# COASTAL PROTECTION MEASURES & MANAGEMENT STRATEGY FOR SOUTH OCEAN BEACH

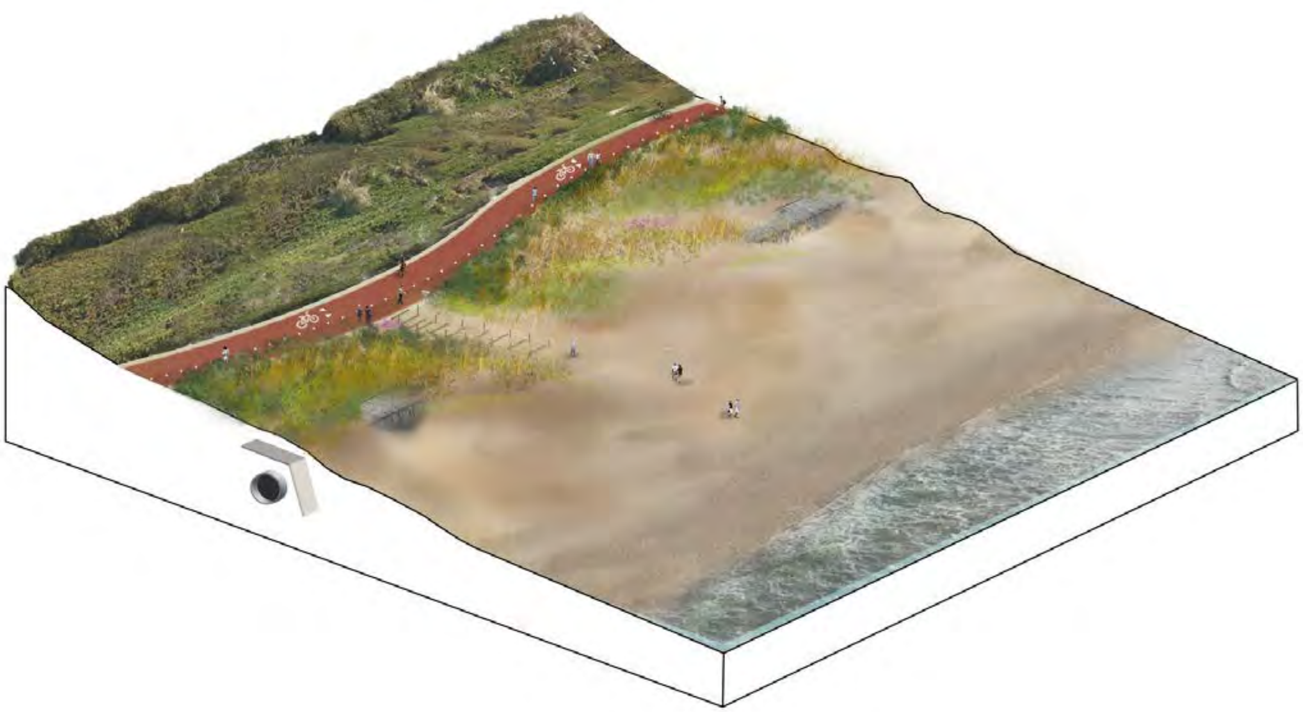
## Ocean Beach Master Plan: Coastal Management Framework

Prepared for
San Francisco Public Utilities Commission

April 24, 2015

Under Contract to SPUR


Prepared by
SPUR
ESA PWA
Moffatt & Nichol
McMillen Jacobs Associates
AGS, Inc.

