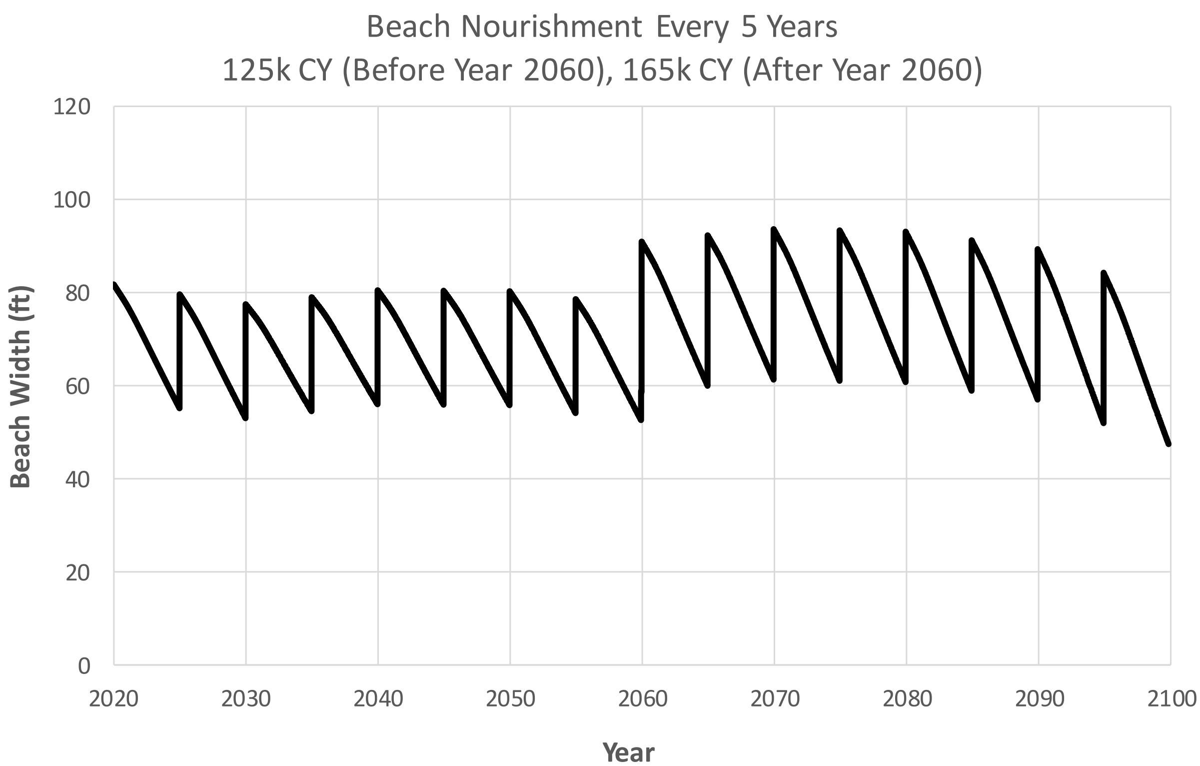


Figure 2-35: Beach Width Variation with a Compound Beach Nourishment Scenario under RCP8.5 Medium/High Risk Aversion SLR Projection.

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Appendix A-1
Wentworth Grading Scale

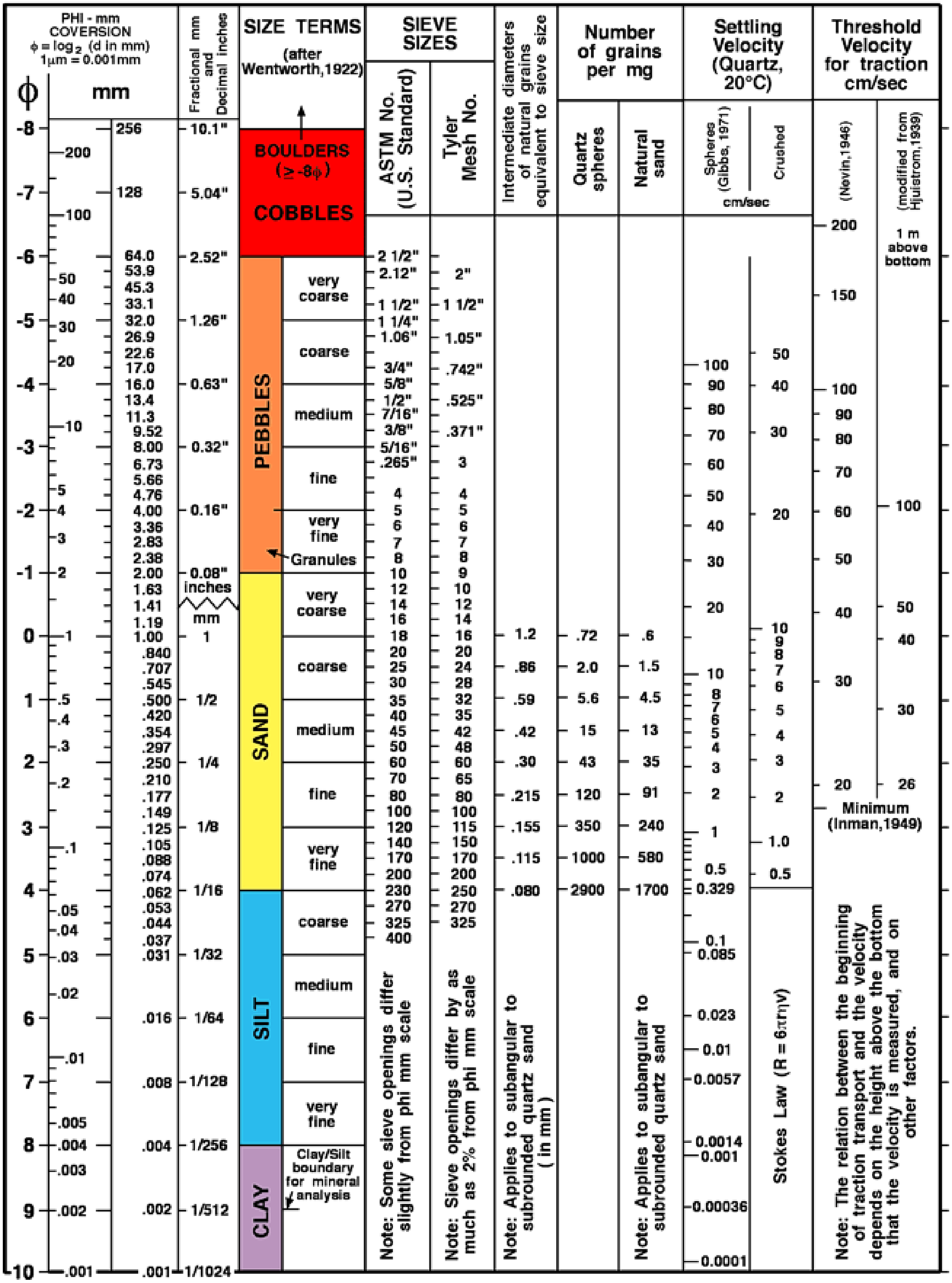

Figure A-1: Wentworth Grading Scale.

Appendix B:
Geotechnical Assessment Report

DRAFT

GEOTECHNICAL ASSESSMENT REPORT

CONCEPTUAL ENGINEERING REPORT (CER) PHASE

SOUTH OCEAN BEACH COASTAL EROSION AND

WASTEWATER INFRASTRUCTURE PROTECTION

SAN FRANCISCO, CALIFORNIA

CONTRACT PRO.0092

AGS Job No. AGS-18-003

Prepared for:

SAN FRANCISCO PUBLIC UTILITIES COMMISSION

Prepared by:

AGS

APRIL 25, 2019

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TABLE OF CONTENTS

TABLE OF CONTENTS

TABLE OF CONTENTS

TABLE OF CONTENTS

1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical assessment conducted by AGS, Inc. (AGS) for the Conceptual Engineering Report (CER) phase of the proposed South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection Project in San Francisco, California. The project site is located within the Great Highway alignment, between Sloat and Skyline Boulevards as shown on Plate 1 – Site Vicinity Map and Plate 2 – Site Location Plan.

The purpose of this report is to provide our initial findings and preliminary geotechnical recommendations for CER evaluations. Concurrently, AGS is conducting a geotechnical study for this project. The results of our geotechnical study will be presented in a Geotechnical Data Report (GDR) and a Geotechnical Interpretative Report (GIR).

1.2 PROJECT DESCRIPTION

The proposed South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection Project primarily include the following two elements:

1. Structural protection of the Lake Merced Transport (LMT); and
2. Improvement of the shoreline conditions.

Our geotechnical study for this project is focused on the protection of the LMT. We understand that the goal of our study of the shoreline is to investigate the conditions and engineering properties of the native bluff materials at the beach.

We also understand that the Ocean Beach Master Plan (dated May 2012) would allow for coastal retreat, including eventual rerouting of the Great Highway behind the zoo via Sloat and Skyline Boulevards. A new access road including two 12-foot wide lanes plus two 4-foot wide shoulders for the Oceanside Water Pollution Control Plant (OWPCP) and an approximately 16-foot wide coastal trail along the seaward side (west) of the new access road would be constructed.

The Long-Term LMT Protection Feasibility Study Report (Moffat & Nichol, February 2015), which was part of the Ocean Beach Master Plan: Coastal Management Framework (dated April 2015), outlined a concept of protecting the tunnel with a low-profile wall. This low-profile wall would be constructed below grade on the seaward side of the LMT. The offset of the low-profile wall from the seaward edge of the tunnel would be about 16 feet, except for the northern part adjacent to the existing Sloat beach access parking lot, where the offset would be increased to about 42 feet.

We understand that the preferred option for the low-profile wall is a secant pile wall with tiebacks. The current concept plans (dated April 1, 2019) indicate that the project site is subdivided into five reaches, distinguished by erosion characteristics and type of erosion control installed at each one. The top of wall elevations and the offset from the seaward edge of the LMT would vary at the five reaches as shown in Table 1.

TABLE 1
REACH DESCRIPTIONS

Reach	Stationing	Top of Wall Elevation[1] (feet)	Minimum Lateral Offset from LMT (feet)	Description
North	10+80 to 19+55	+14.50 to +15.50	42	This reach includes the National Park Service Facilities, including a public parking lot and restrooms. A sandbag revetment was installed in 2010
EQR	19+55 to 24+55	+15.50 to +16.10	16	Emergency Quarrystone Revetment. This was constructed in 1997-1999
Rubble	24+55 to 33+70	+16.10 to +17.10	16	An un-engineered shoreline protection structure consisting of concrete rubble. Rubble is limited to the lower beach levels and there are over-steepened cliffs in this area
Bluff	33+70 to 36+65	+17.10 to +17.75	16	Rubble is limited to the lower beach levels. There are over-steepened cliffs in this area.
South	36+65 to 42+60	+17.75 to +18.50	16	The site of the 2010 Emergency Bluff Toe Protection, a large quarrystone revetment

The secant pile wall would consist of overlapping unreinforced and reinforced drilled, cast-in-place concrete piles (called “primary unreinforced” and “secondary reinforced” piles, respectively) installed at approximately 5-foot center-to-center spacing. Both the primary unreinforced and secondary reinforced piles would be approximately 3 feet in diameter. The

[1] Elevations in this study are based on NAVD88, unless otherwise noted.

primary unreinforced piles would be drilled first and filled with concrete, followed by the secondary reinforced piles drilled between and partially cutting into the primary piles. The toe of the primary unreinforced piles would be set at approximately Elevation -10 feet. The secondary reinforced piles would be extended to greater depths as determined by structural analysis. A 4-foot square continuous pile cap would be constructed for the secant pile wall with the top set at an elevation approximately 6 feet above the crown of the LMT tunnel. As currently planned, tiebacks spaced every 10 feet would be installed at inclination of 1½ horizontal to 2 vertical (1½H:2V) downward from the pile cap to provide additional lateral support.

Initially, the secant pile wall would be concealed. However, over time, as beach recession occurs, the secant pile wall would be exposed (with the seaward side lowered to Elevation +2 feet in front of the wall). Ultimately, the landward side of the secant pile wall would become a 3 horizontal to 1 vertical (3H:1V) backslope. For erosion protection of the ultimate 3H:1V backslope against extreme wave runup in the future, an approximately 4 feet thick layer of either controlled low strength material (CLSM) or soil-cement mix would be constructed as a cover for the ultimate 3H:1V backslope (called "ultimate backslope cap").

1.3 REVIEW OF EXISTING DATA

The project area has been the subject of several previous geotechnical studies listed below. Available data from the previous studies have been compiled and used to provide a basis for our study approach and develop our field exploration program for this study.

- Geotechnical Report, Westside Pump Station Reliability Improvements, San Francisco, California, by GTC, Inc., 2016.
- Draft Report Geotechnical Study, Slope Stability Hazard Evaluation, Great Highway Stabilization, San Francisco, California, AGS. Inc., 2010.
- Preliminary Engineering Study, Lake Merced Tunnel, The Great Highway, San Francisco, California, Treadwell & Rollo, 2002.
- Lake Merced Transport Tunnel Geotechnical Design Summary Report, Parsons Brinckerhoff Quade & Douglas, Inc., 1990.
- Geotechnical Data Report, Lake Merced Transport, San Francisco, California, AGS, Inc., 1989.
- Preliminary Geotechnical Investigation, Lake Merced Transport Project, San Francisco, California, Harding-Lawson Associates, 1981.
- Geotechnical Engineering Evaluation, Southwest Ocean Outfall Project, Woodward-Clyde Consultants, 1977.
- Geologic Exploration Studies, Southwest Ocean Outfall Project, Woodward-Clyde Consultants, 1977.
- Review and Evaluation of Existing Data, Southwest Ocean Outfall Project, Woodward-

Clyde Consultants, 1977.

- Preliminary Report, Offshore Geophysical Survey, Southwest Ocean Outfall Project, Woodward-Clyde Consultants, 1977.
- Onshore Seismic Refraction Survey, Southwest Ocean Outfall Project, Woodward-Clyde Consultants, 1977.
- West Side Transport Soil Investigation Phase I, Harding-Lawson Associates, 1976.

1.4 FIELD EXPLORATION

The field exploration program performed by AGS include geotechnical borings, monitoring well installation, cone penetrometer test soundings, a geophysical survey, and vacuum potholing.

1.4.1 Geotechnical Borings

Geo-Ex Subsurface Exploration of Dixon, California, completed six borings up to 100 feet depth during February 11-15th, 2019 and March 5-6th, 2019. These borings were drilled with a CME-55 track-mounted drill rig using solid flight continuous sampling for the upper 20 feet of Borings B-3 through B-4, and rotary wash drilling method was incorporated for the remainder of the borings below 20 feet depth. Borings were backfilled with cement grout.

1.4.2 Monitoring Well Installation

Geo-Ex Subsurface Exploration installed three monitoring wells up to 30 feet depth using solid stem drilling method. The screened interval of the wells was 2.5 feet to 27 feet depth, on March 7th, 2019. The wells were covered with a traffic-rated Christy box.

1.4.3 Cone Penetrometer Test (CPT) Soundings

Gregg Drilling of Martinez, California, completed fourteen CPT soundings to a maximum depth of 100 feet during February 11th to 15th and March 1st, 2019. Two of the CPT soundings included seismic piezocone soundings at 5 foot intervals to obtain shear wave velocity of the soil.

1.4.4 Geophysical Survey

Southwest Geophysics of San Diego completed four Multi-Channel Analysis of Surface Waves (MASW) lines on February 27-28th, 2019. One 800-foot line, ML-1, was completed in the vicinity of the Southwest Ocean Outfall (SWOO) and three 150 foot-lines were completed, one survey line each in the following locations: ML-2 in the Rubble Reach, ML-3 in the EQR Reach, and ML-4 in the North Reach.

1.4.5 Vacuum Potholing

Badger Daylighting of San Jose completed three vacuum potholes on March 8th, 2019 and three additional potholes on March 28-29th, 2019. The top of the LMT was not located in the first three potholes due to the holes collapsing. However, two of the three potholes attempted on March 28-29th identified the top of the tunnel. The tunnel was tagged at 20.5 feet in the PH-5A (to the east) and at 20 feet in PH-5B (to the west). The third pothole, PH-6A, collapsed at about 6 feet depth.

1.5 LABORATORY TESTING

A geotechnical laboratory-testing program was conducted on the samples obtained from the field to determine their physical and mechanical characteristics. Tests included moisture content, dry density, sieve and hydrometer, Atterberg Limits, triaxial compression unconsolidated-undrained testing, and corrosivity. Thirteen samples from representative soil layers were sent for X-Ray Diffraction (XRD) and thin-layer petrographic testing, and we expect results to be included in the upcoming Geotechnical Data Report (GDR) and Geotechnical Interpretive Report (GIR).

1.6 EVALUATION CRITERIA

This geotechnical assessment is performed for the CER phase of the proposed South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection Project. For the CER phase, our initial findings and preliminary geotechnical recommendations that include the following have been developed for CER evaluations of this project:

- Site conditions;
- Geologic setting;
- Faulting and seismicity;
- Subsurface soil conditions;
- Groundwater conditions;
- Geologic and seismic hazards;
- Design groundwater level;
- Sequencing considerations;
- Secant pile wall;
- Lateral earth pressures;
- Buoyancy resistance;

- Tiebacks;
- Soil-cement mix cap;
- Site clearing;
- Excavation and temporary shoring;
- Dewatering during construction;
- Earthwork; and
- Soil corrosivity.

1.7 CODES AND STANDARDS

AGS understands that the following codes and standards are applicable to our geotechnical study for this project:

- American Society of Civil Engineers Standard 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7-16);
- 2019 California Building Code (CBC); and
- San Francisco Public Utilities Commission General Seismic Requirements for Design of New Facilities and Upgrade of Existing Facilities, Revision 3, June 2014 (SFPUC GSR 2014).

2.0 INITIAL GEOTECHNICAL FINDINGS

2.1 SITE CONDITIONS

2.1.1 Surface Conditions

The project alignment is located along the Great Highway, starting at Sloat Boulevard and continuing towards Skyline Boulevard to the south. The Westside Pump Station and the beach access parking lot are located at the northern end of the alignment, and the San Francisco Zoo is located west of the alignment. The southern end of the project alignment is approximately 1,000 feet south of the Oceanside Water Pollution Control Treatment Plant (OSP), which is located on the eastern side of the northbound lanes of the Great Highway. The project location is presented on Plate 1, Site Vicinity Plan, and Plate 2 – Site Location Plan.

Topographically, the site ground elevation is approximately +30 feet in the northern two-thirds of the site, from Sloat Boulevard at approximately station 12+00 to station 32+00. From station 32+00 onward, the site ground elevation gradually increases to about +70 feet at 500 feet north of Skyline Boulevard. Then, the site ground elevation decreases to approximately +50 feet at Skyline Boulevard.

Seaside bluffs in various stages of erosion, and some rip-rap stabilized seaside slopes, are located between 30 feet to 70 feet seaward of the western edge of the Great Highway. The bluffs range in height from 20 to 25 feet at the northern end of the site, up to greater than 50 feet in height at the southern end of the site. Based on our review of Google Earth and the topographic survey provided to us by the SFPUC (2015), these bluffs are sloping at approximately 3.5H:1V in the northern end of the site to 1.75H:1V in the southern end of the site.

A total of approximately 1,600 lineal feet of riprap improvements were installed along three reaches of the alignment in 2010 after the El Niño storm events of 2009-2010, which caused continued erosion and collapse of portions of the Great Highway. The supporting bluffs slipped out in some areas and the southbound lane was undermined and the pavement collapsed. At several locations along the alignment, the pavement of the former alignment of the Great Highway and its associated beach access parking lots, now decommissioned, are remaining on site, and overhanging on over-steepened slopes. A concrete k-rail barrier separates the abandoned southbound roadway from the current southbound lane.

2.1.2 Site History

Historical records maintained by the City and County of San Francisco (CCSF) indicate the Ocean Beach shoreline has been subject to episodic retreat, but that recent bluff retreat is concentrated in the area south of Sloat Boulevard, particularly since the mid 1990s. Records and historic shoreline changes were compiled and summarized in studies by the US Army Corps of Engineers, (1992, 1996), Moffatt & Nichol (1995), CH2M Hill (1996), Treadwell and Rollo (2002), and the United States Geological Survey (USGS, 2005, 2012). These studies complement recent surveys of the beach and coastal bluffs by the City and County of San Francisco (CCSF) and others, to determine the extent of recent bluff failure and evaluate the contributing factors.

In the 1800s, the project site was part of an embayment leading to the outlet from Lake Merced, according to USGS historical topographic maps (1859, 1873, 1896). From the late 1880s until the early 1900s, the embayment was filled and the shoreline was extended approximately 200 feet seaward to construct the road that would later become the Great Highway. The northern part of the alignment, south of Sloat Boulvard, in particular, was extended more than 200 feet seaward, and the Great Highway was constructed between 1920 and 1928.

The Westside Pump Station was constructed in 1985. In response to concerns about beach retreat due to erosion threatening the infrastructure, the Coastal Commission stipulated large scale beach nourishment as a condition of the permit for the pump station. The Lake Merced Transport Tunnel was constructed in 1992 and the Oceanside Water Pollution Control Plant was constructed in 1993.

Severe beach retreat from wave run-up occurred during the strong El Niño climatic events of 1994-1995, 1997-1998, and 2009-2010. The 1994-1995 El Niño event caused the bluff top edges to recede 20 to 30 feet, and bluff toes to recede 30 to 40 feet in an a 500-foot reach north of the Zoo entrance on the Great Highway. An unstable bluff face also formed from the northern 100 feet of the 500-foot reach. Continued bluff failure and sand slumping occurred along this bluff face, despite installation of rock protection. During the summer of 1996, quarry stone boulders (originally buried under the beach during construction of the SWOO in the 1980s) were exposed as the beach had dropped. A steel pile also was exposed in the intertidal zone, which was apparently placed during construction in the early twentieth century. This 500-feet stretch, called the Emergency Quarry Stone Revetment (EQR) Reach, experienced further bluff retreat during the El Niño climactic pattern of the winter of 1997-1998.

During the winter of 1997-1998, wave action removed 50 feet of bluff width in a 300-foot long reach starting at the entrance to the OSP and proceeding south (South Reach). In order to inhibit further bluff failure, the CCSF placed sand to fill the scarp in the fall of 1999, but the sand barrier washed away by April of 2001.

Following the 1990s El Niño climatic events there were strong storms, but little documented coastal retreat at the project site, until the winter of 2009-2010. Winter storms caused pavement collapse at the south end of the Sloat parking lot (EQR Reach) and in the South Reach. The shoulder of the Great Highway collapsed at a location opposite the OSP (Bluff Reach and South Reach). The CCSF responded by constructing the Emergency Bluff Toe protection rock revetment, consisting of large boulders, in the South Reach in 2010. A sandbag revetment was constructed in the North Reach in 2010.

In 2012, the CCSF implemented the Sand Backpass Project, where 50,000 cubic yards of sand was excavated from north Ocean Beach along the O'Shaughnessy Sea Wall and place it in erosion "hot spots" south of Sloat Boulevard. Wind-erosion control measures, such as natural brush fencing and coarse sand were installed to reduce windblown sand from being transported onto the parking lots and Great Highway south of Sloat Boulevard. To maintain public safety, approximately 700 sandbags were placed in areas with defined, severe erosion south of Sloat Boulevard.

According to a LiDAR study of a 2,000-kilometer stretch of the California and Oregon coastlines by the USGS (Barnard, 2017), the winter of 2015-2016 was one of the strongest El Niño winters on record. During the winter of 2015-2016, winter wave energy flux was approximately 50 percent above normal and water levels were elevated to +11 cm (approximately 4.4 inches) above summer levels. Barnard reported that shoreline retreat due to erosion during the winter of 2015-2016 was 76 percent above the normal winter shoreline retreat, and 27 percent higher than any other winter, including the 1997-1998 El Niño event.

2.1.3 Lake Merced Transport (LMT) Tunnel

The Lake Merced Transport (LMT) Tunnel is a 14-foot inside diameter, 16-foot outside diameter tunnel that was constructed in 1992 as part of San Francisco's Clean Water program. The LMT begins at the Westside Pump Station and continues 2.6 kilometers south and terminating at the Lake Merced Pump Station at the intersection of John Muir Drive and Skyline Boulevard. The tunnel was excavated with a tunnel boring machine (TBM) using a non-pressurized face with a reinforced cast-in-place lining. Loose, flowing sands were encountered during the boring of the northernmost 1800 feet of tunnel. From station 18+11 of the old stationing (which corresponds to approximately station 29+00 of the project stationing) and continuing to station 41+80 of the old stationing (beyond the project alignment), the tunnel was bored through weakly cemented silty sand without the need for dewatering. In addition to the issues with flowing sands, the excavation encountered a ship's hull at the beginning of the excavation, near Sloat Boulevard.

2.1.4 Southwest Ocean Outfall (SWOO)

The Southwest Ocean Outfall (SWOO) is a 23,400 feet outfall tunnel that crosses underneath the LMT in the vicinity of the OSP and extends westward carrying sanitary and stormwater flows into the ocean. The onshore concrete box structure is 12 by 12 feet at the beginning of the tunnel, and becomes a concrete pipe with a 12-foot inside diameter extending into the ocean from the headwall of the SWOO. The outfall was constructed between 1981 and 1985 using cut and cover on land and using a trestle for offshore construction. The terminus of the outfall is at a water depth of about 80 feet.