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# TABLE OF CONTENTS

# Coastal Protection Measures & Management Strategy for South Ocean Beach

*Ocean Beach Master Plan: Coastal Management Framework*

**Page**

## List of Tables



## List of Figures

## List of Appendices

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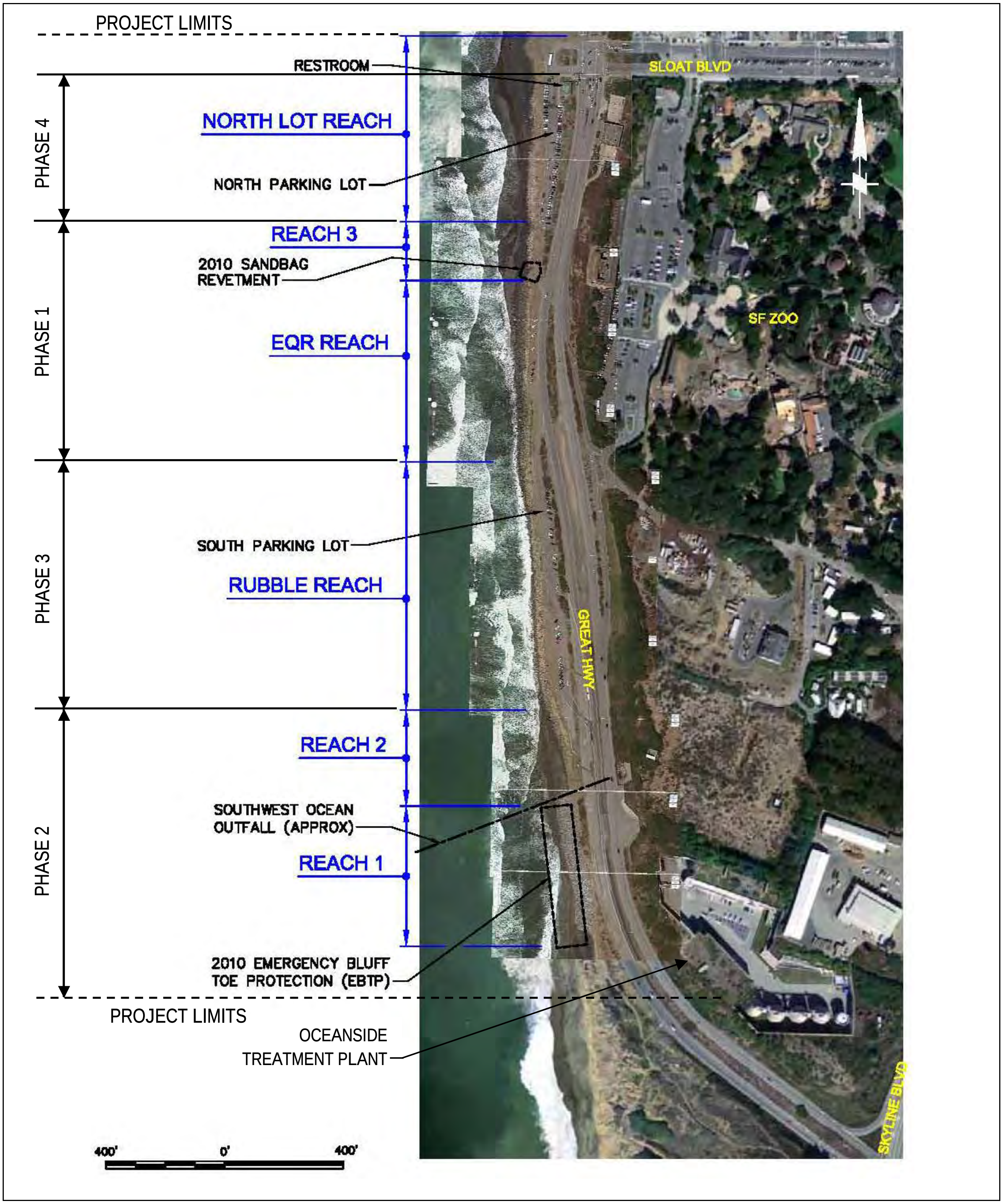
# 1 SUMMARY OF FINDINGS

The Ocean Beach Master Plan (OBMP, SPUR 2012) study area encompasses the beach and adjacent lands from the high-water mark to the property line at the eastern edge of the Lower Great Highway, and from the beach's northern extent at the foot of the Cliff House to the Fort Funston bluffs (it excludes private property). This project focuses on the OBMP recommendations for the southern reach - South Ocean Beach (SOB) [Figure i, Project Site] which is south of Sloat Boulevard where erosion hazards are chronic and jeopardize critical City and County of San Francisco (CCSF) infrastructure.