Exploratory borings drilled in 1977 by Woodward-Clyde for construction of the SWOO encountered a thin layer of loose surface sands along the entire alignment. This layer was approximately 2 to 4 feet thick and up to 6 to 8 feet thick in localized areas. The loose sand was underlain by medium dense to dense sand, increasing in density with depth.

During the construction of the LMT in 1992, the TBM encountered two sheet piles at Station 22+80 (old stationing) in the vicinity of the SWOO. Exploratory borings drilled by AGS in 1989 encountered deep fills with concrete, copper and plastic fragments at depths of up to 35 feet in the vicinity of where the SWOO crosses underneath the LMT.

2.2 GEOLOGICAL SETTING

The geologic conditions of the project site are presented on Plates 3A and 3B – Regional and Local Geologic Maps.

2.2.1 Regional Geology

The project study area is located along the coastal bluffs on the southwest side of San Francisco, and is bounded to the west by the Pacific Ocean. The San Francisco Peninsula constrains the western side of the San Francisco Bay, a northwest-trending structural depression called a "bay block" that was submerged by rising sea level during the Holocene. This bay block is within the Coast Ranges geomorphic province, a region characterized by generally northwest-trending mountain ridges, valleys, and faults. The bay block is bounded by the San Andreas fault to the west and by the Hayward fault to the east. The San Andreas fault crosses the coastline approximately three kilometers southwest of the project alignment. The San Francisco Peninsula is underlain by bedrock of the Franciscan Complex. In the project vicinity, the Franciscan bedrock surface is estimated to occur deeper than 300 feet (Schlocker, J., and Bonilla, M.G., 1972), and, based on Boring B-1 from Woodward-Clyde 1977, may be at a depth of approximately 400 feet.

Past episodes of tectonism have folded and faulted the rock of the Coast Ranges, creating northwest-trending ridges and valleys characteristic of this area. The project site lies on the northern end of a topographic depression, the Merced Basin: an ancient sedimentary basin bounded by the San Andreas Fault to the southwest and the Serra Fault to the northeast. The Serra is a northwest striking, southwest dipping thrust fault that is currently thought to be inactive, but may have been active during the Holocene (Kennedy, 2002). The Merced Basin is thought to be an extensional pull-apart basin, which became filled with sediments of Franciscan origin as the basin subsided and the rocks in the Franciscan subduction zone were uplifted.

The Merced Basin is also a subset of the Colma Channel, an ancient watercourse leading from San Francisco Bay to the Pacific Ocean. This northwest trending channel, which was formed during late Cretaceous and Tertiary time, is bounded to the west by the San Andreas Fault and to the east by the inactive San Bruno Fault and the present day San Bruno Mountains. The San Bruno Fault is thought to be an inactive, westward dipping normal fault.

During the early Pleistocene Sangamon interglacial, about 100,000 years ago, sea level was higher than it is today and much of the San Francisco shoreline, including the Merced Basin and the Colma Channel, was below water and connected to the Pacific Ocean. The Merced Basin and the Colma Channel were filled with marine as well as continental (Franciscan) sediments. During the middle Pleistocene Wisconsin glacial, approximately 15,000 years ago, the Merced Basin, Colma Channel and parts of San Francisco Bay, were above water. The shoreline of what we now call Ocean Beach stretched up to twenty miles westward, above sea level. During this time, the Sacramento River flowed to the ocean and deposited sand derived from the granitic, plutonic and andesitic materials of the Sierra Nevada at Ocean Beach.

As glaciers melted and sea level rose, approximately 10,000 years ago, the Merced Basin and Colma Channel were filled with alluvial fan and remains above sea level to the present day. The sediments filling the Merced Basin are up to 1,700 meters thick and are now called the Merced Formation.

The present-day bluffs at Ocean Beach are uplifted Merced and Colma Formation sedimentary units with a lithology reflecting the landward and seaward movement of the shoreline over episodes of glacial maximums and minimums. There are alternating layers of marine sediments, such as silts and clays, coarser sediments such as sand and gravel deposited in the surf zone, “backshore” sediments such as fine-grained sands, silts and muds deposited along coastal embayments, as well as nonmarine estuarine fine-grained sediments and wind-blown sands.

2.2.2 Site Geology

In the project vicinity, the major geologic units are: historical artificial fill (Qaf), Holocene-age dune sand (Qd) and beach sand (Qb), Pleistocene-age Colma Formation (Qc), Pliocene-age Merced Formation (Qm) and Jurassic and Cretaceous Franciscan Complex (KJf).

2.2.2.1 Artifical Fill (Qaf)

In the vicinity of Sloat Boulevard, Ocean Beach is separated from the Great Highway by a rubble wall about 100 feet wide and 20 to 23 feet above high water. This rubble wall was built in the late 1800s to early 1900s in order to provide a surface for the construction of the Great Highway. The rubble consists of angular fragments of red chert, sandstone and greenstone of the Franciscan Formation, in a mixture of sand and clay. Artificial fill that resulted from grading operations during development of the project site is derived from native sediments, making it difficult to distinguish from dune sands and weathered unconsolidated Colma Formation sands.

The artificial fill consists mainly of reworked dune sand, with occasional gravel and construction debris, and is commonly underlain by dune sand. The thickest fill occurs as infill along the bluffs, and as backfill around drainage pipes and other utilities. In the near-surface, the fill consists of clayey or sandy angular gravel.

2.2.2.2 Dune Sand (Qd)

In the project vicinity, Holocene dune sand deposits (Qd) extend from the western edge of Lake Merced to the coast. These deposits were fed by sand blown east from Ocean Beach and were deposited in the post-glacial period within the last 10,000 years. The thickness of the dune sand ranges from light cover at the tops of the highest bluffs, to up to 50 feet inland of the coast in the project. Near-surface dune sands tend to be poorly graded, fine to medium grained clean sand, whereas sands at depth may have light cementation or laminations.

2.2.2.3 Beach Sand (Qb)

Beach sand in the project vicinity is comprised of loose, well-sorted quartz and feldspar sand, which grades fine to coarse depending on its location in the surf zone. The beach sand at Ocean Beach has heavy mineral laminations, as well as thick layers of sand comprised of magnetite at the beach surface at the toe of the bluffs. These sands primarily originated from the Sierra Nevada during the previous low-stand sea level, when the San Francisco Bay was above water. Some of the sands are also comprised of continental (Franciscan) sediment outwash.

2.2.2.4 Colma Formation (Qc)

The Colma Formation overlies the tilted beds of the Merced Formation at an angular unconformity. Inland units of the Colma Formation include poorly consolidated colluvial, stream and eolian deposits, whereas coastal deposits of the Colma are more likely to be marine (nearshore and backshore) and estuarine in nature. Like the Merced Formation, the facies of the Colma Formation were formed with similar sedimentary material as well as in a similar depositional environment created by the rising and falling sea level causing transgression and regression of the shoreline.

Yi (2005), McGuire (2009) and others have described the Colma as a an approximately 3-foot thick “thin erosional remnant” at Ocean Beach in the project vicinity, and up to about 40 feet thick progressing south to Thornton Beach. However, Woodward-Clyde (1977) indicated that the Colma is up to 200 feet thick in the vicinity of the SWOO and overlies the Merced, and Bonilla (1988) maps the Colma as underlying the northern two-thirds of the site.

The Colma Formation generally consists of oxidized, reddish brown, predominantly medium-grained quartz-feldspar arkosic sand with heavy mineral laminations, and bedding ranging from horizontal up to dipping 5 degrees East. Facies of the Colma Formation at depth may include fine-grained micaceous silty sand, silt, thin clay lenses, and lenses of rounded fine gravels consisting of red chert, green chert, Monterey formation laminated rock, and blue schist.

According to Bonilla (1998) and Kennedy (2002), the Colma Formation is of latest Pleistocene age and was deposited between about 70,000 to 130,000 years ago.

2.2.2.5 Merced Formation (Qm)

The Merced Formation overlies the Franciscan Complex in the project vicinity, and consists of an accumulation of poorly consolidated sand, clay, gravel and silt sediments, which were deposited almost continuously in the late to early Pleistocene. Clifton and Hunter (1988) mapped a sequence of approximately 40 facies in the variably tilted and uplifted Merced exposures in the seaside cliffs, from the southern edge of the project site near Boring B-6 and continuing south approximately 7 kilometers to Mussel Rock. These sequences consist of marginal marine sediments, such as shelf, nearshore, backshore, embayment and fluvial facies, and their arrangement is indicative of alternate transgression and regression of the sea during geologic time. Yi (2009) mapped exposures of the Merced Formation in the project vicinity, from Sloat Boulevard to Thornton Beach, which is approximately 4.5 kilometers south of Sloat.

According to Hall (1965), Clifton (1988), Yi (2009), Kennedy (2002) and McGuire (2005), the Sequence X and Y of the Merced Formation are exposed closest to the project vicinity. These facies are generally weakly lithified to well-cemented, thinly bedded silts, sands, clays and pebbly shell hash deposited in a shallow marine environment. Merced Formation at depth in the project vicinity is assumed to be characterized by light gray to dark gray and black fine-grained sand high in heavy minerals such as magnetite, and dark bluish gray fat clays with silty interbeds. Micaceous material indicative of backshore deposits is also possible in the Merced.

Based on the tectonic history of the Serra Fault, the Merced Formation can show bedding ranging from near-horizontal in the project vicinity, to up to 25 degrees and striking northeast in the vicinity of Fort Funston and Mussel Rock. Based on Woodward-Clyde's Boring B-1, the Merced Formation in the project vicinity can extend to approximately 400 feet depth at the contact with the Franciscan Formation.

2.2.2.6 Franciscan Complex (KJf)

Franciscan Complex rocks underlying the project site and its vicinity include graywacke sandstone, siltstone, claystone and shale.

2.3 SUBSURFACE CONDITIONS

AGS met with Professor John Caskey of SFSU on March 19th, 2019 to discuss the subsurface stratigraphy at the project site. Professor Caskey and his graduate students have studied the Colma and Merced formations in the site vicinity and south of the site vicinity for the past two decades. Chimi Yi mapped outcrops of the Merced Formation on the cliff exposures starting at Sloat Boulevard and continuing south 7 kms to Thornton Beach (Yi, 2005). Yi also tested samples from the Colma, Merced, and Dune Sand units for grain size distribution and petrography. Drew Kennedy hypothesized that the Merced Formation has been folded by and is bounded by the Serra Fault in the project vicinity, and he used optical luminescence to date the units (Kennedy, 2002). In a personal communication dated March 2019, Caskey indicated that the project site is likely to be chiefly underlain by Merced Formation in the near surface. He added that it is extremely difficult to distinguish between the bottom-most units of the Colma and the uppermost units of the Merced formations, as they are comprised of nearly identical material. This is a point to which others have also alluded (Woodward-Clyde 1977, Clifton and Hunter 1988). In their 1977 report, Woodward-Clyde indicates that the project site is underlain in the near surface by dune sands, then Colma formation up to 200 feet depth, and by Merced Formation up to 5,000 feet depth.

The results of the processed data from the geophysical survey were not available at time of publication of this CER. Preliminary results show the contact of fill or dune sand to Colma is at approximately 20 feet depth in most of the MASW lines, with the exception of deeper fill up to 35 feet in ML-1 in the vicinity of the SWOO. AGS is also awaiting laboratory testing results, including the petrographic and X-Ray diffraction testing of soil samples, which will be available at time of publication of the GDR and GIR.

The site stratigraphy shown in Table 2 and on Plates 4A to 4E represents AGS Inc.'s estimate of the location of the units. The subsurface stratigraphy along the SWOO cross section trending east to west towards the ocean, crossing the LMT, is presented on Plate 5.

TABLE 2
SUMMARY OF SUBSURFACE CONDITIONS

Reach	Representative Borings ID	Fill Thickness (feet)	Dune Sand Thickness (feet)	Colma Formation Thickness (feet)	Merced Formation Thickness (feet)	Maximum Depth of Exploration (feet)
North	B-1, R3-1*, B-6**, CPT-2	20	5-15	50-65	>31.5	101.5
EQR	B-2, CPT-3, CPT-4	10-20	0-10	30-40	>31.5	100.5
Rubble	B-3, B-4, B-5**, CPT-7, CPT-9	5-15	5-10	35-55	>31.5	101.5
Bluff	B-5, R2-1*, CPT-10	10-20	5-15	40-60	>6.3	79.7
South	B-6, R1-C2*, R1-C3*, R1-C1*, R1-B3*, B1-B1*, R1-A1*, CPT-11, CPT-12, CPT-13	30-40	0-10	>10	>40	76.3

*Boring from AGS 2010

**Boring from AGS 1989

2.3.1 North Reach

The subsurface stratigraphy of the North Reach is characterized by abundant fill and dune sand. This is the location of the former embayment where a ship's hull dating to the 1800s was found during construction of the LMT. The North Reach has up to 20 feet of loose to medium dense poorly graded sand fill overlying 5 to 15 feet of medium dense to dense dune sand. The dune sand is underlain by approximately 50 to 65 feet of dense to very dense Colma Formation, consisting of light gray, clean poorly graded sand, sand with silt, and silty sand. Sand grain is predominantly medium, with occasional lenses of very fine and coarse grained sand and rounded fine gravels. The Colma formation is underlain by dark gray sand and stiff clay of the Merced Formation, starting at a depth of approximately 70 feet below ground surface.

2.3.2 EQR Reach

A 10- to 20-foot layer of medium dense artificial fill overlies up to 10 feet of light gray and dark gray loose to medium dense poorly graded dune sand. Underlying the dune sand is a 30- to 50-foot thick layer of dense to very dense sand with silt and silty sand of the Colma formation. This sand is reddish brown in color and grades to gray at approximately 45 feet depth. Stiff gray sandy silt of the Merced Formation begins at approximately 50 feet below ground surface and grades into silty sand and clayey sand.

2.3.3 Rubble Reach

The Rubble Reach has between 5 to 15 feet of loose to very dense sandy artificial fill. This is underlain by 5 to 10 feet of medium dense to dense poorly graded dune sand. The Colma formation begins at approximately 20 feet depth and is approximately 35 to 50 feet thick. The Colma Formation consists of dense to very dense reddish brown sand with silt and silty sand, grading to bluish gray at approximately 40 feet depth in Boring B-3 and at about 68 feet below ground surface in Boring B-4. The Colma Formation in Boring B-4 is unique in that the silty sand was medium dense to dense instead of very dense throughout most of the soil column, and a 10-feet layer of silty fat clay and sandy fat clay was encountered at 44 feet depth. In addition, abundant micaceous material was found in the yellowish and oxidized reddish brown sandy silt and silty sand samples between 35 to 64 feet below ground surface. The Colma Formation is underlain by dark bluish gray silts, clays and sands of the Merced Formation at approximately 70 feet depth in Boring B-3 and approximately 50 feet depth in CPT-7 and CPT-9.

2.3.4 Bluff Reach

The Bluff Reach has 10-20 feet of medium dense silty clayey sand or silty sand fill, underlain by 5 to 15 feet of medium dense poorly graded dune sand. Based on the exposed utility pipeline within a more than 15-foot deep eroded gully in this reach, indicating weaker, less cohesive material, and based on material indicative of fill up to greater than 20 feet depth in Boring R1-C1, we note that there may be about 20 of uncontrolled fill in localized areas. Approximately 40 to 60 feet of dense to very dense sand with silt and silty sand of the Colma formation underlies the dune sand. Based on the CPT-10 sounding extended to 76 feet below ground surface, the Merced is likely to have been encountered at approximately 70 feet below ground surface.

2.3.5 South Reach

The South Reach is the location of the roadway pavement being undercut by erosion in the winter of 2010. Evidence of artificial fill up to about 30 feet below ground surface is observable from the beach looking up at the soil exposures. Brick fragments, concrete and metal fragments were found embedded in the face of the cliff on March 19th. 2019. Borings drilled by AGS in

2010 encountered concrete fragments, cast iron pipe, copper wire pieces, plastic debris, and lenses of angular gravel up to about 30 feet depth. Boring B-6 drilled by AGS in 2019 encountered rubber or tar material at approximately 25 feet depth and refused on concrete at 35 to 38 feet depth.

The subsurface stratigraphy in the South Reach is generally composed of 30 to 40 feet of loose to medium dense artificial fill consisting of sand with silt, silty sand, and angular gravel lenses, as well as concrete, metal and other debris. The fill is underlain in some borings by up to 10 feet of medium dense dune sand with silt. The fill or dune sand layers are underlain medium dense to very dense Colma Formation consisting of light gray or reddish brown and black laminated sand with silt or silty clayey sand. The Merced Formation is expected to underlie the fill or dune sand at approximately 40 feet depth in the southernmost exploration locations (CPT-12 and CPT-13).

2.4 FAULTING AND SEISMICITY

2.4.1 Surface Fault Rupture

The site is not located within an Alquist-Priolo earthquake fault zone (CGS, 2007). Therefore, the risk from surface fault rupture is considered to be very low. The northern portion of the site from station 11+00 to Station 36+00 site is located within a seismic hazard – liquefaction study zone, which indicates a potential for permanent seismically-induced ground deformation.

2.4.2 Historical Seismicity

The project area is located in a seismically active region subject to periodic earthquakes causing strong to violent ground shaking of the site. The San Andreas Fault is about 2.6 kilometers (km) southwest of the site and is the major fault system in the region. Further from the project site are the San Gregorio Fault, which is 7.7 km southwest of the site, the Hayward Fault, which is about 27.4 km to the northeast; both are also significant seismic sources. Other major active faults considered capable of causing significant shaking at the project site by the United States Geological Survey (USGS) include the Point Reyes, Monte Vista-Shannon, Mount Diablo Thrust, Calaveras, Green Valley, West Napa, Greenville and Great Valley faults. Active fault traces and epicenters of recent earthquakes are shown on Plate 6 – Earthquake Epicenters and Fault Map. Historic earthquakes attributed to each fault are listed in Table 3 - Historical Earthquakes.

TABLE 3

HISTORICAL EARTHQUAKES

Date	Magnitude	Fault	Epicenter Area
June 24, 1808	6.0[5]	Unknown	Uncertain, San Francisco Bay Area
June 10, 1836	6.5[1], 6.8[5]	San Andreas	San Juan Bautista
June 1838	7.5[1], 7.0[5]	San Andreas	San Juan Bautista
Nov. 26, 1858	6.25[5]	Calaveras	San Jose Area
February 26, 1864	6.0[5]	San Andreas	South Santa Cruz Mountains
March 26, 1864	6.0[5]	San Andreas	Santa Cruz Mountains
October 8, 1865	6.3[2], 6.5[5]	San Andreas	South Santa Cruz Mountains
October 21, 1868	7.0[2,5]	Hayward	Berkeley Hills, San Leandro
February 17, 1870	6.0[5]	San Andreas	Los Gatos
April 12, 1885	6.25[5]	San Andreas	South Diablo Range
May 19, 1889	6.25[5]	Concord-Green Valley	Antioch
April 24. 1890	6.25[5]	San Andreas	Pajaro Gap
April 19, 1892	6.5[5]	Great Valley	Vacaville
April 21, 1892	6.25[5]	Great Valley	Winters
June 20, 1897	6.25[5]	Calaveras	Gilroy
March 31, 1898	6.5[5]	Rodgers Creek	Mare Island
April 18, 1906	8.0[3]	San Andreas	Golden Gate
July 1, 1911	6.6[4], 6.5[5]	Calaveras	Diablo Range, East of San Jose
October 22, 1926	6.1[5]	San Gregorio	Monterey Bay
April 24, 1984	6.1[5]	Calaveras	Morgan Hill
October 17, 1989	7.1[5]	San Andreas	Loma Prieta, Santa Cruz Mountains
August 24, 2014	6.0[6]	West Napa	South Napa, American Canyon

1) Borchardt & Toppozada (1996)
2) Toppozada, et al. (1981)
3) Petersen, et al (1996)
4) Real, et al (1978), Toppozada (1984)
5) Ellsworth, W.L. (1990)
6) GEER (2014)

2.4.3 Regional Active Faulting

The maximum moment magnitude earthquake (Mmax) is defined as the largest earthquake that a given fault is considered capable of generating. The Mmax earthquake on the San Andreas Fault would be a magnitude 8.05 event occurring approximately 2.6 km (1.6 miles) from the project site. The seismicity associated with each pertinent fault within 70 kilometers, including estimated slip rates, is summarized in Table 4 - Fault Seismicity.

TABLE 4
FAULT SEISMICITY

Fault Name	Distance to site (km) [2]	Maximum Moment Magnitude [1]	Contributing Segments [2]	UCERF 3 Slip Rate [1] (mm/year)
San Andreas	2.6	8.05	Peninsula (SAP) + Santa Cruz Mountains (SAS) + Offshore (SAO) + North Coast (SAN)	17.0 17.0 24.0 24.0
San Gregorio Connected	7.7	7.50	San Gregorio (North) + San Gregorio (South	7.0 3.0
Hayward-Rodgers Creek	27.4	7.33	Rodgers Creek (RC) + Hayward Northern (HN) + Hayward Southern (HS)	9.0 9.0 9.0
Point Reyes	37.3	6.90	Point Reyes	0.1
Monte Vista - Shannon	39.6	6.50	Monte Vista-Shannon (MVS)	0.6
Mount Diablo Thrust	44.2	6.70	Mount Diablo Thrust North Mount Diablo Thrust South	2.0 2.0
Calaveras	44.8	7.03	Calaveras North (CN) + Calaveras Central (CC) + Calaveras South (CS)	6.0 15.0 15.0
Green Valley Connected	49.1	6.80	Green Valley	4.0
West Napa	53.4	6.70	West Napa (WN)	1.0
Greenville Connected	61.6	7.00	Greenville North Greenville South	3.0 3.0
Great Valley 5, Pittsburg Kirby Hills	66.2	6.70	Great Valley 5, Pittsburg Kirby Hills	1.5

1) WGCEP (2003, 2008), Working Group on California Earthquake Probabilities
Map distance to the nearest segment, based on USGS Quaternary Fault and Fold Database (2006)

2) WGCEP (2008), Tables I-1 and I-3 of Appendix I. Parameters for Faults in California, 2008, "Documentation for the 2008 Update of the United States National Seismic Hazard Maps" and UCERF3

2.5 GROUNDWATER

Table 5 summarizes the groundwater level data obtained by AGS from our field exploration and previous reports.

TABLE 5
GROUNDWATER LEVEL DATA

Source	Well Name	Depth to Groundwater (feet)	Elevation (NAVD88) (feet)	Reach	Date Measured (feet)
AGS 2019	MW-1	22.0	9.4	North	3/15/2019
AGS 2019	MW-4	22.0	8.3	Rubble	3/15/2019
AGS 2019	MW-5	23.3	5.5	Bluff	3/15/2019
AGS 1989	B-5	19.5	9.9	Rubble	5/24/1989
AGS 1989	B-6	23.5	7.9	North	5/24/1989
W-C 1977	OW-2	12.3	20.1	North	7/28/2006
HLA 1977	HLA-54	20.5	11.4	North	6/24/1977
W-C 1977	WC-4	29.5	6.9	Bluff	6/6/1977
W-C 1977	WC-10	35.0	13.5	South	6/6/1977

In addition to reviewing the water levels recorded in previous borings, AGS reviewed hydrographs from four monitoring wells located in the project site in the SFPUC's 2017 Annual Groundwater Monitoring Report (2018). The monitoring wells in the SF Zoo vicinity show that groundwater levels have fluctuated between -2 to +8 NAVD88 between 2005 and 2017, whereas the well located in the vicinity of the SWOO showed groundwater levels of between +10 to +14 NAVD88 between 2003 and 2017. According to Woodward Clyde Consultants (1977, SWOOP Geotechnical Report), the groundwater elevation across the site ranges from +13 NAVD88 east of the Great Highway to elevation +7 NAVD88 west of the Great Highway. AGS also reviewed the CGS Hazard Map (2000) which indicated that the water level is approximately 10 feet below ground surface in the project vicinity. We note that the groundwater contour was created using AGS 1989 Borings B-5 and B-6.

Lake Merced is approximately 1,200 feet southeast of the southernmost Boring B-2. The water level in Lake Merced is approximately elevation +20 NAVD88. According to the North Westside Groundwater Basin Management Plan (CCSF, 2005), studies indicate that there is no saltwater intrusion in the groundwater of Ocean Beach, and that groundwater levels toward the northern and western parts of Lake Merced have remained above sea level, resulting in a combination of apparent subsurface outflow toward the ocean and hydraulic resistance against seawater intrusion from the ocean. The Westside Groundwater Basin Management Plan (2005) report did, however, indicate that between 1940 and 1970, the flow of water in Lake Merced changed from a northwesterly direction (towards to ocean) to a southwesterly direction (towards Daly City due to pumping.

Woodward-Clyde estimated that the groundwater gradient is approximately 3.5 feet per 1000 feet of horizontal distance as the ground slopes down westward (from Lake Merced toward the ocean), also suggesting a potentiometric flow of groundwater from Lake Merced to the ocean. Woodward Clyde reported that pore water salinity test results indicate that the groundwater beneath the site is fresh water to as deep as elevation -350 NAVD88.

We thus expect groundwater levels at the site to be controlled by seasonal variations in the level of Lake Merced, or fluctuations of the lake's water level due to controlled outflow provided by the weir.

2.6 SOIL CORROSIVITY

AGS performed a corrosivity evaluation of site soils. Five samples were selected to represent the range of soils expected to contact the improvements (the secant pile wall and tie-backs): Fill, Dune Sand, and Colma Formation.

Soil samples were analyzed for the following parameters:

1. As-received resistivity (ASTM G57)
2. Electrical (minimum) resistivity – (Caltrans 643)
3. pH (ASTM G51)
4. Water-soluble chloride anion content (ASTMD4327)
5. Water-soluble sulfate anion content (ASTM D4327)

Corrosivity test results are shown in Table 6 – Corrosivity Potential.

TABLE 6
CORROSIVITY POTENTIAL

Boring ID	Depth	Resistivity at 15.5°C	Chloride	Sulfate		pH	ORP (Redox) at 21°C	Moisture	Soil Description
	(feet)	(ohm-cm)	mg/kg	mg/kg	%		E_H (mv)	%	
B-1	30-31.5	5,370	8	73	0.0073	8.3	451	16.1	Gray SAND w/ Silt
B-2	2.5-4.5	6,236	13	139	0.0139	8.2	486	7.2	Reddish Brown Silty SAND w/ Gravel
B-3	8.5-10	8,554	67	58	0.0058	8.6	509	0.4	Olive SAND w/ Silt
B-4	45.5-46	524	867	1,167	0.1167	7.9	277	32.3	Olive Gray CLAY w/ Sand
B-6	30-31.5	8,016	31	223	0.0223	7.6	447	10.5	Reddish Brown Silty SAND

The corrosivity test results are discussed in section 3.2. Based on the soil resistivity classification presented by National Association of Corrosion Engineers (2010) and the results of corrosivity testing at the site, the onsite soils are classified as “extremely corrosive” to “moderately corrosive”. According to ACI 318-11, the sulfate concentration measured in one of the corrosivity samples tested for this study indicates a Soil Exposure Class S1.

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 BLUFF RETREAT

According to the Coastal Management Framework (2014), shoreline erosion at South Ocean Beach has increased from about 1 foot per year between 1930-1990 to about 2 feet per year when extending the time interval from 1930 to 2010. In addition, according to a 2012 study by Barnard (USGS), the sand supply to Ocean Beach is likely to decrease in the future, which would lead to increased future erosion rates.

Climate change may lead to stronger and more frequent El Niño Southern Oscillations, which would result in larger and higher energy storms (Barnard, 2017). This means that severe events could cause erosion and beach and bluff retreat on the order of several decades' worth of normal erosion retreat distances. For example, bluff recession of 10 to 50 feet can occur in one winter due to severe events. This amount is in contrast to the progressive, long-term erosion forecast of 110 feet. The US Army Corps of Engineers (USACE, 1992) estimated that an event erosion resulting in bluff recession of 40 feet is likely to occur about once in 30 years. Therefore, at least 40 feet of bluff recession is expected by 2050, in addition to the long-term trend of progressive bluff recession (Moffat and Nichol, 2010).

The Ocean Beach Master Plan (SPUR, 2012) included estimates that the South Ocean Beach shoreline would recede 70 feet by 2050 and 275 feet by 2100 due to sea level rise, plus an additional 1 foot per year due to continuation of historic erosion, yielding total recession distances of 110 feet by 2050 and 365 feet by 2100.

For this project, we assume that the shoreline in front of the secant pile wall will erode to a level of elevation +2 feet NAVD88 in the long term condition, and that the bluffs over top of the secant pile wall will erode down to the CLSM improved soil, which is at a 3H:1V slope.

3.2 DESIGN GROUNDWATER LEVEL

Based on our review of the groundwater conditions encountered in various geotechnical studies performed within the project site and the available groundwater level monitoring data from the San Francisco Public Utilities Commission (SFPUC) Annual Groundwater Monitoring Reports for the Westside Basin (as discussed in the "Groundwater" section, we recommend that a groundwater level at Elevation +16 feet be used for design purposes in the North, EQR, and Rubble Reaches and part of the Bluff Reach. We recommend a design groundwater level at

Elevation +21 feet be used in the South Reach and part of the Bluff Reach.

3.3 SEISMIC DESIGN CRITERIA

Based on the methods of SFPUC General Seismic Requirements (SFPUC 2014 GSR), site specific foundation-level spectral accelerations were developed for the project alignment. According to Section 2.2.3 of the SFPUC GSR, design ground motions should be developed using a 5 percent probability of exceedance in 50 years (975-year return period) for structures which fall in Seismic Performance Class III. Section 2.8 of the SFPUC outlines acceptable procedures for estimating dynamic earth pressures acting on retaining walls.

3.3.1.1 Design Ground Motions

Based on the methods of the SFPUC 2014 GSR and Section 20 of ASCE 7-16, the site is classified as Site Class D – Stiff Soil with estimated shear wave velocity profiles in the upper 30 meters, or 100 feet, of the ground surface (Vs30) of about 220 meters per second (mps), or about 720 feet per second (fps).

3.3.1.2 Site Specific Seismic Design Parameters

Site specific seismic design parameters were determined in accordance with the methods of the SFPUC 2014 GSR. Correlations between distance from a causative fault and values of the peak horizontal accelerations and the effects of local soil conditions on peak ground accelerations have been developed for the site by using various attenuation relationships. In 2014, a unified series of seismic models called the Next Generation Attenuation West-2 (NGA-West2) relationships became available for site specific ground motion estimation [Abrahamson, Silva, and Kumai (2014), Boore, Stewart, Sayhan, and Atkinson (2014), Campbell and Bozorgnia (2014), Chiou and Youngs (2014), and Idriss (2014)]. These predictive relationships are pertinent to shallow crustal earthquakes in active tectonic regions such as the project area.

3.3.1.3 Probabilistic Methods

The Probabilistic Seismic Hazard Analysis (PSHA) analysis was performed using the OpenSHA "Hazard Spectrum Application" version 1.7.0 (2018) to estimate the site accelerations associated with the 975-year return period seismic events at the site. A 975-year return period ground motion corresponds to a 5 percent probability that the ground motion will be exceeded over a 50 year period. In accordance with the SFPUC 2014 GSR and a Performance Class III, a 975-year return period spectrum has been developed for the site by using applicable, equally-weighted attenuation relationships of the NGA West2, excluding Idriss & Boulanger. The UCERF3 fault source model (2015) was selected for our analysis. Based on our analyses, average acceleration response spectra with 5 percent chance of being exceeded over a 50-year period were computed and rotated to ROTD100 maximum direction using correction factors published in Shahi and Baker (2014).

3.3.1.4 Deterministic Methods

For determination of the deterministic horizontal bedrock spectral accelerations, AGS equally weighted results from four NGA West2 attenuation relationships, excluding Idriss, to estimate 84^{th} percentile ground motions for the design earthquake event (Mw = 8.05 and distance = 2.6 kilometers). Based on guidance of the SFPUC 2014 GSR, values were held to a minimum of those shown on Figure 21.1-1 of ASCE 7-16 for Site Class D The accelerations were rotated to ROTD100 maximum direction using correction factors published in Shahi and Baker (2014).

3.3.1.5 Recommended Spectral Accelerations

AGS developed a design acceleration response spectrum corresponding to a 5 percent structural damping ratio at the ground surface for the proposed secant pile wall, presented in Table 6 and depicted graphically on Plate 7. Seismic design parameters SMS, SM1, SDS and SD1 should be determined by the Structural Engineer using the procedure outlined in Section 21.4 of ASCE 7-16 using the data presented in Table 7.

TABLE 7
RECOMMENDED SPECTRAL ACCELERATIONS

Structural Period	Probabilistic MCE_R * (5% Damping)	Deterministic MCE_R ** (5% Damping)	Minimum: Probabilistic or Deterministic MCE_R (Smaller of Columns B or C) (5% Damping)	Deterministic Lower Limit (5% Damping)	Design Response Spectrum (5% Damping)
(sec)	(g)	(g)	(g)	(g)	(g)
A	B	C	D	E	F
0.01	0.73	0.87	0.73	0.87	0.43
0.02	0.73	0.87	0.73	0.87	0.43
0.03	0.73	0.86	0.73	0.86	0.43
0.05	0.80	0.91	0.80	0.97	0.54
0.08	0.97	1.04	0.97	1.20	0.65
0.10	1.12	1.18	1.12	1.43	0.74
0.15	1.33	1.38	1.33	1.50	0.80
0.20	1.48	1.56	1.48	1.56	0.87
0.25	1.61	1.76	1.61	1.76	0.95
0.30	1.73	1.95	1.73	1.95	1.02
0.40	1.83	2.21	1.83	2.21	1.08
0.50	1.81	2.27	1.81	2.27	1.07
0.75	1.53	2.06	1.53	2.06	0.90
1.00	1.27	1.81	1.27	1.81	0.74
1.50	0.92	1.40	0.92	1.40	0.53
2.00	0.70	1.08	0.70	1.08	0.41
3.00	0.46	0.71	0.46	0.71	0.26
4.00	0.32	0.49	0.32	0.49	0.19
5.00	0.24	0.36	0.24	0.36	0.14
7.50	0.12	0.18	0.12	0.18	0.07
10.00	0.07	0.11	0.07	0.11	0.04

** Based on 5% probability of collapse in 50 years based on Method 1 of ASCE 7-16 and risk coefficients*

CRS = 0.888 and CR1 = 0.873 and direction corrected to ROTD100 with factors by Shahi and Baker (2014)

*** Corrected to ROTD100 maximum direction with factors by Shahi and Baker (2014)*

3.4 LIQUEFACTION

Soil liquefaction is a phenomenon in which saturated (submerged) cohesionless soils lose their strength due to the build-up of excess pore water pressure, especially during cyclic loadings such as those induced by earthquakes. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movements, if not confined. Soils most susceptible to liquefaction are loose, clean sands. Silty sands and low-plasticity silts may also liquefy during strong ground shaking.

The nature of liquefaction depends greatly on the characteristics of the soil. In loose soils, liquefaction results in significant loss of soil strength, which can lead to large deformations. In dense soils, although a condition of liquefaction can be initiated, the tendencies for loss of strength and deformations are resisted by dilation of the soils. Deformation in dense soils results in a tendency for soil volume increase (dilation), which in turn results in reduction of pore water pressures, increase in effective stresses, and increased resistance to further deformations.

3.4.1 Liquefaction Triggering

The liquefaction potential of soils at the site was evaluated using a simplified, analytical, and empirical procedure that is correlated with the liquefaction behavior of saturated soils during historic earthquakes (Youd and Idriss, 2001; Idriss and Boulanger, 2008; Boulanger and Idriss, 2014). The primary data utilized in the analysis consisted of CPT test results analyzed with the computer program CLIQ, using a thin-layer correction. Borehole liquefaction analysis was performed using standard penetration test (SPT) and Modified California (MC) sampler blow counts obtained from the borings drilled at the site, as well as select previous borings drilled by AGS and others on the seaward side of the tunnel. The SPT and MC blow counts recorded in the field were corrected for various factors to obtain corrected N-values, which were used in the liquefaction analysis. The factors used to obtain corrected N-values, included the effects of overburden pressure, rod length, sampler type and size, and fines content.

The liquefaction analysis was conducted according to the method set forth in Idriss and Boulanger (2014), using the following parameters:

- Magnitude 8.05 earthquake;
- Geomean PGA_M; and
- Groundwater at Elevation +16 feet NAVD88 in the northern portion of the site, and Elevation +21 feet NAVD88 in the higher elevation portions of the Bluff Reach and the South Reach.

The results of our evaluation indicate that, in general, there are two layers of potentially liquefiable soil along the project alignment: the upper layer is approximately 5 to 7 feet thick and is located between approximately 15 to 25 feet depth, and the lower layer is approximately 5 to 10 feet thick and is located between 30 to 60 feet depth.

The upper 5- to 7-foot thick liquefiable layer appears to trend along the tunnel spring line between approximately 15 to 22 feet depth in the North, EQR and Rubble Reaches. Along the Bluff and South Reaches, the upper liquefiable layer is located between approximately 20 to 25 feet depth, and is also along the tunnel spring line. The upper liquefiable layer generally begins at the design groundwater depth in most of the borings, due to looser material in the upper 20 to 25 feet of most borings.

The 5- to 10-foot thick lower potentially liquefiable layer is located between approximately 50 to 60 feet depth in the North and EQR reaches, with the exception of a localized area around the vicinity of neighboring borings: B-2, CPT-3, and SCPT-3. In Boring B-2, a 20-feet thick layer of silty sand and sandy silt was identified between 50 to 70 feet depth. In CPT-3 and SCPT-3, an additional liquefiable layer was identified between 29 to 35 feet depth. This location is where the former roadway pavement has broken away and is being undercut by erosion, and over-steepened cliffs are presently without rock protection. The cliffs in the vicinity of CPT-3 are approximately 20 feet away from the southbound lane of the Great Highway, which is the smallest distance observed by AGS.

In the Rubble and Bluff Reaches, the 5- to 10-foot thick lower potentially liquefiable layer is located between approximately 30 to 40 feet depth, with the exception of the area near CPT-9, where a liquefiable layer was identified at 45 to 52 feet depth.

The lower liquefiable layer in the South Reach is at variable depths and has variable thicknesses, due to localized areas of of liquefiable loose to medium dense fill, as well as non-liquefiable dense to very dense layers of fill or native material in this reach. Based on AGS 2010 Boring R1-B3, a thick liquefiable layer was identified between 25 to 50 feet depth in the vicinity of the SWOO and the adjacent erosional gully, which is indicative of fill material. A 10-foot thick lower potentially liquefiable layer was identified at between 30 to 40 feet depth in AGS 2010 R1-A1 and potentially liquefiable soil was identified between 25 to 35 feet depth and 40 to 45 feet depth in AGS 2010 R1B1. At the end of the project alignment, in the vicinity of CPT-12, the lower potentially liquefiable layer begins at 45 feet, which is the depth to groundwater.

3.4.2 Consequences of Liquefaction

The main effects of liquefaction at the site include settlement of the ground surface, lateral deformation, development of excess pore water pressure, buoyancy effects on the below groundwater structures, loss of allowable bearing pressure, downdrag force on the proposed secant pile retaining wall and/or pile cap, and increased lateral pressures on below grade structural elements, and foundations extending below the groundwater table. Liquefaction of soils underlying the existing below ground structures may also induce temporary buoyant uplift pressures.

3.4.3 Liquefaction-Induced Settlement

Liquefaction of the in-situ, loose to medium dense, saturated site soils may occur and would result in liquefaction-induced settlement.

The liquefaction analyses were performed based on findings from our subsurface exploration. The estimated seismically-induced settlements and the thickness of the liquefiable layers for the borings and CPTs are presented on Table 8.

As seen from Table 8, for the majority of the alignment, potentially liquefiable soils exist below the LMT tunnel spring line, with estimated liquefaction-induced settlements ranging from 0.5 to 4 inches.

TABLE 8
ESTIMATED LIQUEFACTION-INDUCED SETTLEMENTS

Boring or CPT	Reach	Seaward or Landward of Tunnel	Ground Surface Elevation (NAVD88) (feet)	Maximum Depth of Boring (feet)	Design Depth to Groundwater (feet)	Depths of liquefiable layers (feet)	Liquefaction-induced settlement (inches)
B-1	North	Seaward	31	101.5	15	14-20	1.0
AGS 1989 B-6	North	Landward	31	81	15	15-20 55-58	2.0 0.5
CPT-1	North	Seaward	31	10.5	15	-	-
CPT-2	North	Seaward	29.5	62.5	13.5	14-24 51-54	2.0 1.0
B-2	EQR	Seaward	30	81.5	14	14-15 50-70	0.5 4.0
CPT-3	EQR	Seaward	30	100.4	14	15-22.5 33-35 60-70	2.0 0.5 2.0
SCPT-3	EQR	Seaward	30	80.4	14	14-22 29-35 60-72	1.5 0.5 2.0
CPT-4	EQR	Seaward	30.5	80.5	13.5	14-20 42-46	1.5 1.0
CPT-5	EQR	Landward	28	60.4	12	12-23 48-50	4.5 0.5
B-3	Rubble	Seaward	28.5	101.5	12.5	-	-
B-4	Rubble	Seaward	29	81.5	13	-	-
AGS 1989 B-5	Rubble	Seaward	29	66.5	13	15-20	1.5
CPT-6	Rubble	Landward	29	40.4	13	13-19 31-38	2.5 2.0
CPT-7	Rubble	Seaward	29	80.5	13	13-22 28-32 38-40	2.5 0.5 0.5

TABLE 8, CONTINUED
ESTIMATED LIQUEFACTION-INDUCED SETTLEMENTS

Boring or CPT	Reach	Seaward or Landward of Tunnel	Ground Surface Elevation (NAVD88) (feet)	Maximum Depth of Boring (feet)	Design Depth to Groundwater (feet)	Depths of liquefiable layers (feet)	Liquefaction-induced settlement (inches)
CPT-8	Rubble	Landward	29	40.2	14	21-27 32-34	0.5 0.5
CPT-9	Rubble	Seaward	29	80.5	14	45-52	0.5
B-5	Bluff	Seaward	31	51.5	15	-	-
CPT-10	Bluff	Seaward	36	79.7	20	20-24 32-33	0.5 0.5
AGS 2010 R1-C2	South	Seaward	39	31.5	20	20-25	1.0
AGS 2010 R1-B3	South	Seaward	43	71.5	24	25-50	5.0
AGS 2010 R1-B1	South	Seaward	46	48	25	25-30 40-45	2.0 1.0
AGS 2010 R1-A1	South	Seaward	50	41.5	30	30-40	2.0
CPT-11*	South	Landward	60	40.4	40	-	-
CPT-12*	South	Seaward	65	76.3	45	44.5-50 56-63	0.5 1.0
B-6*	South	Seaward	70	38	49	-	-
CPT-13*	South	Seaward	85	80.5	60	60-64 72-74 78-80	2.0 0.5 0.5

*These borings and CPTs are located beyond Station 42+60, which is the extent of the proposed secant pile wall.

3.4.4 Liquefaction-Induced Lateral Deformation

Liquefaction-induced lateral deformation is another phenomenon which can occur during a seismic event. At present, the LMT is located as close to approximately 35 feet from free faces of 20 to 30 feet cliffs, particularly near the Rubble Reach, where several deeply incised erosional gullies have formed. During a major earthquake, large liquefaction-induced lateral deformations could occur toward the free face, where significant amounts of liquefiable soils exist as a continuous layer. The secant pile wall should be extended above the potentially liquefiable soils and be designed to restrain the forces associated with these potential lateral deformations. Geotechnical recommendations in that regard will be provided in the GIR.

3.4.5 Liquefaction-Induced Settlement below LMT

Based on Table 8 above, the estimate liquefaction-induced settlement below the LMT (based on depth below the spring line) could be as shown in Table 9.

TABLE 9
LIQUEFACTION-INDUCED SETTLEMENT BELOW LMT

Reach	Liquefaction-induced Settlement below Spring line of LMT (inches)
North	0.5 to 1
EQR	0.5 to 4
Rubble	0.5 to 2.5
Bluff	0.5 to 1
South	0.5 to 1.5

To assess the impact of liquefaction-induced settlement on the structural integrity of the LMT tunnel, a numerical modeling study using finite element or finite difference analyses (such as FLAC) may be needed during the design phase.

3.4.6 Liquefaction-Induced Lateral Earth Pressure

If the soils behind the secant pile wall liquefy during a major earthquake, the lateral earth pressure exerted on the wall would be momentarily increased. Our recommendations on the liquefaction-induced lateral earth pressure for design are presented in the "Lateral Earth Pressures" section.

3.4.7 Liquefaction-Induced Uplift Pressure

Soil liquefaction can also result in an increase in uplift pressure on buried structures surrounded by liquefiable soils during a major earthquake. Such condition generally occurs when saturated, loose to medium dense sandy soils around the buried structures liquefy and lose their shear resistance against uplift from buoyance of the buried structures. The majority of the potentially liquefiable soils encountered are either up to about 6 feet above (or near) the crown of the LMT or relatively thin localized layers that are approximately 1 to 2 feet thick adjacent (from crown to invert) to the tunnel. The risk of uplift of the LMT during a major earthquake could be evaluated by numerical modeling, similar to that performed by Jacobs Associates in September 2014 taking into account the reduction in shearing resistance of liquefied soil during earthquakes.

3.4.8 Liquefaction Mitigation

The consequences of liquefaction, such as liquefaction-induced settlement and lateral deformations, have been discussed in the previous sections. If the consequences of liquefaction are not acceptable to the design team, possible mitigation measures are discussed below.

Ground improvement of potentially liquefiable soils may consist of either deep soil mixing (DSM) or chemical grouting. Other ground improvement techniques are also available; however, they are apparently not feasible due to the site constraints, specifically over the concern of potential impact to the LMT.

DSM is a technique that involves mixing cementitious materials and in-place soils with a hollow-stem auger and paddle arrangement. Augers up to about 3 feet in diameter are commonly used to create soil-cement cells, and overlapping soil-cement columns are required in order to be an effective ground improvement. DSM may generate a significant amount of spoils that must be controlled and handled.

Chemical grouting involves the injection of low viscosity liquid grout (such as silicates, resins, microfine cements or polyurethane) under pressure into the pore spaces of granular soils that results in hardening of the soils by binding the soil grains together.

DSM and chemical grouting are typically constructed through a design-build contract and specific design recommendations will depend on the methods and equipment used by the specialty contractors. If liquefaction mitigation would be considered, AGS can provide preliminary recommendations in the GIR.

3.5 UNSATURATED, EARTHQUAKE-INDUCED SETTLEMENT

Loose, unsaturated sandy soils tend to compress during dynamic shaking. These unsaturated sandy soils can settle and densify from earthquake shaking. This is a concern at this site because the site soils tend to be loose to medium dense in the upper 20 feet and they are unsaturated. Furthermore, the sea cliffs are not laterally confined in the upper 15 to 20 feet, which could exacerbate the risk of lateral spreading.

AGS used the Liquefy Pro program to estimate the vertical settlements expected from the unsaturated sandy layers in the borings. LiquefyPro is a program that uses a procedure developed by Tokimatsu and Seed (1987) which relates $SPT_{(N1)}$ value to relative density and uses Silver and Seed's (1971) study that gives the settlement of dry sand as a function of the cyclic shear strain, the number of strain cycles, and the relative density of the sand. AGS used CLIQ to calculate dry sand settlements in the CPTs based on Robertson and Shao (2010).

Table 10 shows the estimated vertical deformations due to earthquake-induced settlements of dry soils at the site.

We consider that the impact of unsaturated, earthquake-induced settlement (compaction settlement) on the LMT will be negligible. The soil-cement mix (CLSM) may crack. It is our opinion that the soil-cement mix cap will still function and the damage will be limited to pedestrian tripping hazards. Compaction settlement may impact the access road and trail. Repair of the access road and trail, consisting of surface re-grading, may be needed after a seismic event.

TABLE 10
ESTIMATED EARTHQUAKE-INDUCED DRY SAND SETTLEMENTS

Reach	Boring or CPT	Thickness of Unsaturated Layer (feet)	Earthquake-induced settlement (inches)
North	B-1	15.0	11.0
	AGS 1989 B-6	15.0	10.5
	CPT-1	15.0	>5.0*
	CPT-2	13.5	9.0
EQR	B-2	14.0	4.5
	CPT-3	14.0	2.5
	SCPT-3	14.0	1.5
	CPT-4	13.5	3.5
	CPT-5	12.0	8.5
Rubble	B-3	14.0	4.5
	B-4	13.0	2.5
	AGS 1989 B-5	13.0	1.5
	CPT-6	13.0	7.5
	CPT-7	13.0	3.5
	CPT-8	14.0	0.5
	CPT-9	14.0	0.1
Bluff	B-5	24.0	3.5
	CPT-10	20.0	1.0
South	B-6	39.0	1.5
	AGS 2010 R1-B3	24.0	3.5
	AGS 2010 R1-C2	20.0	7.5
	CPT-11	40	0.5
	CPT-12	45	0.5
	CPT-13	60	12.0

*Maximum depth of CPT-1 was less than total thickness of unsaturated layer.

3.6 LANDSLIDES

The project site is generally not located within a State of California designated Seismic Hazard Zone for earthquake-induced landslides (CGS, 2000). The sea cliff adjacent to the southern end of the project site (south of approximately Stationing 42+00) is mapped by the CGS to be in an area considered potentially susceptible to earthquake-induced landslides. Based on our review of the published geologic data, including Bonilla (1998) and Clifton and Hunter (1999), the materials mapped at the sea cliff mostly consist of the Merced Formation with favorably-oriented beddings (dipping into the slope of the sea cliff). As noted above, the landsliding hazard associated with the project site is primarily due to coastal erosion. It is our opinion that, after the construction of the proposed secant pile wall in conformance with our geotechnical recommendations, the potential for future landsliding adversely affecting the LMT tunnel would become low.

3.7 SEA LEVEL RISE

Sea level fluctuates under tidal influence and is projected to increase over the next 50 to 100 years due to increased greenhouse gas emissions melting the polar ice caps.

Sea level rise has several implications for this site: (1) Narrowing of the shoreline (beach width diminishment) due to higher still water level; (2) Increased wave runup height; and (3) Increased rate of coastal bluff erosion due to higher dynamic water level from storms.

According to the One SF Sea Level Rise hazard map, which is based off the National Research Council projection of a most likely sea level rise of 36 inches by 2100, sea level is expected to reach the southern edge of the southbound lane of the Great Highway in 2100. The NRC also projects a potential sea level rise of 66 inches if ice melting accelerates beyond current conditions, plus an additional 40 inches for storm surges and king tides.

According to data from the California Coastal Commission (2018) and the State of California Sea Level Rise Guidance (2018) the projected sea level rise is expected to be 1.9 feet by 2050 and 6.9 feet by 2100, assuming a “Medium-High Risk Aversion” scenario, which is a 1-in-200 chance scenario, based on Kopp et al. (2014).

The design sea level for 2100 used in this study is +9 NAVD88. The design life of the LMT improvements is 100 years.

3.8 TSUNAMI

The Tsunami Inundation Map for Emergency Planning (San Francisco North Quadrangly, June 2009, State of California) indicates that the project site is within an area at risk for tsunami inundation. The tsunami inundation line extends from the shoreline up to and including the Great Highway between Station 12+00 to Station 22+00. Between Station 22+00 to Station 33+00, the tsunami inundation line extends to the immediate west of the southbound lane of the

Great Highway.

We note that tsunami risk, including the inundation zone, is increased with sea level rise.

3.9 CONSTRUCTION SEQUENCING CONSIDERATIONS

We understand that, at present, the preferred construction sequence for the secant pile wall is as follows (Alternative A):

1. Install the primary unreinforced and secondary reinforced concrete piles of the secant pile wall by drilling from the existing ground surface;
2. Construct the soil-cement mix cap (for the ultimate 3H:1V backslope) by jet grouting (mechanically mixing the existing soils with cementitious grout in place);
3. Excavate down to the bottom of pile cap elevation with open cut excavations (with dewatering where necessary) on both sides of the secant pile wall;
4. Construct the pile cap;
5. Install tiebacks after the pile cap has reached sufficient strength; and
6. Backfill the excavations with properly compacted engineered fill.

We understand that the following alternative is also under consideration (Alternative B):

1. Install the primary unreinforced and secondary reinforced concrete piles of the secant pile wall by drilling from the existing ground surface;
2. Excavate down to the bottom of pile cap elevation with open cut excavations (with dewatering where necessary) on both sides of the secant pile wall;
3. Construct the pile cap;
4. Install tiebacks after the pile cap has reached sufficient strength;
5. Construct the soil-cement mix cap (for the ultimate 3H:1V backslope) with controlled low strength material (CLSM), which consists of a fluid, workable mixture of cement, aggregate and water (to be placed in sections with terraced wooden forms); and
6. Backfill the excavations with properly compacted engineered fill.

From a geotechnical engineering standpoint, some considerations that may influence the selection of these sequencing alternatives are presented in Table 11.

TABLE 11
SEQUENCING ALTERNATIVES

Alternatives	Advantages	Limitations
Alternative 1 soil-cement mix cap by jet grouting	• Landward open cut slope steeper than 1½H:1V possible, if soils above soil-cement mix cap also to be improved. Otherwise, condition of open cut slope no steeper than 1½ H:1V would remain.	• Relatively high cost of jet grouting • Difficult to QA/QC • Could result in uneven finished surface of soil-cement mix cap that may be undesirable for the ultimate condition
Alternative 2 soil-cement mix cap by CLSM	• Relatively low cost of CLSM • Reliable QA/QC • Relatively homogeneous product	• Flat landward open cut slope affecting existing roadway • Requires CLSM placement in sections with terraced wooden forms

3.10 SECANT PILE WALL

As discussed in the “Project Description” section, the secant pile wall would consist of overlapping primary unreinforced and secondary reinforced piles (both drilled, cast-in-place concrete piles approximately 3 feet in diameter). Initially, the secant pile wall would be concealed. However, over time as beach recession occurs, the secant pile wall would be exposed (with the seaward side lowered to Elevation +2 feet in front of the wall), resulting in a retaining wall height ranging from approximately 12.5 to 16.5 feet. Ultimately, the landward side of the secant pile wall would become a 3 horizontal to 1 vertical (3H:1V) backslope.

As noted above, a soil-cement cap would be constructed for the ultimate 3H:1V backslope. If this soil-cement cap would be constructed as a continuous blanket running longitudinally along the entire length of the secant pile wall, it could potentially act as a barrier to groundwater flow and may cause the groundwater level behind the wall to rise above design groundwater level. Therefore, adequate drainage should be provided behind the pile cap such as installation of a subdrain system discharging to a suitable free-drainage outlet. The discharge system should be designed properly to avoid any slope instability.

Tiebacks would be installed at the pile cap, extending back into the landside beneath the LMT tunnel with a minimum clearance of 5 feet. Our geotechnical recommendations for tiebacks are presented in the “Tiebacks” section.

The drilled piles for the secant pile wall should be designed so that the vertical, horizontal or rotational loads are within the design and operational limits. In addition to the weight of the wall, pile cap and backfill placed above, the vertical loads on the drilled piles should also include the downward load from the tiebacks. On a preliminary basis, for vertical compression (downward) loads, the drilled piles should be designed for an allowable downward skin friction of 500 pounds per square foot (psf) in dense soils for dead plus live loads. This value includes a factor of safety of 2 may be increased by 1/3 to include wind and seismic loads. Uplift resistance may be calculated to be 75 percent of the skin friction in compression. The drilled piles should extend to a depth below the potentially liquefiable zones with zero skin friction in the liquefiable soils and account for liquefaction-induced downdrag force of 20 tons.

The secant pile wall should be designed to resist lateral earth pressures based on the long-term retaining condition as described above. Our preliminary geotechnical recommendations on lateral earth pressures are presented in the “Lateral Earth Pressures” section.

Based on our review of the existing data and the subsurface conditions encountered in our field exploration for this study, caving and seepage in sandy soils should be expected during drilling of the pile holes. Casing (preferably rotated down with the drilling equipment) or use of slurry displacement method would be required to maintain an open pile hole for installation of reinforcing steel and placement of concrete. Concrete would be required to be placed by tremie method to displace the water out of the pile holes.

It is important to confirm that the drilled piles installed are structurally sound and do not contain significant defects. Therefore, post-construction integrity testing (such as crosshole sonic logging or gamma-gamma) should be performed to evaluate the quality of the completed drilled piles. In general, sonic logging is most suited for integrity evaluation within steel cage and consists of vertical access tubes (steel or PVC pipe) installed in the drilled piles before placing the concrete. Once the drilled piles are completed, a compression wave source is lowered down one tube and a receiver down another while taking readings of the wave propagation through the drilled piles. Voids, if present, will show up as anomalies in the wave propagation pattern. Similarly, gamma-gamma testing ensures sufficient concrete cover over steel cage. The testing utilizes an electric winch to pull a 4-foot probe with the radioactive source at the end, up through PVC pipes installed in the concrete. As the probe moves up through the tubes, it reads average concrete densities at set intervals. These intervals are then plotted and analyzed for average bulk density versus pile depth. Deviation in average bulk density are used to identify pile anomalies or defects and to assess pile/concrete quality.

3.11 LATERAL EARTH PRESSURES

Preliminary lateral earth pressures for CER evaluations of the secant pile wall are presented on Plates 8A to 8E – Preliminary Lateral Earth Pressures and discussed below.

The secant pile wall may be designed to resist active or at-rest earth pressures (depending on whether it is designed as a flexible wall or rigid walls). Active earth pressures should be used for flexible walls that are free to rotate by at least 0.004 x H, where H is the height of the long-term exposed height of the secant pile wall. At-rest earth pressures should be used for rigid walls that are restrained and not capable of this magnitude of movement.

Seismic lateral earth pressures should also be included in the design of the secant pile wall. For seismic condition, the secant pile wall should be designed for the additional seismic pressure increment (see Plates 8A to 8E). The additional seismic pressure increment was computed using the method of Al Atik and Sitar (2009) and should be added to active earth pressures.

As discussed in the "Liquefaction" section, if the soils behind the secant pile wall liquefy during a major earthquake, the lateral earth pressure exerted on the wall would be momentarily increased due to liquefaction-induced excess pore water pressure. For those soils that will be subjected to liquefaction behind the wall, the liquefaction-induced lateral earth pressure can be calculated using an equivalent fluid pressure of 120 pcf (based on an active earth pressure coefficient, K_A, of 1). The liquefaction-induced lateral earth pressure and the seismic lateral earth pressure discussed above are two different scenarios that will not occur simultaneously. The secant pile wall design should be checked against both to see which scenario is more critical.

Where applicable, the increase in lateral earth pressure due to backfill compaction should be considered. The additional compaction-induced earth pressure is presented on Plate 9.

There may be an intervening stage in which the backslope is steeper than 3H:1V when the material in front of the wall has been eroded. Since the backslope is engineered fill (or improved soils), it is possible that the intermediate backslope would be temporarily a very steep or locally a subvertical slope.

If vertical surcharge loads are anticipated within the zone above an imaginary 45-degree line projected up from the long-term exposed bottom of secant pile wall (Elevation +2 feet), the additional lateral earth pressures from the surcharge should be included in the secant pile wall design. The nominal lateral earth pressure from traffic load based on an equivalent soil height of 1 foot (vertical surcharge of 150 psf), where applicable, is shown on Plates 8A-8E. For other surcharge loads (such as heavy construction equipment or stockpiled materials) that may occur within the zone above the imaginary 45-degree line, AGS should be consulted to estimate the effect of such surcharge loads.

In addition to tiebacks, lateral loads would be resisted by passive earth pressures acting against the long-term embedded portion of the secant pile wall. Passive earth pressures are included on Plates 8A-8E. Table 12 shows the soil properties used for each of the material layers used in the development of the lateral earth pressures.

3.1 BUOYANCY RESISTANCE

Based on our review of the 2015 Ocean Beach Master Plan Coastal Management Framework (CMF), we understand that Jacobs Associates performed numerical modeling studies to assess the vulnerability of the LMT to bluff retreat and loss of existing overburden. The results of their numerical modeling studies were presented in a report (dated September 23, 2014) incorporated as Appendix 4 of the 2015 CMF.

During our review of the 2014 Jacobs Associates report, we noted that a long-term condition (Condition 3) was analyzed with a groundwater level at the crown of the LMT tunnel (approximately Elevation +10 to +13 feet) to represent a fully buoyant tunnel empty of effluent. The study found that "it would require at least 6 feet of cover on top of the tunnel to counterbalance the buoyant forces exclusive of any safety factor." As discussed in the "Design Groundwater Level" section, we recommend a design groundwater level at Elevation +16 feet, which ranges from approximately 1 to 5 feet above the crown of the LMT. We recommend that the minimum 6 feet of cover on top of the tunnel (to counterbalance the buoyant forces) be checked against our recommended design groundwater level at Elevation +16 feet.

TABLE 12
SOIL PROPERTIES FOR LATERAL EARTH PRESSURES

Reach	Design Groundwater Elevation (NGVD) (Feet)	Layer Description	Top of Layer Elevation (NGVD) (Feet)	Thickness (feet)	Total Unit Weight (pcf)	Phi (deg)	Cohesion (psf)
North	+16	Silty Gravelly Sand	+31	20	120	33	0
		Poorly Graded Sand	+11	10	120	34	0
		Poorly Graded Sand with Silt	+1	50	125	36	0
		Silty Sand and Sandy Silt	-49	>30	125	27	300
EQR	+16	Silty Gravelly Sand	+31	15	120	33	0
		Poorly Graded Sand	+16	5	120	34	0
		Poorly Graded Sand with Silt	+11	50	125	36	0
		Silty Sand and Sandy Silt	-39	>20	125	27	300
Rubble	+16	Silty Gravelly Sand	+31	10	120	33	0
		Poorly Graded Sand	+21	10	120	34	0
		Poorly Graded Sand with Silt	+11	30	125	36	0
		Silty Sand and Sandy Silt	-19	>30	125	27	300
Bluff	+16 to +21	Silty Gravelly Sand	+36	15	120	33	0
		Poorly Graded Sand	+21	10	120	34	0
		Poorly Graded Sand with Silt	+11	40	125	36	0
		Silty Sand and Sandy Silt	-29	>10	125	27	300
South	+21	Silty Gravelly Sand	41	30	120	33	0
		Poorly Graded Sand	11	10	120	34	0
		Poorly Graded Sand with Silt	1	30	125	36	0
		Silty Sand and Sandy Silt	-29	>30	125	27	300

3.2 TIEBACKS

3.2.1 Design Criteria

We understand that, due to the long-term exposed height of the secant pile wall ranging from approximately 15 to 18 feet and a 3H:1V backslope, tiebacks would be installed to provide the necessary lateral support. The subsurface conditions on site generally consisting of sandy soils below groundwater would be susceptible to caving. The drilling method to install tiebacks at various locations should consider the potential for caving. Where caving is anticipated to occur, drilling fluids or casing should be used to stabilize the drill hole.

Based on the current concept plans, the tiebacks are being proposed to be installed at an inclination of 1½H:2V (approximately 53 degrees below the horizontal). We understand that this relatively steep angle of installation is to meet the required clearance with the LMT tunnel and to keep the construction work within the project limits.

Tiebacks are typically installed at inclination between 15 and 30 degrees below the horizontal and inclination up to 45 degrees below the horizontal can generally be installed by most contractors. If possible, consideration should be given to moving the secant pile wall further seaward (perhaps by approximately 5 feet). This would allow easier installation of tiebacks at the more common 45 degrees (or less) to attract more qualified contractors and to increase in tieback efficiencies.

To minimize the potential effects of liquefaction-induced settlement on the tiebacks, the bonded section of the tiebacks should be located entirely above the potentially liquefiable zones identified below the LMT as shown in Table 13 for the five reaches.

TABLE 13
TOP OF LOWER LIQUEFACTION ZONE

Reach	Depth to Top of Lower Liquefaction Zone Elevation (feet)
North	50-55
EQR	40-60
Rubble	30-45
Bluff	32-33
South	30-45

For preliminary design purposes, an allowable soil/grout bond strength of 2,000 psf (beyond the active zone defined by a plane extending up at an angle of 60 degrees with the horizontal, from the long-term exposed ground surface in front of the secant pile wall at Elevation +2 feet, or in soils below potential liquefaction zone, whichever is the deeper) may be considered. This preliminary allowable soil/grout bond strength includes a factor of safety of 2. It should be noted that the bond strength of tiebacks will depend on the construction method used by the contractors. The project specifications should allow for modification of the bond strength based on values that are demonstrated from field verification testing.

The tiebacks should be designed for a marine environment anticipated in the long-term condition. Double corrosion protection would be required with factory pre-grouted encapsulation of the bar within a corrugated plastic sheath. Also, the tieback system should be re-stressable, if needed, when the top of the secant pile wall is exposed in the future.

3.2.2 Testing and Acceptance Criteria

We recommend that at least two sacrificial tiebacks (at each reach) be selected for verification testing to verify the bond strength used in the design. All production tiebacks should be proof-tested to at least 1.5 times the design load. Detailed recommendations on verification and proof testing procedures would be provided in our geotechnical interpretive report (GIR). The verification and proof testing should be performed under the observation of the project geotechnical engineer.

3.2.3 Tie-back-induced Downdrag Forces

As noted above, in addition to the weight of the wall, pile cap and backfill placed above, the vertical loads on the drilled piles should also include the downdrag forces from the tiebacks and liquefaction-induced settlement. The downdrag force from the tiebacks is essentially the vertical component of the tieback load. Therefore, by increasing the inclination of the tiebacks, the vertical component of the tieback load also increases, thus increasing the vertical load on the secant pile wall and the underlying foundation material. Our estimated downdrag force on the secant pile wall from liquefaction-induced settlement has been discussed in the “Secant Pile Wall” section for the five reaches. The downdrag force on the secant pile wall from tiebacks can be estimated from the equation: $F \times \sin \alpha$, where F is the design load in the tieback and α is the inclination of the tieback below horizontal.

3.3 CONTROLLED LOW STRENGTH MATERIAL (CLSM)

The use of CLSM may be considered for use as the soil-cement mix cap of the ultimate 3H:1V backslope. The requirements of CLSM for the soil-cement mix cap should include:

1. The in-situ density should be no more than 130 pcf;

2. If the CLSM needs to be easily excavatable in the future, the 28-day unconfined compressive strength should be no less than 50 pounds per square inch (psi) and not more than 150 psi;
3. If the CLSM does not need to be easily excavatable in the future, the 28-day unconfined compressive strength should also be no less than 50 psi but can be higher than 150 psi;
4. The physiochemical properties should not be harmful to the LMT tunnel; and
5. The slump should be less than 12 inches but not less than 6 inches.

3.4 EARTHWORK

3.4.1 Site Preparation

The work limits should be properly marked and traffic controlled in accordance with City and County of San Francisco requirements, and then cleared of any obstructions, including pavements and any debris hindering work. Vegetation and landscaping (if any) in the construction areas should be stripped and disposed of outside the construction limits. Safety fencing should be installed in accordance with OSHA, and all other applicable requirements, including warning fencing placed near the edge of deep open excavations and silt fencing or other environmental protective fencing required by environmental compliance manager. Affected structures, equipment, and debris should be abandoned, disassembled, or demolished and disposed of outside the construction limits. Based on our review of the LMT tunnel as-built plans, there is an existing Army Bunker with invert at approximately Elevation +23½ feet near the south end of the secant pile wall (approximately Station 42+00). It is anticipated that the secant pile wall would have to either locate away from the existing Army Bunker or bridge over it. Likewise, the secant pile wall would also have to be designed for bridging over the existing 12-foot by 12-foot SWOO structure at approximately Station 36+50.

Existing underground utilities located within the project site, if affected by construction activities, should be relocated or protective measures taken prior to construction. All debris generated from the demolition of underground utilities, including abandoned pipes, should be removed from the site as construction proceeds.

During excavation, any observed soft or loose zones should be compacted in-place or excavated and replaced with properly compacted backfill. Upon completion of excavation, backfill may be placed in accordance with the recommendations presented in the following sections.

3.4.2 Excavation Characteristics

The Contractor should review the available data, in order to independently evaluate the type of equipment required to complete the proposed excavations to the required depths. Based on our review of the existing data and the subsurface conditions encountered in our field exploration for this study, it appears that conventional earth moving equipment may be used to remove most of the on-site soils. Existing underground utilities or other structures may require jackhammering or hoe-ram to remove.

3.4.3 Unshored Excavations

During construction, the contractor must maintain safe and stable slopes and provide shoring as necessary. All cuts deeper than 4 feet must be sloped or shored in accordance with the current requirements of OSHA and Cal-OSHA. Shallow excavations above the groundwater level may be sloped if space permits. Soils at the site appear to generally be OSHA Class C soils, and may be sloped no steeper than 1.5H:1V. Sloping of excavations should conform to OSHA requirements, and should be monitored by the contractor to verify stability to ensure worker safety.

Heavy construction equipment, building materials, and excavated soil should be kept away from the edge of the excavation at least a distance equal to, or greater than, the depth of the excavation.

During wet weather, runoff water should be prevented from entering excavations, and collected and disposed of outside the construction limits. To prevent runoff from entering the excavation, a perimeter berm may be constructed at the top of the slope. In addition, it is recommended that the sidewalls of the excavation be covered by plastic sheets to prevent saturation of the earth material.

3.4.4 Fills and Backfills

Fills and backfills may be placed under and around the pile cap of the secant pile wall, utility trenches, and pavement during construction of this project.

Fills and backfills may either be structural or nonstructural. Structural fills and backfills are those defined as providing support to foundations, and pavements. Nonstructural fills and backfills include all other fills such as those placed for landscaping, and not planned for future structural loads. Structural fills and backfills should be compacted to at least 95 percent relative compaction (as determined by ASTM D1557-12); nonstructural fills and backfills should be compacted to at least 90 percent relative compaction.

Due to the concern of potential damage that may be caused by compaction of fill and backfill to the existing LMT tunnel, the use of heavy compaction equipment directly above the LMT tunnel should be avoided. In those areas, the addition of a layer of geotextile (such as Mirafi 600x or approved equivalent) placed underneath the CLSM (if used as the soil-cement mix cap for the ultimate 3H:1V backslope) could be considered.

All structural fills and backfills should be granular fills with no pieces larger than 3 inches in any dimension, no more than 20 percent passing the No. 200 sieve, a Liquid Limit of 35 or less, a Plasticity Index of 12 or less, and should be placed in 8-inch lifts, moisture-conditioned to near-optimum moisture, and compacted to 95 percent relative compaction (as determined by ASTM D1557-12). Non-structural fills should meet the same requirements, but should be compacted to at least 90 percent relative compaction.

Samples of imported fill and backfill materials should be submitted to the geotechnical engineer prior to use for testing to establish that they meet the above criteria.

The existing on-site soils are generally suitable from a geotechnical perspective for use as engineered fill, provided they are free of debris, hazardous materials and other deleterious matter.

The fill and backfill materials should be placed and compacted under the full time observation and testing of the project geotechnical engineer.

3.5 DEWATERING DURING CONSTRUCTION

Groundwater levels at the site will fluctuate due to rain and other causes. As discussed above, we recommend a design groundwater level of Elevation +16 feet in the North, EQR and Rubble Reaches and a design groundwater elevation of +21 feet in the Bluff and South Reaches. Therefore, excavations for construction of the pile cap and installation of tiebacks for the secant pile wall may extend below the groundwater level.

The contractor should make an independent evaluation of the groundwater levels at the site, and be responsible for providing an adequate dewatering system during construction. A properly designed, installed, and operated dewatering system should accomplish the following:

- Lower the groundwater table inside the excavation or intercept seepage which will emerge from the sides or bottom of the excavation;

- Improve the stability of the excavation and prevent disturbance of the bottom of the excavation;
- Provide a reasonably dry working area in the bottom of the excavation; and
- Provide for collection and removal of surface water and rainfall.

During excavation for construction, it is recommended that the water level be maintained at least two feet below the bottom of the excavation until construction is complete, and until the weight of the constructed structure (or installed utilities) is sufficient to resist buoyancy. Selection of the equipment and methods of dewatering should be left up to the contractor, and the contractor should be aware that modifications to the dewatering system may be required during construction, depending on conditions encountered.

The hydraulic conductivities of the subsurface materials vary in response to the heterogeneous, anisotropic media. Within the proposed excavation depth for construction of the secant pile wall (including construction of pile cap and installation of tiebacks), granular deposits were generally encountered in the upper 20 to 30 feet. Granular deposits encountered in our borings generally consist of poorly graded sand with silt, silty sand, and clayey sand with hydraulic conductivities probably in the range of $1x10^{-1}$ to $1x10^{-3}$ cm/s.

Water collected during dewatering should be tested for contamination prior to its disposal. Because the potential for contamination of groundwater was not explored in this study, recommendations are not given herein for proper disposal of collected water.

3.6 FLEXIBLE PAVEMENT

We assume that Great Highway will be rerouted. For the new access road to OWPCP, the new asphalt concrete pavement should be designed based on the Caltrans Flexible Pavement Design Method with an assumed R-Value of 15 and Traffic Index (TI) as determined by the project civil engineer.

The uppermost 12 inches of all pavement subgrade soils should be moisture conditioned to near optimum moisture content and compacted to at least 95 percent relative compaction (as determined by ASTM D1557-12) to provide a smooth, unyielding surface. All fill and backfill materials should be placed in lifts not exceeding approximately 8 inches in loose thickness. If zones of soft or saturated soils deeper than 12 inches are encountered during excavation and compaction, deeper excavations may be required to expose firm soils. This should be determined in the field by the project geotechnical engineer.

Class 2 aggregate base should be placed in thin lifts in a manner to prevent segregation; uniformly moisture conditioned; and compacted to at least 95 percent relative compaction to provide a smooth, unyielding surface.

The performance of pavements will be dependent upon a number of factors, including subgrade conditions at the time of paving, runoff, and loading. Runoff should not be allowed to seep below pavements from adjacent areas. Proper drainage below the pavement section helps prevent softening of the subgrade and has a significant impact on pavement performance and pavement life. Periodic maintenance should be performed throughout the life of the proposed pavements including periodic seal coats and crack maintenance/sealing.

Should import material be used to establish the proper grading for the new pavement, the import material should be approved by the project geotechnical engineer before it is brought to the site. The select import material should meet the following requirements:

- Have an R-value of not less than 30;
- Have a Plasticity Index not higher than 10;
- Not more than 15 percent passing the No. 200 sieve;
- No rocks larger than 3 inches in maximum size;
- Have a pH of 6.5 to 7.5;
- Have a minimum resistivity of 5000 ohms/cm; and
- Have a maximum soluble sulfate content of 0.2 percent by weight.

3.7 CORROSION POTENTIAL

The corrosivity test results are discussed in section 3.2. Based on the soil resistivity classification presented by National Association of Corrosion Engineers (2010) and the results of corrosivity testing at the site, the onsite soils are classified as “extremely corrosive” to “moderately corrosive”. According to ACI 318-11, the sulfate concentration measured in one of the corrosivity samples tested for this study indicates a Soil Exposure Class S1.

Corrosive soils may adversely affect the foundations and buried utilities. AGS recommends that all buried metal piping and reinforced concrete be properly protected against corrosion depending upon the critical nature of the structure. A Corrosion Engineer should be consulted for the development of long-term site-specific corrosion protection measures.

4.0 CLOSURE

This report has been prepared in accordance with generally accepted professional geotechnical engineering practice for the exclusive use of SFPUC for the proposed South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection Project in San Francisco, California. No other warranty, express or implied, is made.

The initial geotechnical findings and preliminary recommendations presented in this report are based on the data obtained from the borings and CPTs performed for this study, and other geotechnical information previously obtained by others in the project area. The nature and extent of variations between the borings and CPTs may not become evident until construction. In the event variations appear, it may be necessary to reevaluate the findings and recommendations presented herein.

The information in this report is primarily intended for use by design engineers. It is the responsibility of the owner or its representative to ensure that the applicable provisions contained herein are incorporated into the plans and specifications and that the necessary steps are taken to see that the contractor carry out such provisions in the field.

The use of this report or its contents requires prior consent of AGS. In addition, the use of any information contained in this report for purposes other than those expressly stated is at the user's own risk.

Respectfully submitted,
AGS, Inc.

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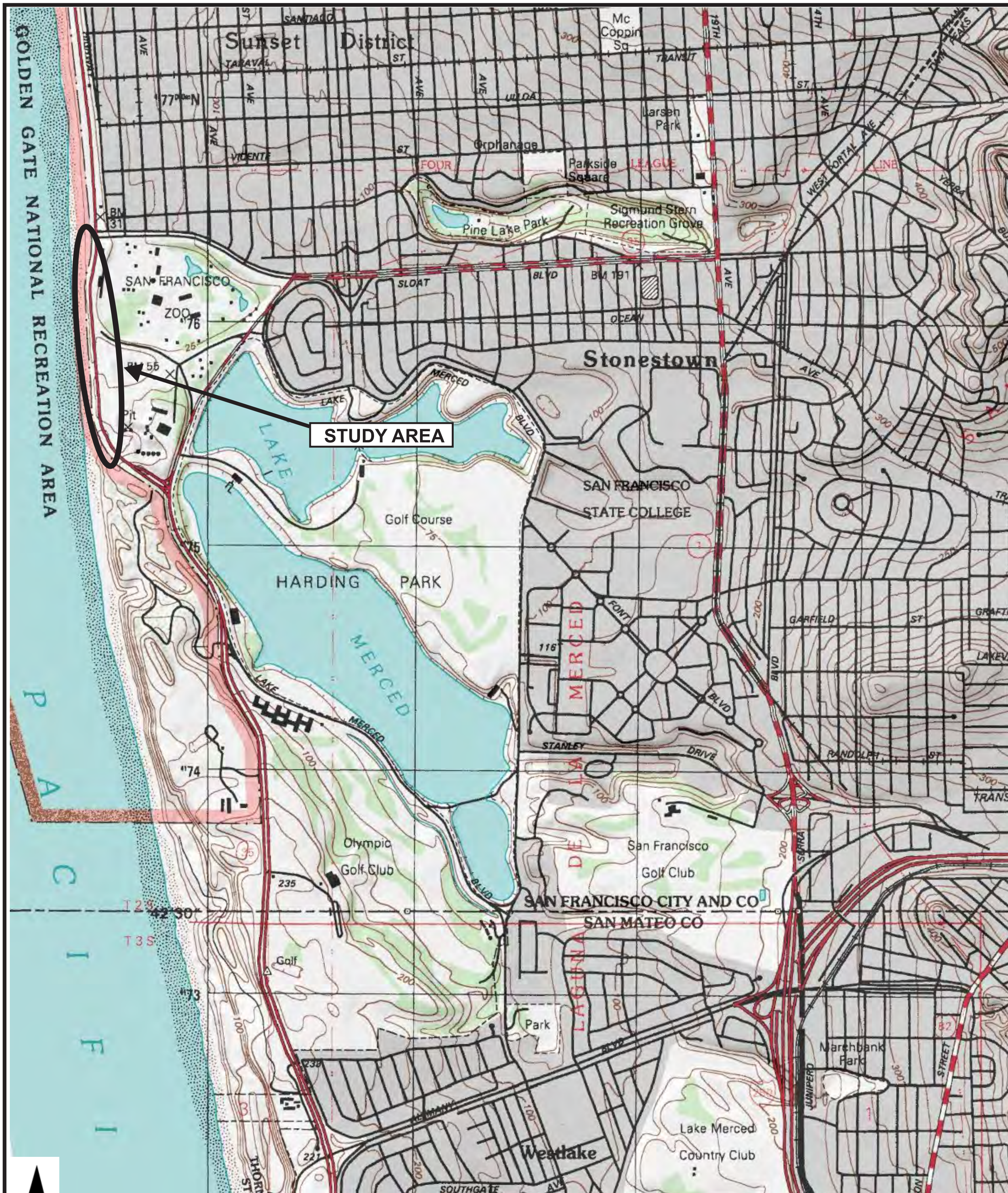
PLATES



Source: USGS, 7.5 Minute Topographic Map of the San Francisco South (2018) Quadrangle.

SITE VICINITY MAP SOUTH OCEAN BEACH COASTAL EROSION WW INFRASTRUCTURE PROTECTION PROJECT SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: APR. 2019	PLATE 1

N

SCALE

0 150 300 feet

Approximate Scale

LEGEND

B-6 Boring Location

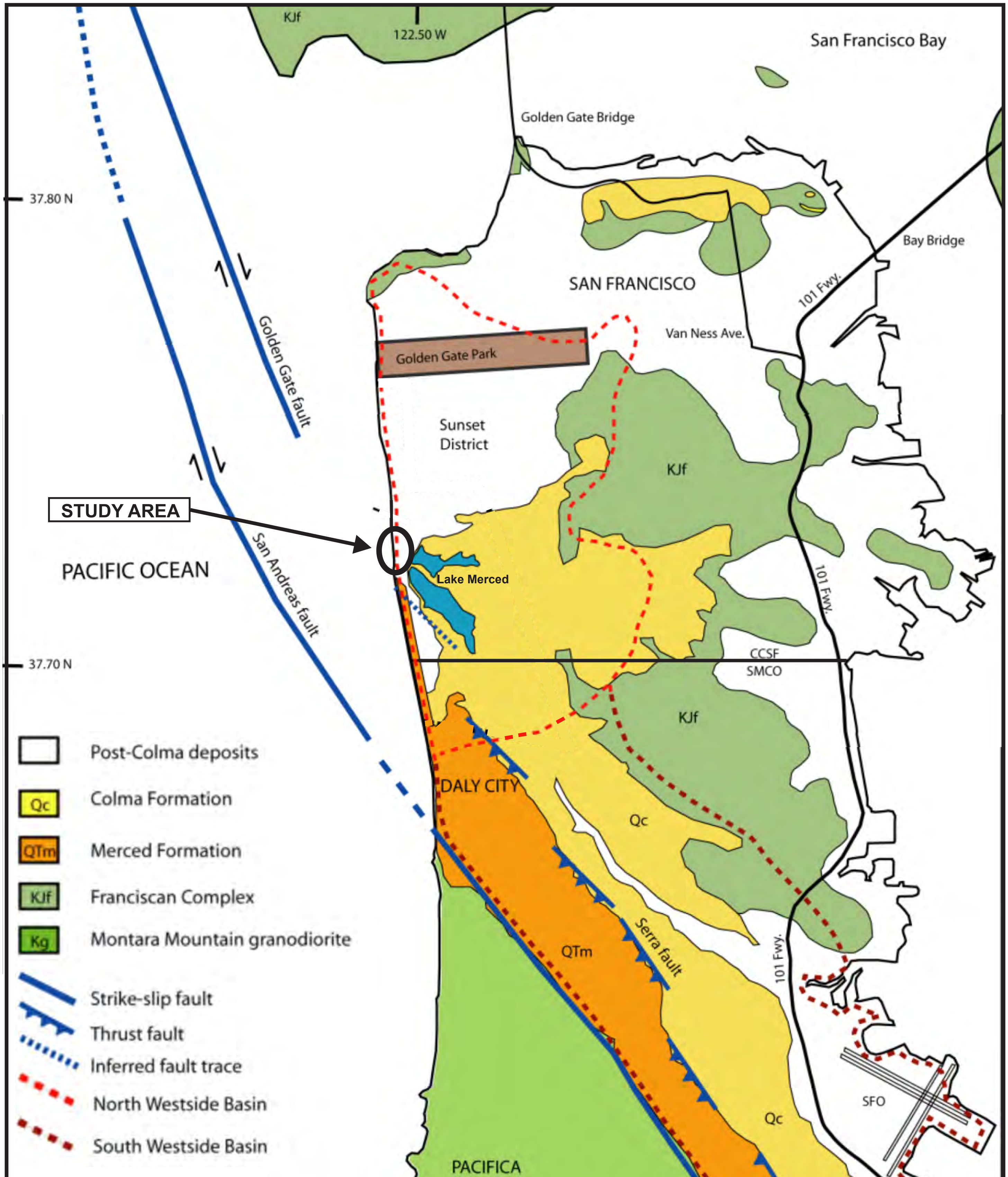
CPT-13 CPT Location

PH-2b Potholing Location

Approximate location of Seismic Survey Array

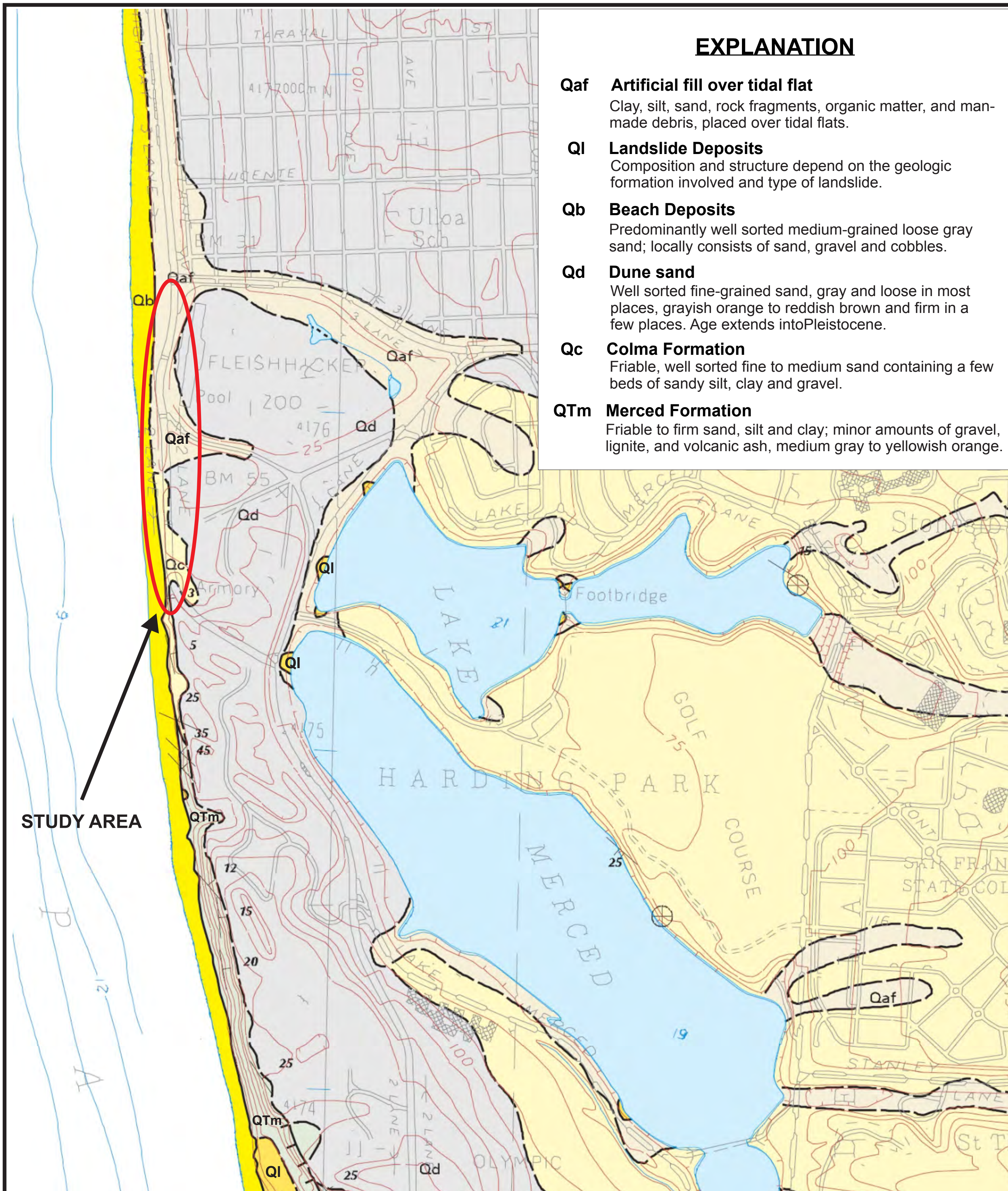
Approximate location of Southwest Ocean Outfall (SWOO)

SITE LOCATION PLAN SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: APRIL 2019	PLATE 2



Source: McGuire (2009), Adapted from Clifton and Hunter (1999)

REGIONAL GEOLOGY MAP SOUTH OCEAN BEACH COASTAL EROSION WW INFRASTRUCTURE PROTECTION PROJECT SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: APR. 2019	PLATE 3A



EXPLANATION

Qaf **Artificial fill over tidal flat**
Clay, silt, sand, rock fragments, organic matter, and man-made debris, placed over tidal flats.

Ql **Landslide Deposits**
Composition and structure depend on the geologic formation involved and type of landslide.

Qb **Beach Deposits**
Predominantly well sorted medium-grained loose gray sand; locally consists of sand, gravel and cobbles.

Qd **Dune sand**
Well sorted fine-grained sand, gray and loose in most places, grayish orange to reddish brown and firm in a few places. Age extends intoPleistocene.

Qc **Colma Formation**
Friable, well sorted fine to medium sand containing a few beds of sandy silt, clay and gravel.

QTm **Merced Formation**
Friable to firm sand, silt and clay; minor amounts of gravel, lignite, and volcanic ash, medium gray to yellowish orange.

Source: Bonilla, 1998, USGS Preliminary Geologic Map of the San Francisco South 7.5' Quadrangle

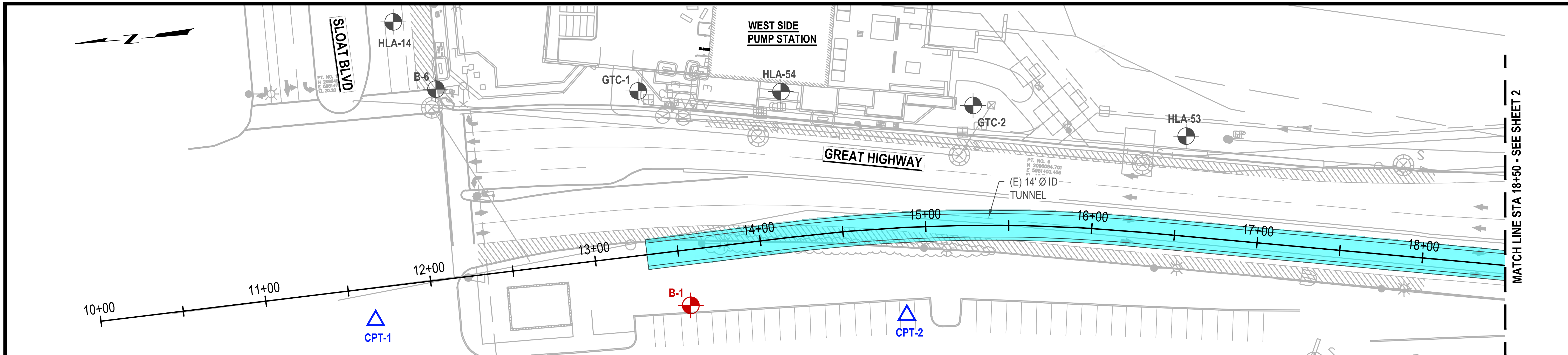
1000 0 1000 2000 ft
Scale

LOCAL GEOLOGY MAP

SOUTH OCEAN BEACH COASTAL EROSION
WW INFRASTRUCTURE PROTECTION PROJECT
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: APR. 2019	PLATE 3B



PLAN
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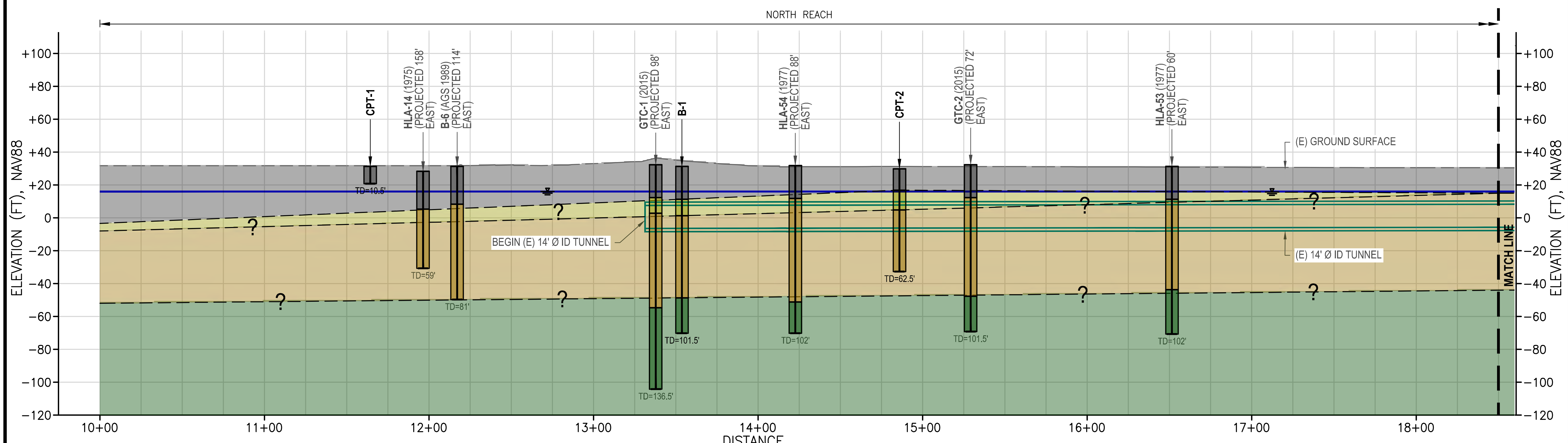


PROFILE
HOR.: 1"=60'
VERT.: 1"=60'

LEGEND

- ARTIFICIAL FILL
- DUNE SAND
- COLMA FORMATION
- MERCED FORMATION
- ESTIMATED DESIGN GROUNDWATER
- B-1 TD=TOTAL DEPTH (FT)
- 0+00 GEOLOGIC CROSS SECTION
- SOIL BORING (AGS 2019)
- CONE PENETROMETER TEST SOUNDING (AGS 2019)
- PREVIOUS SOIL BORINGS

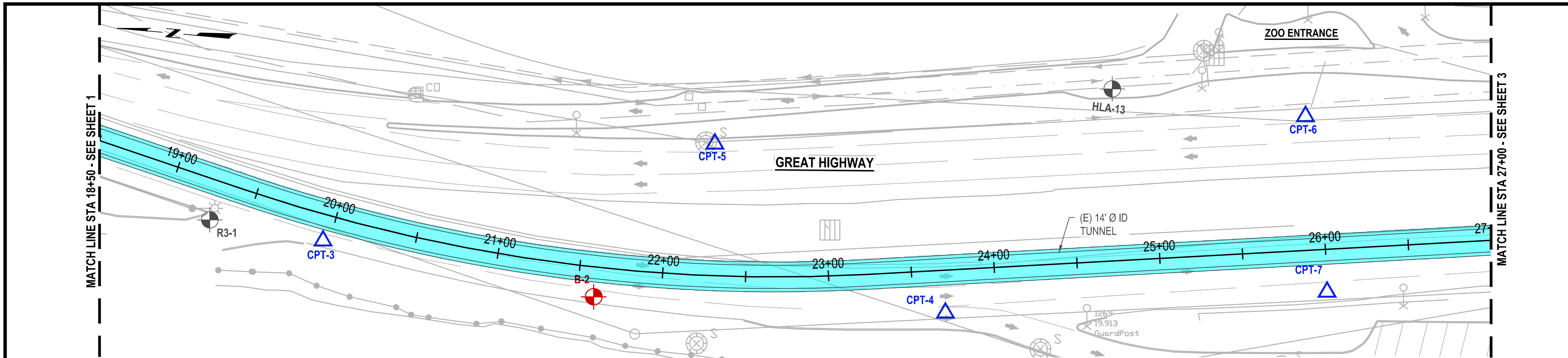
NOTES

1. SFCD = NAV88−11.35 FEET.
2. ORIGINAL PLOT SIZE OF THIS SHEET IS 11"X17".

GENERALIZED SUBSURFACE PROFILE
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO PUBLIC UTILITIES COMMISSION
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO: AGS-18-003	DATE: APR 2019	PLATE 4A



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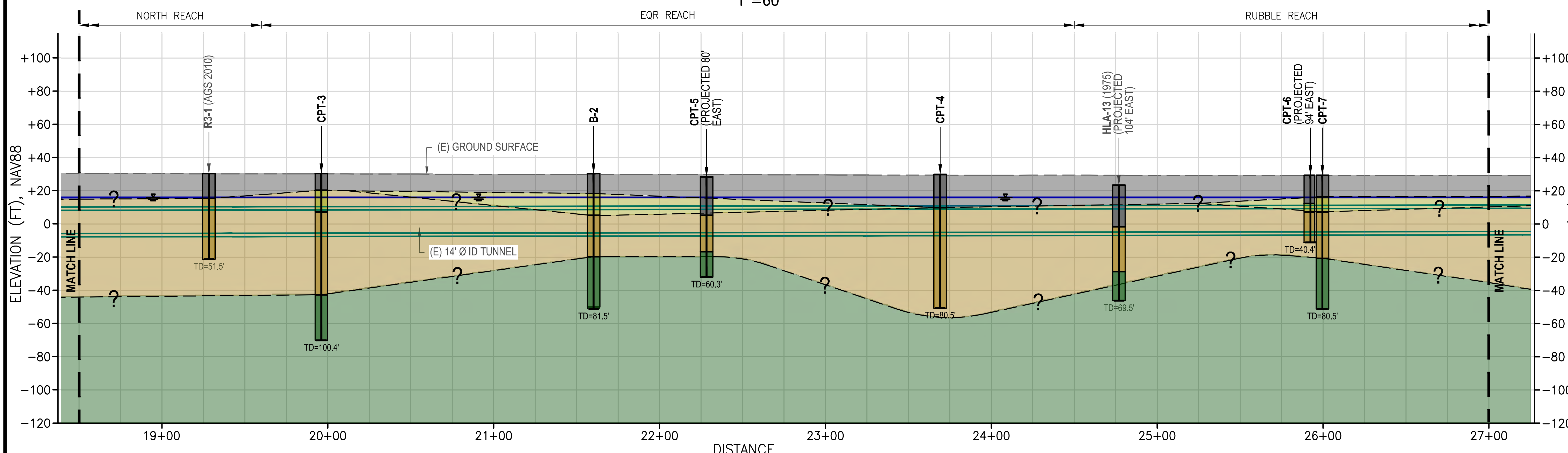


PROFILE
HOR.: 1"=60'
VERT.: 1"=60'

LEGEND

- ARTIFICIAL FILL
- DUNE SAND
- COLMA FORMATION
- MERCED FORMATION
- ESTIMATED DESIGN GROUNDWATER
- B-1 TD=TOTAL DEPTH (FT)
- 0+00 GEOLOGIC CROSS SECTION
- SOIL BORING (AGS 2019)
- CONE PENETROMETER TEST SOUNDING (AGS 2019)
- PREVIOUS SOIL BORINGS

NOTES

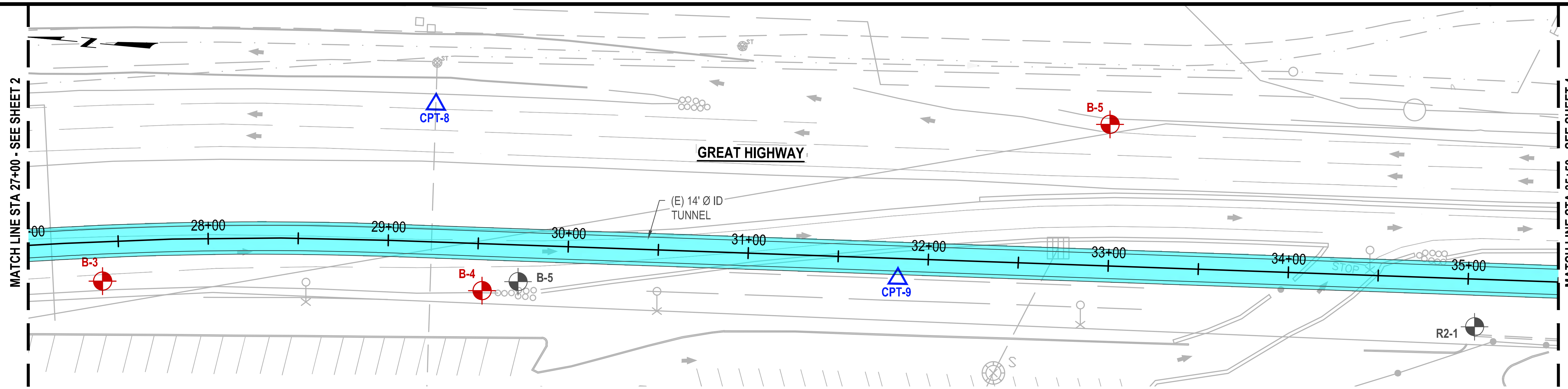
1. SFCD = NAV88−11.35 FEET.
2. ORIGINAL PLOT SIZE OF THIS SHEET IS 11"X17".

GENERALIZED SUBSURFACE PROFILE

SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO PUBLIC UTILITIES COMMISSION
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO: AGS-18-003	DATE: APR 2019	PLATE 4B



PLAN
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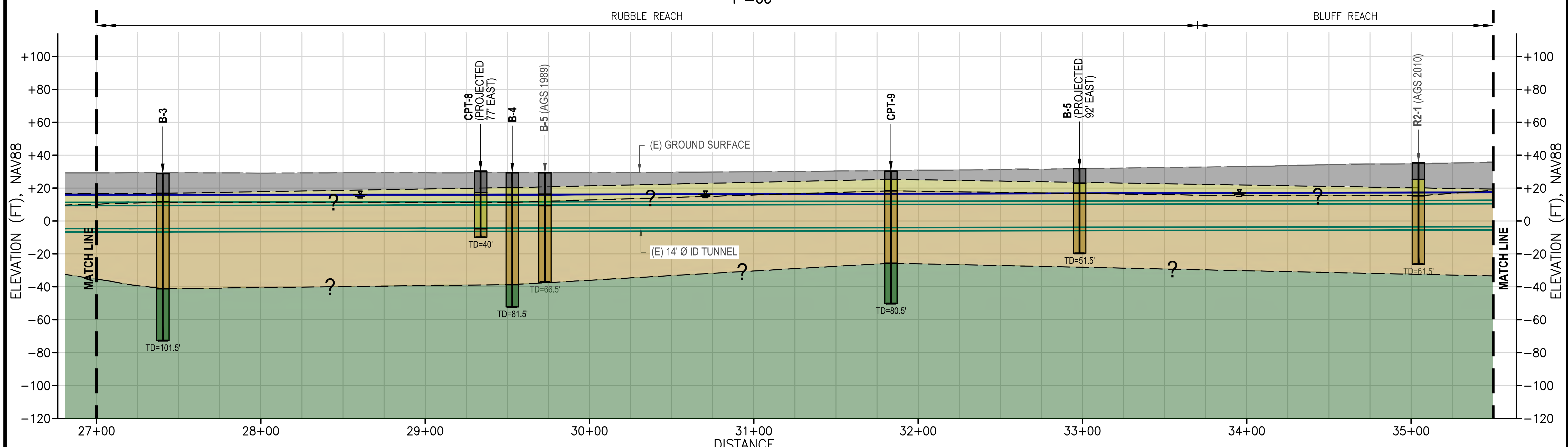


PROFILE

HOR.: 1"=60'
VERT.: 1"=60'

LEGEND

ARTIFICIAL FILL

DUNE SAND

COLMA FORMATION

MERCED FORMATION

ESTIMATED DESIGN GROUNDWATER

B-1

TD=TOTAL DEPTH (FT)

0+00 GEOLOGIC CROSS SECTION

SOIL BORING (AGS 2019)

CONE PENETROMETER TEST SOUNDING (AGS 2019)

PREVIOUS SOIL BORINGS

NOTES

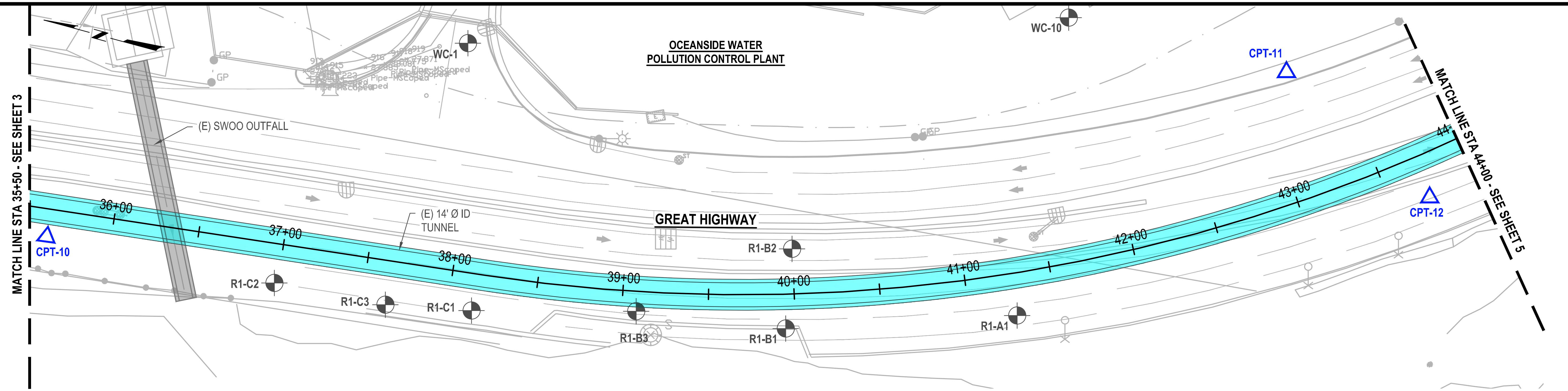
1. SFCD = NAV88−11.35 FEET.
2. ORIGINAL PLOT SIZE OF THIS SHEET IS 11"X17".

GENERALIZED SUBSURFACE PROFILE

SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO PUBLIC UTILITIES COMMISSION
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO: AGS-18-003	DATE: APR 2019	PLATE 4C



PLAN

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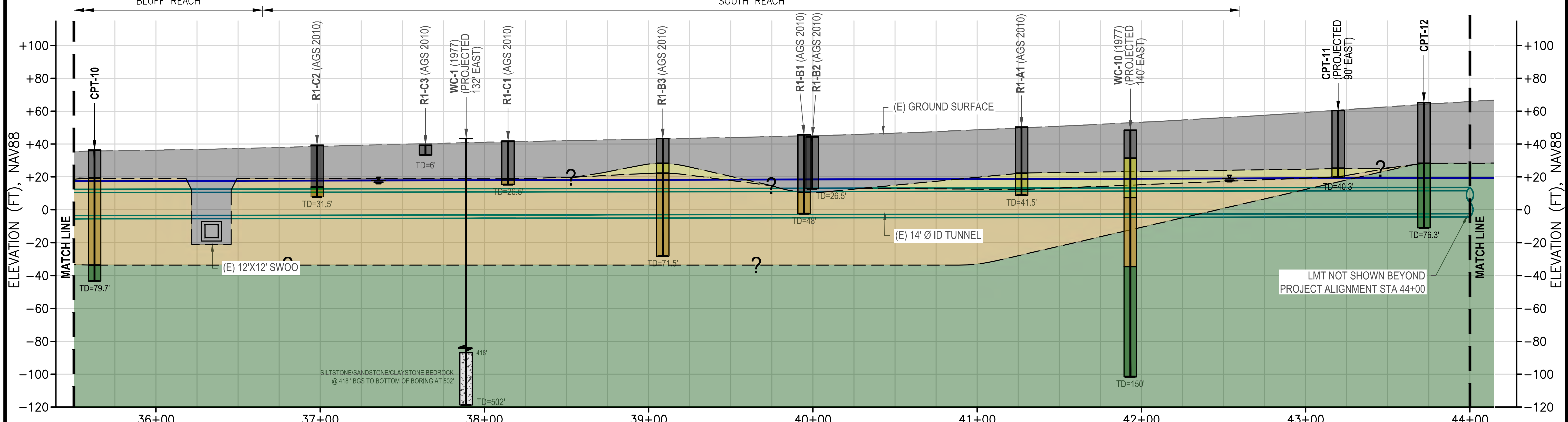


PROFILE

HOR.: 1"=60'
VERT.: 1"=60'

LEGEND

- ARTIFICIAL FILL
- DUNE SAND
- COLMA FORMATION
- MERCED FORMATION
- ESTIMATED DESIGN GROUNDWATER
- B-1 TD=TOTAL DEPTH (FT)
- 0+00 GEOLOGIC CROSS SECTION
- SOIL BORING (AGS 2019)
- CONE PENETROMETER TEST SOUNDING (AGS 2019)
- PREVIOUS SOIL BORINGS

NOTES

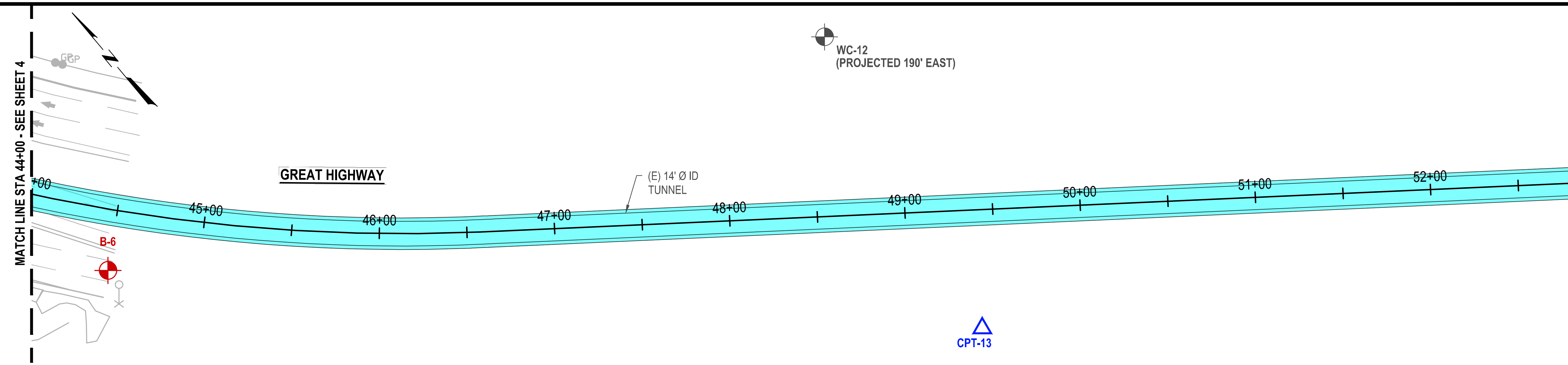
1. SFCD = NAV88−11.35 FEET.
2. ORIGINAL PLOT SIZE OF THIS SHEET IS 11"X17".

GENERALIZED SUBSURFACE PROFILE

SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO PUBLIC UTILITIES COMMISSION
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO: AGS-18-003	DATE: APR 2019	PLATE 4D



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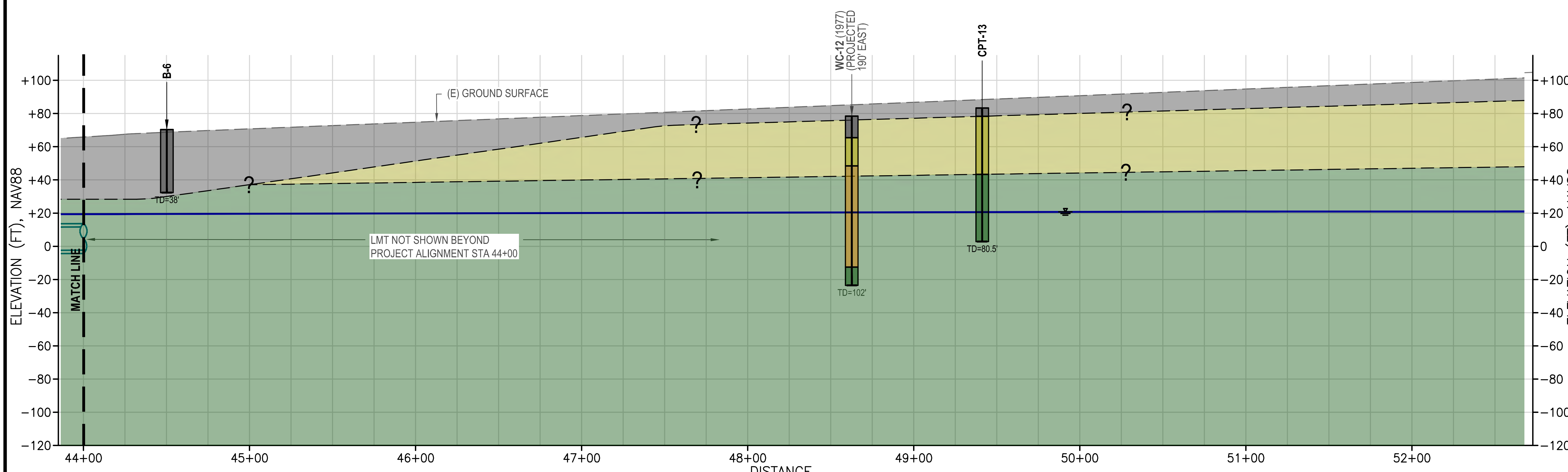


PROFILE
HOR.: 1"=60'
VERT.: 1"=60'

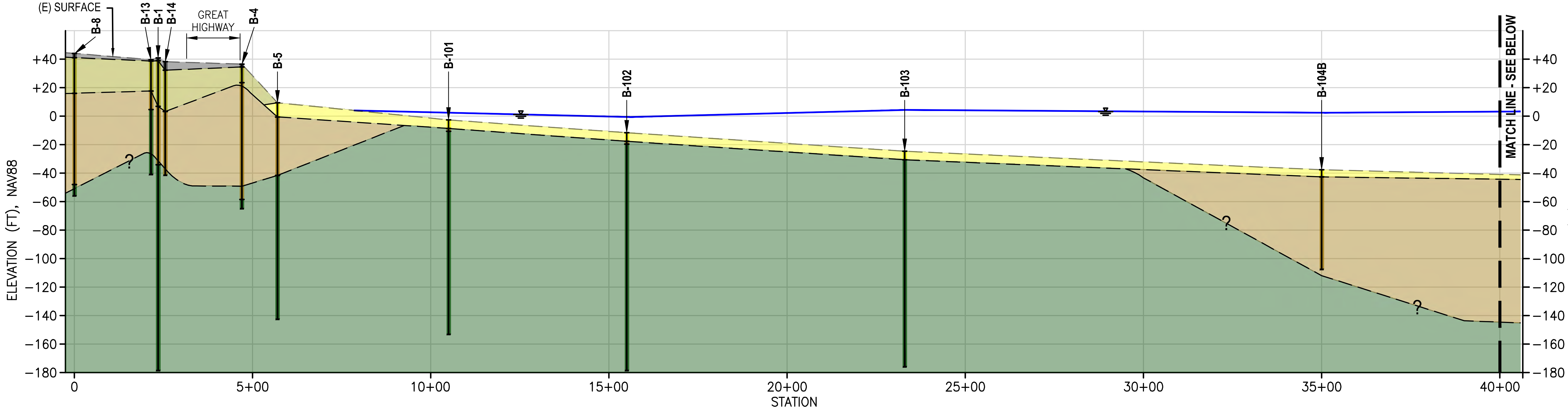
LEGEND

- ARTIFICIAL FILL
- DUNE SAND
- COLMA FORMATION
- MERCED FORMATION
- ESTIMATED DESIGN GROUNDWATER
- B-1 TD=TOTAL DEPTH (FT)
- 0+00 GEOLOGIC CROSS SECTION
- SOIL BORING (AGS 2019)
- CONE PENETROMETER TEST SOUNDING (AGS 2019)
- PREVIOUS SOIL BORINGS

NOTES

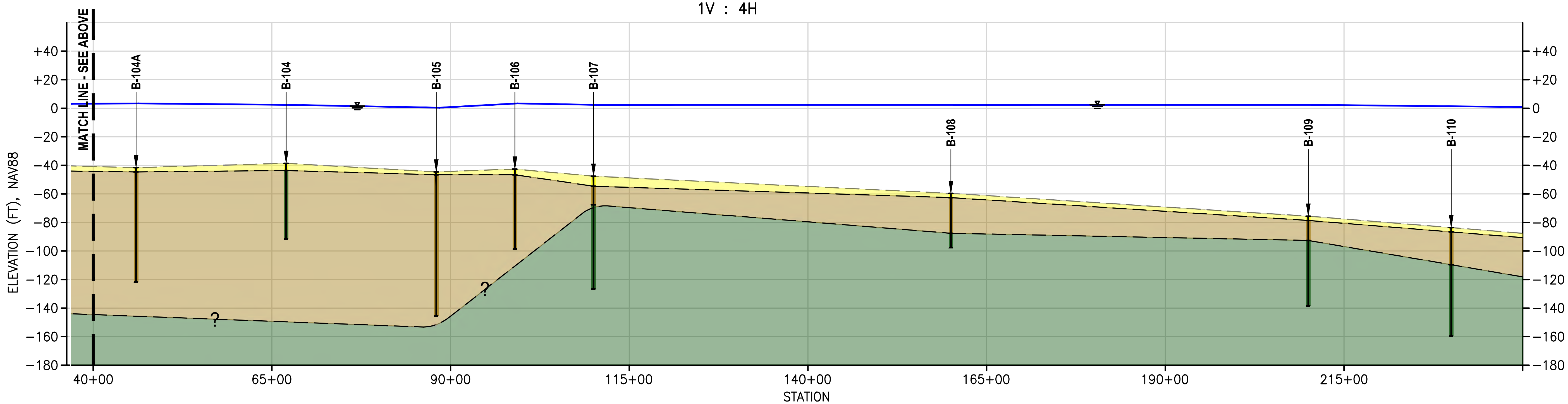
1. SFCD = NAV88−11.35 FEET.
2. ORIGINAL PLOT SIZE OF THIS SHEET IS 11"X17".

GENERALIZED SUBSURFACE PROFILE SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO PUBLIC UTILITIES COMMISSION SAN FRANCISCO, CALIFORNIA		AGS
JOB NO: AGS-18-003	DATE: APR 2019	PLATE 4E



PROFILE

HOR.: 1"=300'
VERT.: 1"=75'
1V : 4H



PROFILE

HOR.: 1"=1500'
VERT.: 1"=75'
1V : 20H

LEGEND

- ARTIFICIAL FILL
- DUNE SAND
- BEACH DEPOSITS
- COLMA FORMATION
- MERCED FORMATION
- WATER LEVEL

B-1

TD=TOTAL DEPTH (FT)

NOTES

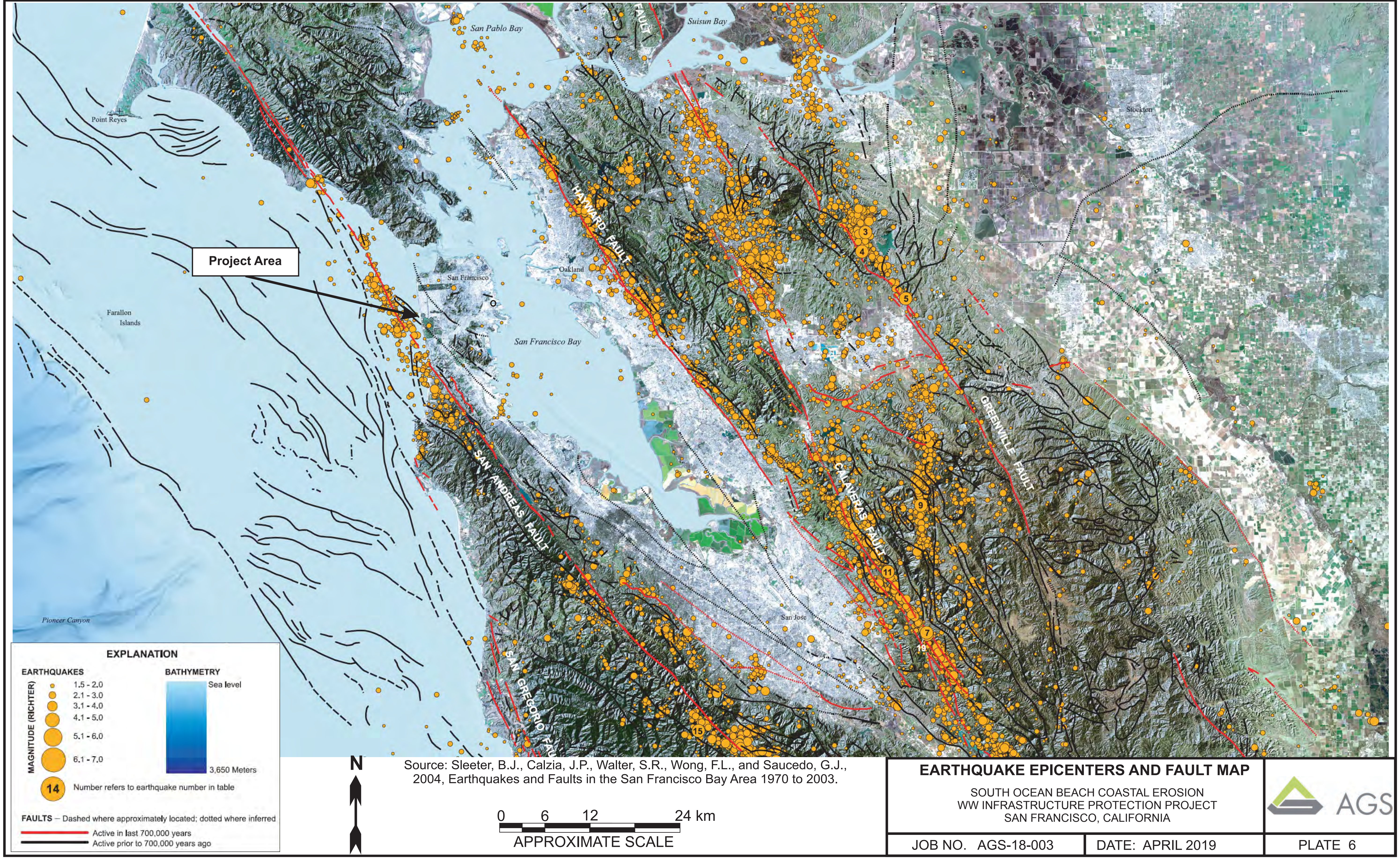
1. NAV88 = SFCD+11.35 FEET
2. PROFILE STATIONING BEGINS AT B-8.
3. SUBSURFACE SHOWN IS BASED ON WOODWARD AND CLYDE BORINGS PRIOR TO CONSTRUCTION OF SWOO.

PRELIMINARY SUBSURFACE PROFILE - SWOO

SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO PUBLIC UTILITIES COMMISSION
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO: AGS-18-003	DATE: APR 2019	PLATE 5



N

Source: Sleeter, B.J., Calzia, J.P., Walter, S.R., Wong, F.L., and Saucedo, G.J., 2004, Earthquakes and Faults in the San Francisco Bay Area 1970 to 2003.

0 6 12 24 km

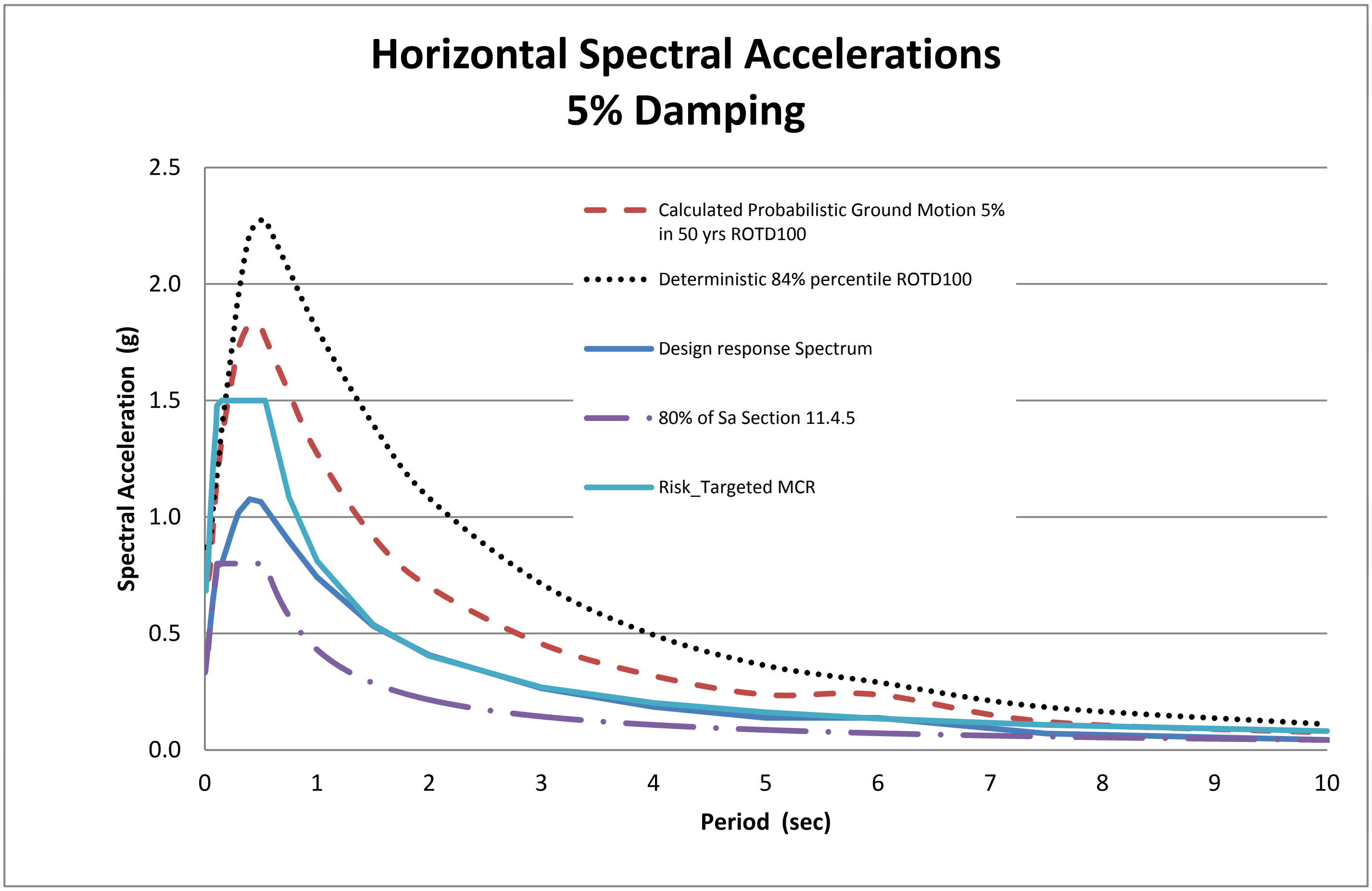
APPROXIMATE SCALE

EARTHQUAKE EPICENTERS AND FAULT MAP

SOUTH OCEAN BEACH COASTAL EROSION
WW INFRASTRUCTURE PROTECTION PROJECT
SAN FRANCISCO, CALIFORNIA

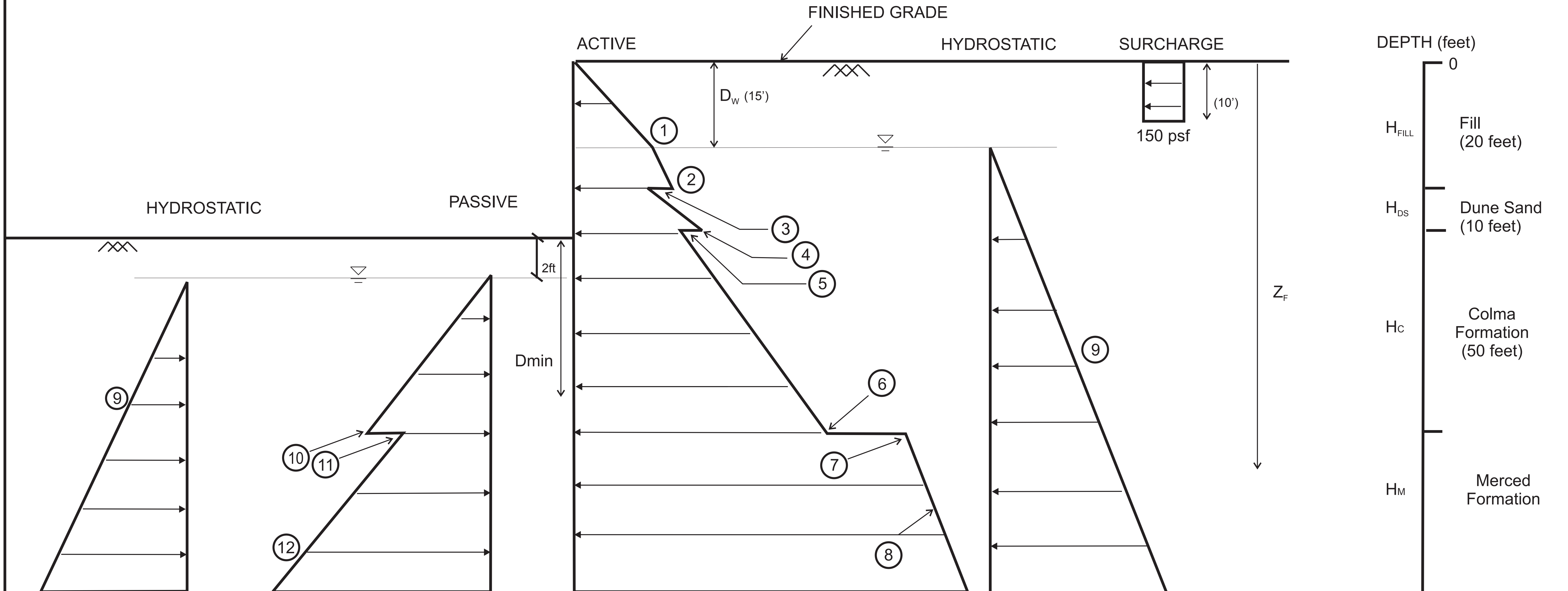
AGS

JOB NO. AGS-18-003	DATE: APRIL 2019	PLATE 6



AGS	**DESIGN RESPONSE SPECTRUM** OCEAN BEACH COASTAL EROSION AND WW INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA	PLATE NO. 7
	Project No.: AGS-18-003 / Date: April 2019	

RESISTING PRESSURES (psf)



NOTES:

1. Add surcharge pressure to active pressures. If surcharge is kept "H" feet behind wall, lateral surcharge will be negligible.

2. Minimum pile penetration (Dmin) for stability purposes only should be determined by balancing driving and resisting pressures.

3. Lateral surcharge pressure is a nominal value for traffic only. Actual values should be based on anticipated surcharges. Additional lateral surcharge pressures should be added to the nominal value shown above, as appropriate.

4. Dynamic Earth Pressure to be added to active pressure
Non-yielding retaining wall - uniform rectangular with base of 32H, where H is height of the wall
Rigid wall - straight triangular shape with base of 45H, where H is height of the wall.

LEGEND:

H = EXCAVATION DEPTH, IN FEET

Z_F = DEPTH BELOW EXISTING GROUND SURFACE, IN FEET

D_w= WATER DEPTH, IN FEET FROM GROUND SURFACE

Dmin = DEPTH TO BOTTOM OF RETAINING WALL, IN FEET

	3H:1V Condition	Intervening Condition
1	650 PSF	1570 PSF
2	750 PSF	1660 PSF
3	730 PSF	1620 PSF
4	930 PSF	1810 PSF
5	850 PSF	1740 PSF
6	1855 PSF	2740 PSF
7	2840 PSF	3725 PSF
8	2840 +25 (Z_f-80) PSF	3725 + 25(Z_f-80) PSF
9	62.4 PCF	
10	8400 PSF	
11	7200 PSF	
12	7200+ 210 (Dmin-50) PCF	

At-Rest Pressure

Use the following multiplier to obtain at rest pressure:
At-rest Pressure = Multiplier x active pressure

Material	Multiplier
Fill	1.53
Dune Sand	1.60
Colma Formation	1.62
Merced Formation	1.45

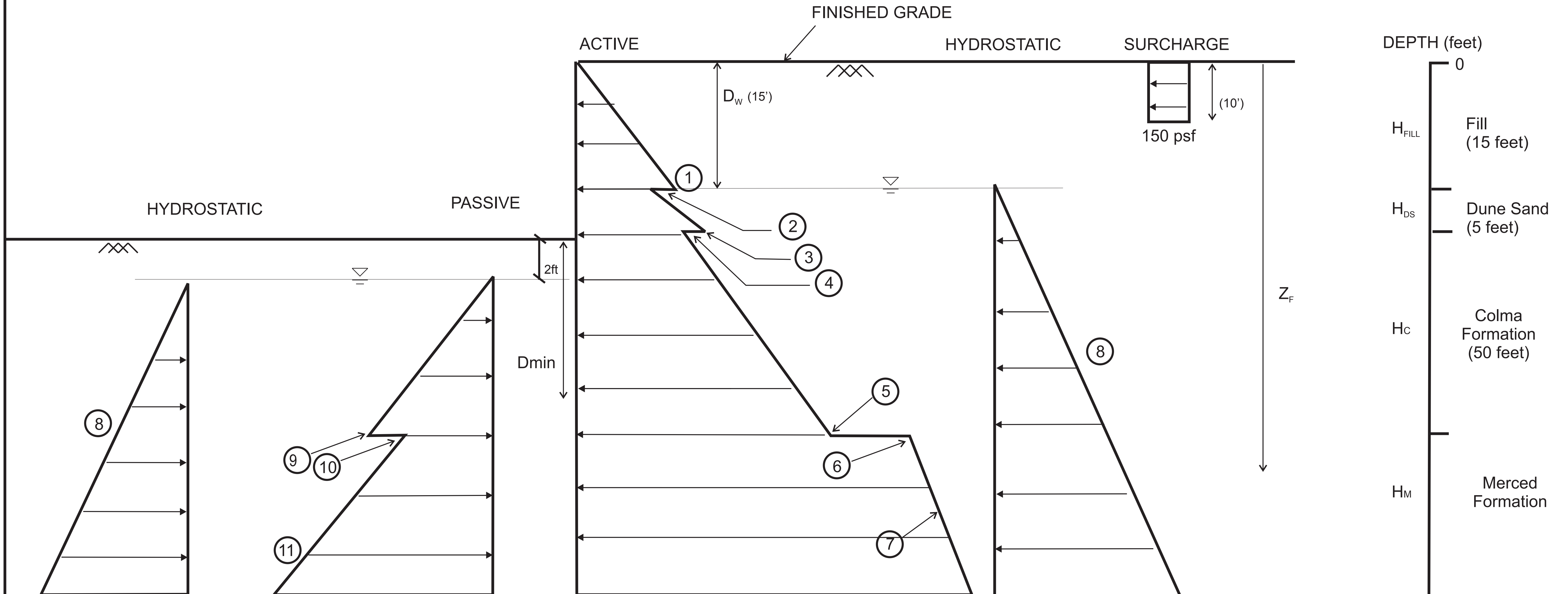
PRELIMINARY LATERAL EARTH PRESSURES
NORTH REACH

SOUTH OCEAN BEACH COASTAL EROSION
WW INFRASTRUCTURE PROTECTION PROJECT
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003 | DATE: APRIL 2019 | PLATE 8A

RESISTING PRESSURES (psf)



NOTES:

1. Add surcharge pressure to active pressures. If surcharge is kept "H" feet behind wall, lateral surcharge will be negligible.

2. Minimum pile penetration (Dmin) for stability purposes only should be determined by balancing driving and resisting pressures.

3. Lateral surcharge pressure is a nominal value for traffic only. Actual values should be based on anticipated surcharges. Additional lateral surcharge pressures should be added to the nominal value shown above, as appropriate.

4. Dynamic Earth Pressure to be added to active pressure
   Non-yielding retaining wall - uniform rectangular with base of 32H, where H is height of the wall
   Rigid wall - straight triangular shape with base of 45H, where H is height of the wall.

LEGEND:

H = EXCAVATION DEPTH, IN FEET

Z_F = DEPTH BELOW EXISTING GROUND SURFACE, IN FEET

D_W= WATER DEPTH, IN FEET FROM GROUND SURFACE

Dmin = DEPTH TO BOTTOM OF RETAINING WALL, IN FEET

	3H:1V Condition	Intervening Condition
1	650 PSF	1570 PSF
2	630 PSF	1360 PSF
3	730 PSF	1620 PSF
4	670 PSF	1410 PSF
5	1670 PSF	2640 PSF
6	2550 PSF	4760 PSF
7	2550 +25 (Z_F-70) PSF	4760 + 25(Z_F-70) PSF
8	62.4 PCF	
9	8400 PSF	
10	7200 PSF	
11	7200+ 210 (Dmin-50) PCF	

At-Rest Pressure

Use the following multiplier to obtain at rest pressure:
At-rest Pressure = Multiplier x active pressure

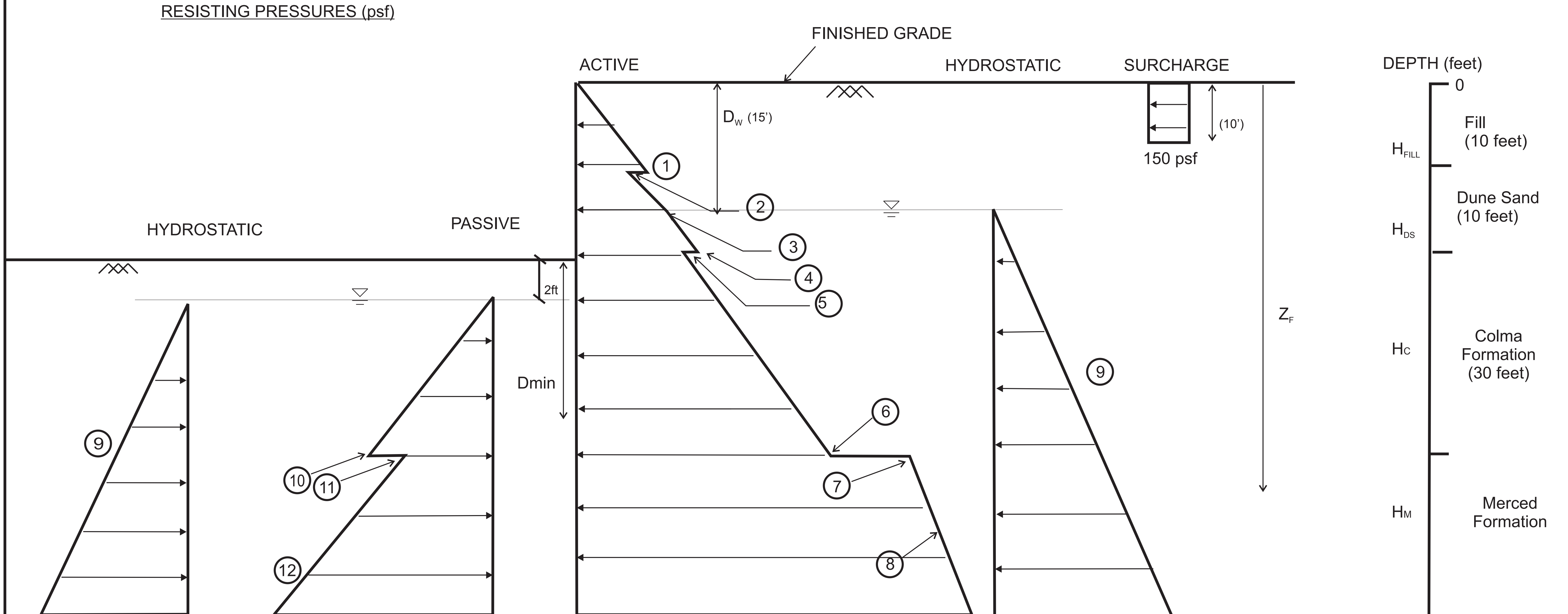
Material	Multiplier
Fill	1.53
Dune Sand	1.60
Colma Formation	1.62
Merced Formation	1.45

PRELIMINARY LATERAL EARTH PRESSURES
EQR REACH

SOUTH OCEAN BEACH COASTAL EROSION
WW INFRASTRUCTURE PROTECTION PROJECT
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003 | DATE: APRIL 2019 | PLATE 8B



NOTES:

1. Add surcharge pressure to active pressures. If surcharge is kept "H" feet behind wall, lateral surcharge will be negligible.

2. Minimum pile penetration (Dmin) for stability purposes only should be determined by balancing driving and resisting pressures.

3. Lateral surcharge pressure is a nominal value for traffic only. Actual values should be based on anticipated surcharges. Additional lateral surcharge pressures should be added to the nominal value shown above, as appropriate.

4. Dynamic Earth Pressure to be added to active pressure
Non-yielding retaining wall - uniform rectangular with base of 32H, where H is height of the wall
Rigid wall - straight triangular shape with base of 45H, where H is height of the wall.

LEGEND:

H = EXCAVATION DEPTH, IN FEET

Z_F = DEPTH BELOW EXISTING GROUND SURFACE, IN FEET

D_W= WATER DEPTH, IN FEET FROM GROUND SURFACE

Dmin = DEPTH TO BOTTOM OF RETAINING WALL, IN FEET

	3H:1V Condition	Intervening Condition
1	430 PSF	980 PSF
2	420 PSF	850 PSF
3	630 PSF	1350 PSF
4	730 PSF	1570 PSF
5	670 PSF	1170 PSF
6	1370 PSF	2140 PSF
7	2100 PSF	4070 PSF
8	2100 +25 (Z_f-50) PSF	4070 + 25(Z_f-50) PSF
9	62.4 PCF	
10	4900 PSF	
11	4200	
12	4200+ 210 (Dmin-30) PCF	

At-Rest Pressure

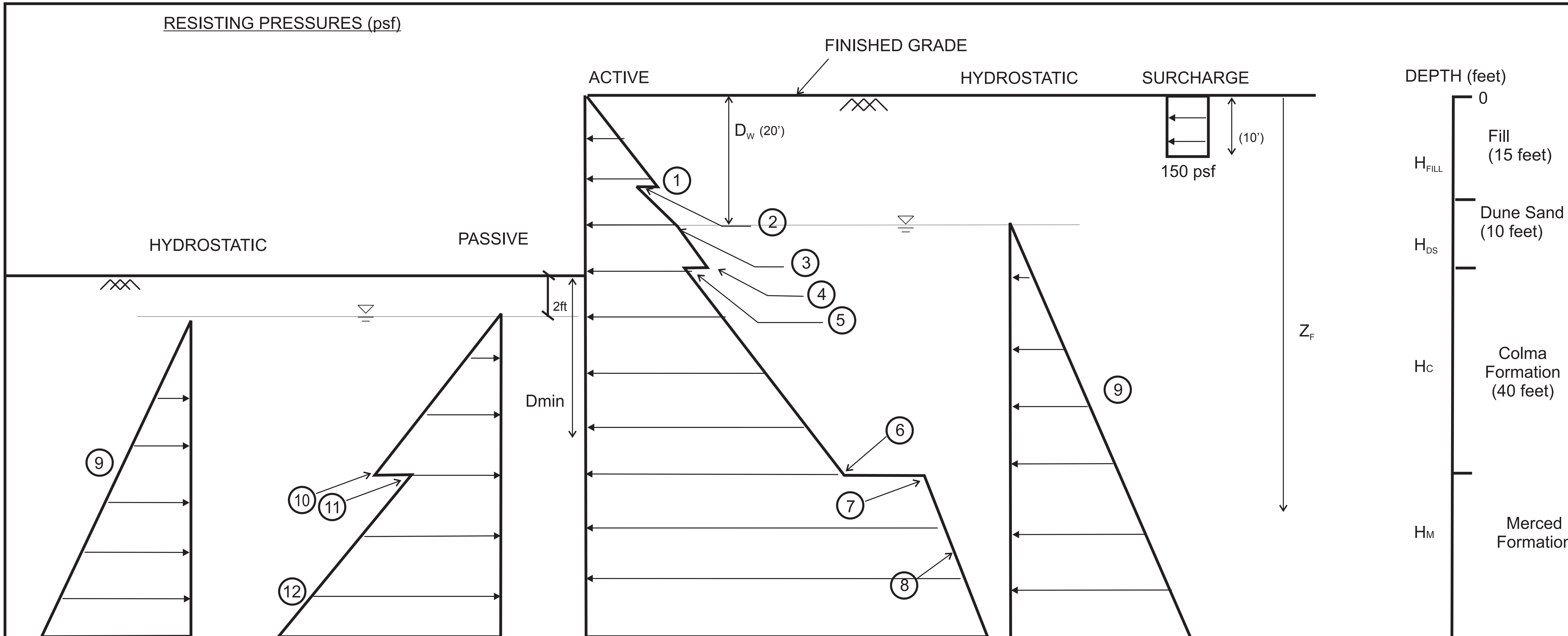
Use the following multiplier to obtain at rest pressure:
At-rest Pressure = Multiplier x active pressure

Material	Multiplier
Fill	1.53
Dune Sand	1.60
Colma Formation	1.62
Merced Formation	1.45

PRELIMINARY LATERAL EARTH PRESSURES
RUBBLE REACH
SOUTH OCEAN BEACH COASTAL EROSION
WW INFRASTRUCTURE PROTECTION PROJECT
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003 | DATE: APRIL 2019 | PLATE 8C



NOTES:

1. Add surcharge pressure to active pressures. If surcharge is kept "H" feet behind wall, lateral surcharge will be negligible.

2. Minimum pile penetration (Dmin) for stability purposes only should be determined by balancing driving and resisting pressures.

3. Lateral surcharge pressure is a nominal value for traffic only. Actual values should be based on anticipated surcharges. Additional lateral surcharge pressures should be added to the nominal value shown above, as appropriate.

4. Dynamic Earth Pressure to be added to active pressure
Non-yielding retaining wall - uniform rectangular with base of 32H, where H is height of the wall
Rigid wall - straight triangular shape with base of 45H, where H is height of the wall.

LEGEND:

H = EXCAVATION DEPTH, IN FEET

Z_F = DEPTH BELOW EXISTING GROUND SURFACE, IN FEET

D_W= WATER DEPTH, IN FEET FROM GROUND SURFACE

Dmin = DEPTH TO BOTTOM OF RETAINING WALL, IN FEET

	3H:1V Condition	Intervening Condition
1	648 PSF	1170 PSF
2	630 PSF	1050 PSF
3	840 PSF	1800 PSF
4	940 PSF	2020 PSF
5	860 PSF	1350 PSF
6	1760 PSF	2750 PSF
7	2700 PSF	5230 PSF
8	2700 +25 (Z_f-65) PSF	5230 + 25(Z_f-65) PSF
9	62.4 PCF	
10	6650 PSF	
11	5700 PSF	
12	5700+ 210 (Dmin-40) PCF	

At-Rest Pressure

Use the following multiplier to obtain at rest pressure:
At-rest Pressure = Multiplier x active pressure

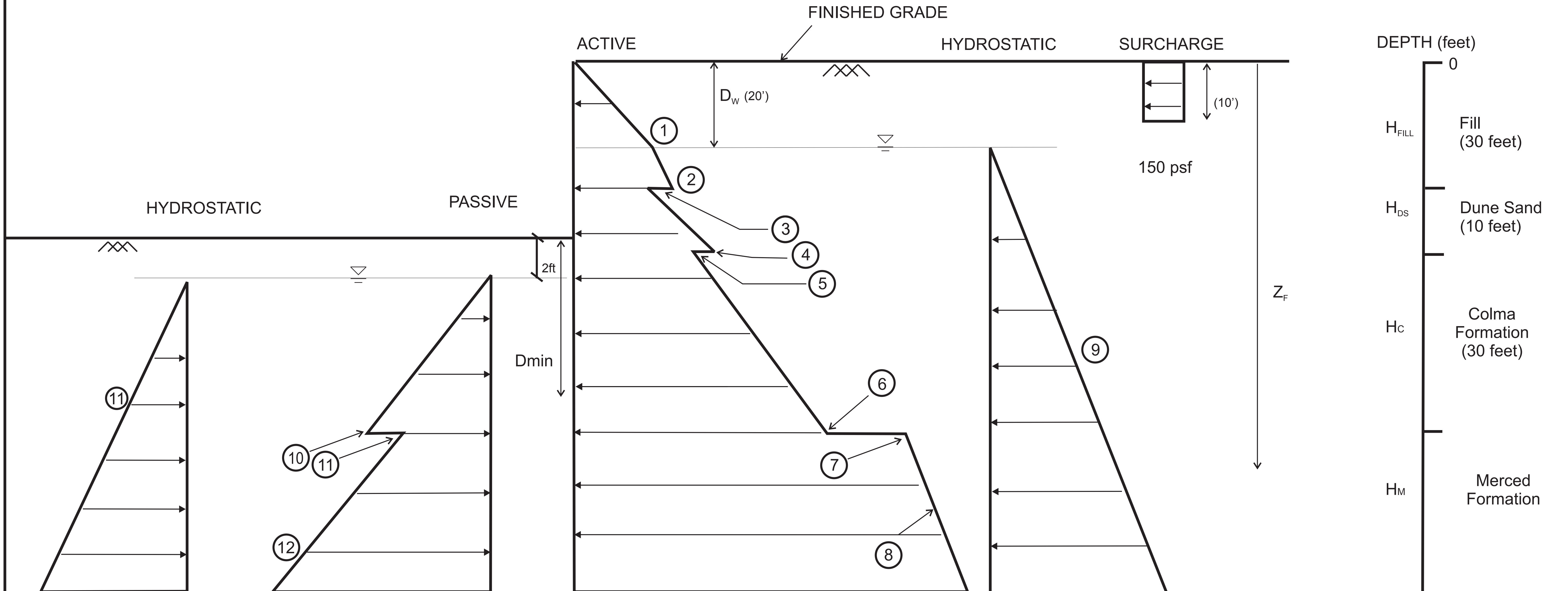
Material	Multiplier
Fill	1.53
Dune Sand	1.60
Colma Formation	1.62
Merced Formation	1.45

PRELIMINARY LATERAL EARTH PRESSURES
BLUFF REACH
SOUTH OCEAN BEACH COASTAL EROSION
WW INFRASTRUCTURE PROTECTION PROJECT
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003 | DATE: APRIL 2019 | PLATE 8D

RESISTING PRESSURES (psf)



NOTES:

1. Add surcharge pressure to active pressures. If surcharge is kept "H" feet behind wall, lateral surcharge will be negligible.

2. Minimum shoring penetration (Dmin) for stability purposes only should be determined by balancing driving and resisting pressures.

3. Lateral surcharge pressure is a nominal value for traffic only. Actual values should be based on anticipated surcharges. Additional lateral surcharge pressures should be added to the nominal value shown above, as appropriate.

4. Dynamic Earth Pressure to be added to active pressure
Non-yielding retaining wall - uniform rectangular with base of 32H, where H is height of the wall
Rigid wall - straight triangular shape with base of 45H, where H is height of the wall.

LEGEND:

H = EXCAVATION DEPTH, IN FEET

Z_F = DEPTH BELOW EXISTING GROUND SURFACE, IN FEET

D_W= WATER DEPTH, IN FEET FROM GROUND SURFACE

Dmin = DEPTH TO BOTTOM OF RETAINING WALL, IN FEET

	3H:1V Condition	Intervening Condition
1	865 PSF	1590 PSF
2	1071 PSF	1940 PSF
3	1040 PSF	2230 PSF
4	1240 PSF	2670 PSF
5	1140 PSF	1780 PSF
6	2140 PSF	3340 PSF
7	3270 PSF	6350 PSF
8	3270 +25 (Z_F-70) PSF	6350 + 25(Z_F-70) PSF
9	62.4 PCF	
10	4900 PSF	
11	4200 PSF	
12	4200+ 210 (Dmin-30) PCF	

At-Rest Pressure

Use the following multiplier to obtain at rest pressure:
At-rest Pressure = Multiplier x active pressure

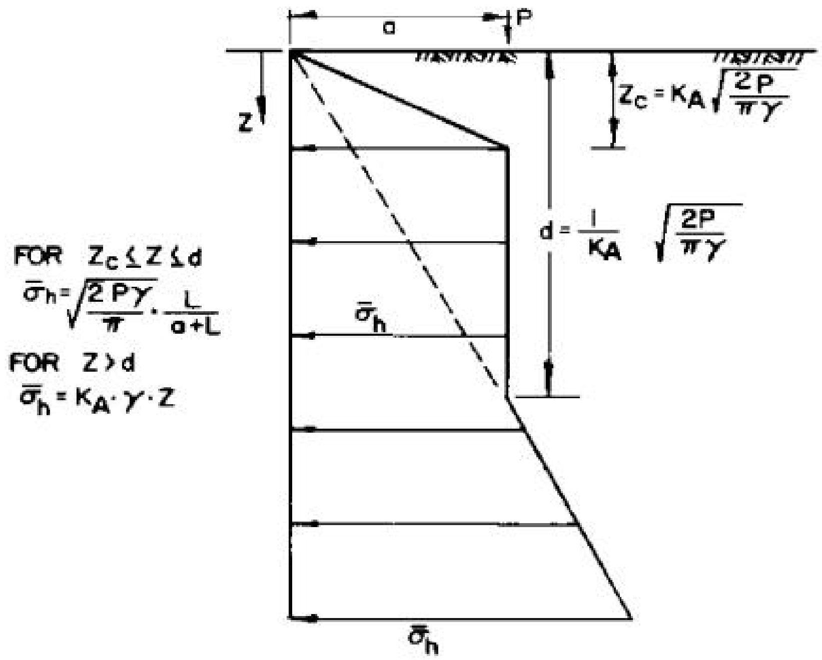
Material	Multiplier
Fill	1.53
Dune Sand	1.60
Colma Formation	1.62
Merced Formation	1.45

PRELIMINARY LATERAL EARTH PRESSURES
SOUTH REACH

SOUTH OCEAN BEACH COASTAL EROSION
WW INFRASTRUCTURE PROTECTION PROJECT
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003 | DATE: APRIL 2019 | PLATE 8E



$$P\text{ (ROLLER LOAD)} = \frac{\text{DEAD WT. OF ROLLER} + \text{CENTRIFUGAL FORCE}}{\text{WIDTH OF ROLLER}}$$

a : DISTANCE OF ROLLER FROM WALL

L: LENGTH OF ROLLER

γ: UNIT WEIGHT OF SOIL = 120 PCF

K_A = 0.85

Source: NAVFAC 7.2-77, Figure 13

HORIZONTAL PRESSURE ON WALLS FROM COMPACTION EFFORT

SOUTH OCEAN BEACH COASTAL EROSION
WW INFRASTRUCTURE PROTECTION PROJECT
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: APR. 2019	PLATE 9

Appendix C:
CER CEQA Checklist

SEWER SYSTEM IMPROVEMENT PROGRAM | Grey. Green. Clean.

Conceptual Engineering Report Checklist for Environmental Review

September 30, 2019

Prepared for:
San Francisco Public Utilities Commission

Section 1: Project Objectives

1. Describe the purpose and need for the project. What will this particular project accomplish?

   Currently, the existing wastewater infrastructure within the South Ocean Beach project area is threatened by chronic coastal erosion of the beach and bluffs, caused by wave action and episodic bluff failures. Critical infrastructure, such as the Lake Merced Transport and Storage Tunnel (LMT), has the most immediate need for protection, as it is located immediately behind the bluff, and is in jeopardy of structural instability and eventual structural failure without some form of engineered protection. Failure of the LMT or parts thereof would cripple the functionality of the Oceanside Wastewater Infrastructure. Additionally, other infrastructure related to public access and recreation such as the parking lot at Sloat and the Great Highway and the bathroom as well as the Great Highway are at risk of eventual failure from chronic erosion.

   Over the years, federal, state, and local agencies have adopted erosion mitigation measures, aimed at protecting the existing shoreline and beach. These efforts have included depositing sand along the bluffs and/or offshore areas and the construction of engineered rock revetment (under emergency permit order).

   Efforts in recent years have focused on the development of the Ocean Beach Master Plan (OBMP), which outlines coastal protection strategies along Ocean Beach through mid-century. The OBMP recommends management and protection measures for the existing essential wastewater infrastructure at Ocean Beach (including the LMT) in conjunction with increasing local access to the beach, improving aesthetics, and improving the beach's ecological functions. This project follows the OBMP guidance and focuses on a solution in the form of managed retreat of the Ocean Beach shoreline in response to chronic erosion and future sea-level rise.

   In 2018, the SFPUC produced an Alternatives Analysis Report, entitled: "Alternative Analysis Report for Coastal Adaptation Strategies for South Ocean Beach Wastewater System." The Alternative Analysis Report analyzed ten (10) options to address the threat of chronic erosion to the LMT and associated Oceanside facilities. The goal of the Alternatives Analysis phase of planning and subsequent report was to analyze engineered solutions that would maintain the operational capacity of the Oceanside facilities, incorporate the guiding principles of the OBMP and comply with regulatory requirements. The following project need was established in the report:

   Project Need:

   Protection of South Ocean Beach to address chronic coastal erosion and seal level rise impacts that:

   (1) Threaten Wastewater Infrastructure
   (2) Degrade Access and Recreation
   (3) Degrade Ecological Condition

2. List and describe specific project objectives (not Program objectives).

The overarching purpose of the project is to implement a long-term coastal management strategy for South Ocean Beach that addresses shoreline erosion and climate-related sea level rise. The specific project objectives are to:

- Preserve and enhance coastal public access, recreation, habitat, and scenic quality at South Ocean Beach
- Maintain current operational capacity of wastewater infrastructure to meet continued compliance with regulatory permits

- Protect the Lake Merced Tunnel, Westside Transport Box, and Westside Pump Station and associated facilities from damage due to shoreline erosion and storm and wave hazards
- Increase resilience to sea level rise
- Maintain emergency vehicle access
- Maintain dedicated service vehicle access to the Oceanside Treatment Plant, Westside Pump Station, and associated facilities
- Maintain visitor access to the San Francisco Zoo

Section 2: Site Plan

Provide a site plan on a topographic map. Everything should be labeled as either new or existing. Information on the site plan should include the following, including square footage, length, diameter, etc.:

1. Structural footprints (general areas): existing and created by the project.

   See Figure 8-1: Overall Concept Plan. The Project involves construction of approximately 3,200 feet of a secant pile wall system that would utilize soil anchors (tiebacks), to protect the LMT. The wall would be located on the seaward side of the existing tunnel (facing the ocean) and would range from 27 to 47 feet horizontally from the centerline of the existing tunnel to the inside face of the wall. The wall would consist of 3 foot-diameter unreinforced (primary) piles and reinforced (secondary) soldier piles. The depth of the primary piles would be approx. 60 feet below existing grade and the depth of the secondary piles would be approx. 100 feet below existing grade. Piles would be tied together with a continuous pile cap/grade beam system approximately 5 feet wide by 4 feet deep. The top of the pile cap would be approximately 6 feet above the crown of the LMT, which is approximately 20-30 feet below the existing grade. The tiebacks would be installed at a spacing of 10 to 15 feet along the pile cap. See Figures 6-9 thru 6-13 showing sections of the wall for each of the five reaches. Soil above the LMT and around the wall will be regraded and sloped in each reach as shown on the Figures.

2. Roadways and parking areas: existing and created by the project (both permanent and temporary).

   The existing parking lot located on the Great Highway will be demolished and a parking lot will be constructed under a separate SF Rec and Park Project south to the Great Highway and Skyline Boulevard (See Figure 8-1: Overall Concept Plan and Figure 8-4: Parking Lot Concept Layout). This parking lot will be constructed prior to the Ocean Beach Long Term Project. During construction this parking lot will be used for construction staging and as part of the scope of work this parking lot will be to modified to maximize parking spaces (from roughly 30 spaces to 55 spaces) and to tie in to the trail and access road. The modified parking lot be reopened to the public after the completion of the project.

   The entire Great Highway from Sloat Boulevard to Skyline Boulevard will be shutdown to the public during construction. Additionally, the beach in the extents of the project will be closed during the entire duration of construction. The existing roadway will be used for construction vehicle access as well as SFPUC access to OSP and WSP facilities. After construction, the existing south bound lanes will be the stabilized area above the new secant pile wall. Over the stabilized soil will be sand, landscaping and potential sand fencing on the surface. Three access points to the beach will be provided. The area of the road that is the existing north bound lanes will be converted to a multi-use recreational area with a separated trail and access road to be used by SFPUC vehicles. See the CER for more details.

3. Utility lines, including construction utilities such as electrical or dewatering lines. *Water crossings should be clearly marked. Estimated pole locations should be marked.*

   See Section 6.5-Existing Utilities of the CER. See Sections 6-14 thru 6-17 for utility plans showing sewer/stormwater, potable water, natural gas, electrical traffic signal, and street light utilities.

4. Standby generators, supervisory control and data acquisition (SCADA) equipment.

   A standby generator will be used for construction, however no permanent standby generator or SCADA equipment is incorporated into the permanent project (post construction).

5. Fencing (permanent and construction).

   Fencing and/or k-rail will be used to close the Great Highway from Sloat to Skyline Boulevard to the public. Fencing will also be used on the beach, during construction activities involving regrading and re-contouring of the bluff, which would be unsafe to construct while the public is on the beach. The fencing on the beach, and subsequent closure could last for approximately 6 months every year of construction duration.

6. Spoils areas.

   The majority of suitable soil will be reused for backfill of the wall and regrading purposes. At the CER level, assume 60% of excavated soil will be reused and 40% will be disposed of (this is approximately 474,000 cubic yards that would be off-hauled). Soil excavation and disposal, and the amount of suitable soil available, will be determined from test pits that will be done in the design phase. All rock and asphalt revetments will be removed from the project site and recycled, to the extent possible.

7. All grading areas, such as cutting into a slope.

   See Figures 6-9 thru 6-13, showing the regrading for each of the five reaches. Total grading area is approximately 800,000 ft^2

8. Laydown/staging areas.

   Proposed staging areas include:

   - Closed lanes of the Great Highway adjacent to project work
   - Existing Parking lot at Sloat (NPS property)
   - Zoo staging area
   - Beach area (only for removal of revetments and large boulders, and for sand placement)
   - Within OSP
   - Within WSP
   - Within Zoo Pump station
   - Within new Rec/Park parking lot

9. Absolute limits on construction area (provide map, square feet/acreage of the project site). *Nothing can occur outside of this area—no parking cars for the workmen, no ground disturbance, nothing. Give yourself enough room to work. However, don't add areas you know you will not need, as it makes the environmental review much more difficult. For example, if you show that an area of trees is within the construction area, BEM will assume those trees will be taken down*.

   See Figure 8-1: Overall Concept Plan

10. Estimated cut/fill information (cubic yards and acreages preferred, but LxWxD is OK). This is necessary for various topical analyses, such as truck haul estimates in the traffic section, land disturbance, etc.

    Approximately 1,185,000 yd^3 of soil will be excavated and approximately 40% of the excavated soil (474,000 yd^3) will be disposed.

11. Maximum depth of excavation.

    Maximum depth of excavation is approximately 35 to 40 feet

12. General information about elevation, and planned changes in topography. This includes spoils areas—provide a cross-section of the fill, or at least some type of quantified description.

    See Figures 6-9 thru 6-13 for sections showing regrading.

13. Specific information about the types of construction equipment to be used. *This is to determine noise and air quality impacts*.

    See constructability section in CER.

14. Information on all structures affected by the project, including age of existing buildings if known. *This is necessary for the historic analysis, and needs to be coordinated with the environmental team member early in the process to determine if further studies are needed. This is especially important if demolition or alteration of structures is planned*.

The LMT was constructed in 1990 under the 1220W-Lake Merced Transport Project. Other facilities in the vicinity of the LMT are the following:

– Westside Pump Station

– Oceanside Treatment Plant

Other structures in the proposed construction area include:

– There is an existing bathroom in the parking lot near Sloat Boulevard and the Great Highway. This is on NPS property.

– There are two abandoned pedestrian tunnels that cross above the existing LMT Tunnel. Both abandoned tunnels are 10 ft tall x 8 ft wide. One tunnel is located approximately 250 ft south of Sloat Blvd, and the other is located approximately 1,300 ft south of Sloat Blvd. The top of these abandoned tunnels is approximately 5 ft below existing grade.

– At approximately 600 feet from the south end of the project, the South West Ocean Outfall (SWOO) crosses under the Great Highway and the LMT. The SWOO is a 12 ft square reinforced-concrete box. The box connects to a 12-ft diameter reinforced-concrete pipe that discharge the treated wastewater into the ocean.

15. Information on off-site spoils areas (and a list of potential landfills if possible). *CEQA addresses environmental impacts on off-site spoils areas*.

– Not known at this time, to be determined from hazardous soil analysis.

16. Official address of site (or mailing address if no "official" address), *if known. Many SFPUC facilities do not have addresses*.

N/A

17. Description of future and operations/maintenance activities.

The LMT operates 24 hours per day, 7 days per week. The structure does not require regular maintenance. Inspections and minor repairs are performed on an as-need basis. The Great Highway above the LMT is currently maintained by PW and the existing bathroom on the Great Highway at Sloat Boulevard is maintained by GGNRA. The proposed project will require staff for sand management as wells as staff for trail maintenance, parking lot and bathroom maintenance, beach access maintenance ongoing landscaping maintenance. Based on the preliminary proposed locations of the items, the trail and parking lot will require RPD maintenance. The beach access points and landscaping will require GGNRA maintenance. The proposed locations of the bathrooms at either Wawona or Sloat Boulevard would require RPD maintenance. Please see section 11 of the CER for Operations and Maintenance Requirements. Sand management would be an ongoing maintenance effort led by the SFPUC.

No increase of existing operations staff levels is anticipated.

18. Information on parking/loading spaces (numbers of each, including handicapped spaces).

See Section 8.2 of the CER for Parking details. The existing parking lot located at Sloat and the Great Highway will be potentially used for staging and eventually demolished for the project. A new parking lot located at the Great Highway and Skyline Boulevard is being constructed under an interim SF Rec and Park project. The parking lot will have approximately 30 spaces. It will be used closed to the public during the Ocean Beach Long Term project and used for construction staging. The project will modify the parking lot constructed by the Rec and Park project to maximize parking spots to approximately 55 parking stalls and to tie in the trail and access road.

19. Preliminary project schedule.

The planning phase of the project will end in September 2019, when the final CER is issued. Following the final CER, the design phase will begin and will end with 100% design issued in January of 2021. Based on the proposed EIR certification date and permit dates, followed by the bid and award process for contract award, the project will start construction in January 2023. The duration of construction is approximately 46 months.

20. Construction durations by type of activity. *While optional during preparation of this checklist, it will be eventually required for the environmental review*.

    See Section 9.3-Wall Construction of the CER.

21. Blowoff locations and information on where discharges will drain. Also, shutdown information when it concerns discharges. *This should be shown on a map*.

    Not applicable. The project has no blowoffs. Any groundwater discharge or runoff will be redirected back to the combined storm/sewer system via existing manholes on the Great Highway.

22. Landscaping plans. *While optional during preparation of this checklist, it will be eventually required for the environmental review. (This is not a requirement for a plan but rather a general description of type of land cover.)*

    Vegetation will be planted on the recontoured bluff and along the multi-use trail. Please see Figure 8-2 and Figure 8-3 of the CER for schematic. The plantings will conform to the following criteria:

- Native
- Climate-Appropriate
- Locally Adaptive
- Non-Invasive
- Low Water required.

Detailed landscape drawings will be developed in the design phase.

Section 3: Land Use

1. Aerials of the project area (including staging areas, spoils areas, etc.).

   See Figure 8-1 and Figures 6-5 through 6-8 of the CER.

2. Information on encroachment issues—will anything (structures, trees) need to be removed from the project site?

   The SFPUC Real Estate group is currently verifying ownership boundaries for the different elements of the project. There will be encroachment on NPS , Caltrans and SF Rec and Park land and MOU's and construction/special use permits will have to be applied for and issued. Additionally, trees will need to be removed in the existing median of the Great Highway. See Figure 12-1 of the CER for preliminary project boundaries.

3. Parcel maps of the area, showing adjacent properties.

   The SFPUC Real Estate group is currently verifying ownership boundaries for the different elements of the project. See Figure 12-1

4. Copy of United States Geological Survey (USGS) 7.5-minute quad maps for the project area.

   See attached preliminary USGS Map.

5. A list of all property owners within 300 feet of the property line of the site if a General Rule Exclusion (GRE), Negative Declaration (NegDec), or EIR is expected. Two sets of address labels are required.

   Per the Environmental Project Manager's (EPM's) direction, this section will be completed by the CEQA consultant.

6. San Francisco Master Plan designation and zoning of the project parcels. Sites outside city limits require local designation/zoning information.

   The project lies within the Western Shoreline Planning Area. It is located within the coastal zone and is classified as zoning district P (public).

7. Present and past use of the site, especially permitted uses, if available.

   Prior to the Oceanside Facilities built in the late 1980's, the site was an open area. Ocean Beach has always been a public area with access to the beach. In 1925 Fleishhacker Pool was built in close proximity to the project site. The pool was eventually filled in and now serves as a parking lot for the zoo. The pool house was demolished and the fragments of the entrance stand today at the zoo parking lot.

8. Information on growth-inducing issues. *This should be coordinated with the environmental manager*.

   The Project would not be growth inducing.

9. Information on any historic preservation requirements.

   The EIR consultant will prepare a Historic Resources Evaluation to evaluate any historic resources in the project area. There was a SHPO and ACHP MOA for the Fleishacker Pool formerly on the zoo parking lot.

10. Information on watershed requirements, including applicable policies of the Watershed Management Plans, if applicable.

    The project will comply with urban watershed management requirements, if applicable. All stormwater will be directed to the combined sewer. Future runoff from the roadway and parking lot will be graded to drain to the existing combined sewer infrastructure at the site.

See Section 13-Environmental Review of the CER for all permits/approvals required.

Section 4: Water, Operations, and Maintenance

1. Dewatering information (estimated location of Baker tanks, location of discharge, estimated quantity if known, etc.)

   Groundwater elevation ranges from 15 feet below grade at the north end of the project to 30 feet below grade at the south end of the project.

   Groundwater will be discharged into the combined sewer system.

2. Information on groundwater levels, if known.

   Groundwater elevation ranges from 15 feet below grade at the north end of the project to 30 feet below grade at the south end of the project.

3. Flood zone maps, if available.

   To be provided.

4. Information on ordinary high water mark for waterways, if applicable.

   Mean high water line is shown in the drawings in the CER.

5. Saltwater intrusion information, if necessary. *Often occurs as a result of dewatering drawdown*.

   N/A

6. Information on operation water quality/quantity issues (such as any planned discharges, diversion rates, planned releases, etc.).

   It is anticipated that all surface water or groundwater will be pumped into the combined sewer system.

Section 5: Hazardous Waste

1. Underground storage tank (UST) information. Coordination with the environmental team member is necessary if USTs exist. A Phase I or II site assessment might be required.

   Not known at this time.

2. Information on chemicals and fuels storage during construction and operation.

   Chemicals used on-site for construction will include diesel (emergency diesel generator) and bentonite (used to keep pile holes open).

   During construction, the contractor will be required to meet county and state fuel storage requirements. No chemicals will be stored on site.

3. Site status on the State's "Cortese List" (list of sites with known hazardous contamination).

   EPM to coordinate

4. Existing Phase I, Phase II, or geotechnical studies. Required if you already have them. However, it is not required for you to perform these studies.

   No Phase I report has been done for the project. The Draft Geotechnical Data Report (GDR) and Draft Geotechnical Interpretive Report (GIR), both prepared by AGS (July 2019), are available. Six environmental borings were drilled as part of the geotechnical field exploration program. The following tests were performed on the samples:

a. Total Petroleum Hydrocarbons – gasoline diesel and motor oil by EPA Method 8015B;

b. California Title 22 Metals by EPA Methods 6010B and 7471A;

c. Hexavalent Chromium by EPA Method 7196A;

d. Volatile Organic Compounds (VOCs) by EPA Method 8260B;

e. Semi-volatile Organic Compounds (SVOCs) by EPA Method 8270C; and

f. Organochlorine Pesticides (OCPs) by EPA Method 8081A.

Results of the testing can be found in the GDR.

Section 6: Noise

1. Information on pile driving, if needed. Indicate the locations and estimated duration of pile/sheet driving.

   The Project involves construction of approximately 3,200 feet of a secant pile wall system that would utilize soil anchors (tiebacks), to protect the LMT. The wall would be located on the seaward side of the existing tunnel (facing the ocean) and would range from 27 to 47 feet horizontally from the centerline of the existing tunnel to the inside face of the wall. The wall would consist of 3 foot-diameter unreinforced (primary) piles and reinforced (secondary) soldier piles. The depth of the primary piles would be approx. 60 feet below existing grade and the depth of the secondary piles would be approx. 100 feet below existing grade. Piles would be tied together with a continuous pile cap/grade beam system approximately 5 feet wide by 4 feet deep. The top of the pile cap would be approximately 6 feet above the crown of the LMT, which is approximately 20-30 feet below the existing grade. The tiebacks would be installed at a spacing of 10 to 15 feet along the pile cap. See Figures 6-9 thru 6-13 showing sections of the wall for each of the five reaches. Secant pile installation and grade beam/pile cap casting will take approximately 12 months of continuous construction.
2. Spec. sheets on any noise-generating operational equipment (such as pumps, compressors, or generators—we also need to know the types of actuators being used on valves). This is used with zoning information to determine if operational noise is within an acceptable range. If not, design changes may be required. This should be coordinated with the environmental team member. These spec. sheets do not need to be of the exact equipment that will be used (as that is probably not known). Spec. sheets of representative equipment can be used.

   See constructability section of the CER for details

Section 7: Aesthetics

1. Spec. sheets on proposed lighting elements. These spec. sheets do not need to be of the exact equipment that will be used (as that is probably not known). Spec. sheets of representative equipment can be used. While optional during preparation of this checklist, it will be eventually required for the environmental review.

   There are five existing street lights in the project area that will need to be removed and replaced in kind.

2. Information on estimated size/height and detail of existing or proposed structures. This includes vaults and proposed access to vaults.

   The proposed low profile wall will be buried. Under extreme storm events detailed in the CER document, the wall could potentially be exposed. The concrete face of the grade beam for the wall would be visible potentially.

3. Information on site lighting.

   Lighting will be added to the multi-use trail and new restroom.

4. Planned color of structures, if known.

   The low profile wall will be concrete, without proposed painting/color.

Section 8: Geology and Soils

1. Information on faults. This includes if the project is located on an Alquist-Priolo Earthquake Fault zone, if known (see http://www.consrv.ca.gov/CGS/rghm/ap/ for more information on these fault zones).

   According to the project Geotechnical Interpretive Report, the project is not located within the Alquist-Priolo earthquake fault zone. The project area is located in a seismically active region however. The San Andreas Fault is approximately 1.6 miles southwest of the site and is the major fault system in the region. Further from the project alignment are the San Gregorio Fault, which is 4.7 miles from the site and the Hayward Fault, which is 17 miles from the site.

2. Information on expansive soil (as per Building code), if known.

   Soil units at the proposed site typically consist of fill, dune sand, colma formation and merced formation. Expansive soils, such as clays, are not anticipated.

3. Geotechnical studies, if available (see hazardous waste above). *Required by the Planning Dept.*

   The Draft Geotechnical Data Report (GDR) and Draft Geotechnical Interpretive Report (GIR), both prepared by AGS (July 2019), are available.

4. Information on geologic work near/adjacent to structures (estimates of vibration effects).

   A secant pile wall with a pile cap/grade beam system and tie backs is proposed. The amount of vibration from the pile driving has not been estimated.

Section 9: Traffic

1. Traffic information, such as proposed haul routes.

   The Great Highway from Sloat to Skyline Boulevard will be permanently closed to the public during the construction duration and permanently thereafter. Construction vehicles will use the two existing north bound existing lanes to access the site and it is anticipated that they will access the site from the south at the Skyline Intersection. Intersection modification at Sloat and the Great Highway and Skyline and the Great Highway is described in the CER. Additionally access for emergency vehicles will be maintained during construction on the two northbound lanes as well as the sand ladder at Sloat, which allows access to the beach. After construction, emergency vehicles will be able to use the access road as well as the sand ladder at Sloat.

2. Estimated staffing levels of existing or proposed facility. *Used to determine parking/traffic issues*.

   No increase in staffing levels is anticipated for any portion of the project.

Section 10: Biological Resources

If any trees greater than 4 inches in trunk diameter or taller than 20 feet will be removed, a plot plan is required showing the location, size, and common or botanic name(s) of each.

EIR consultant to conduct bio surveys and tree surveys as needed.

Section 11: Air Quality

Information on any generators (including map and spec. sheets) for air requirements. Contact the EPM for the latest requirements and refer to Sample CEQA Air Quality Information (eDOCS DM #762889).

Not available at this time, to be provided in detailed design.

Conceptual Engineering Report

Ocean Beach Long-Term Improvements Project

MN+AGS JV

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