This area is in need of coastal protection due to the narrowing of SOB as a result of coastal dynamics and sediment transport. Over the years, CCSF responded to intense erosion jeopardizing city infrastructure with the construction of engineered revetments (boulder embankments) in order to protect the existing shoreline. However, implementation of these projects has affected the beach's natural conditions and access for recreational users. New information related to climate change, sea level rise, the impacts of several significant El Nino events, etc. have modified CCSF's approach to protect SOB and they are now focused on managed retreat. This updated thinking emphasizes the use of low impact technologies inland of the current shoreline that provide multiple benefits and opportunities for integrated management (e.g. protect critical infrastructure and provide for the protection and enhancement of natural resources).

The goal of the project is to further develop long-term coastal protection measures and a management strategy using a multi-objective approach that both protects critical wastewater infrastructure (i.e. Lake Merced Tunnel) and promotes environmental stewardship. The findings and subsequent recommendations presented in this report are based on the team's coastal vulnerability and engineering feasibility analyses of coastal protection and management measures. These concepts were developed with the help of a Technical Advisory Committee. The project team emphasizes options that are reversible, minimally impactful and compatible with the OBMP recommendations. This summary of findings is organized into four categories: 1) project overview, 2) vulnerability assessment and trigger definition, 3) the preferred project concept and 4) recommendations for further analysis.

The preferred project concept for coastal protection and shore enhancement is described in more detail in Section 6 of this report. Conceptual drawings of the restored shore are shown in Figures 4, 5, and 6. As noted in the fourth category, further analysis (e.g. geotechnical, geo-structural, seismic response, etc.) should be completed to inform a final decision on project buffers and triggers, and to inform the subsequent design stages.




SOURCE: Moffatt & Nichol (2012)



**Figure i**
Project Site and Definition of Reaches

## 1.1 Project Site (SOB) Overview

- Existing Conditions at SOB are characterized by:
  - Chronic, ongoing erosion of the beach and bluffs by wave action and episodic coastal storms
  - Variable degrees of exposure along the beach and bluffs to erosion; existing erosion protection includes engineered revetments and sandbags, sand nourishment (i.e. sand moved from the northern end of Ocean Beach to SOB in 2012 and 2014), and exposed rubble (i.e. materials used during the construction of the roadbed associated with the Great Highway, parking lot asphalt, etc.).
  - Vulnerability of the Lake Merced Tunnel (LMT) to coastal erosion in some locations, and increasing in extent and severity over time.
  - Degraded access conditions, including a narrow beach, hazardous engineered revetments, exposed rubble and debris, and eroded parking lots and storm drains.
  - Degraded ecological conditions, including a narrow beach, minimal vegetation, and a lack of continuity with adjacent dune and bluff systems.
  - Emergency engineered revetments (e.g. coastal armoring).
- The project concept for SOB is multi-objective and needs to address coastal hazards while improving shoreline conditions, adapt to sea level rise and be consistent with the OBMP. This multi-objective project includes the following components:
  - Protection of the SFPUC LMT;
  - Allow for coastal retreat and realignment, including eventual removal of the existing roadway and relocation of parking facilities;
  - Removal of existing armoring, rubble, debris and degraded development (e.g. parking lots, storm drains, and associated hardscape that have been damaged and impacted by erosion);
  - Placement of sand and native plants for erosion control and improved ecological function; and,
  - Construction of new open space with public access to and along the shore.
- The project implementation actions spread over several timeframes with some overlap:
  - **Long-term Planning (10-40 Year Timeframe starting 2020):** Continued implementation of OBMP coastal management recommendations; includes phased adaptive management approach; likely to include complex regulatory and implementation processes that will be enacted over the next 10 to 40 years. It is expected that diligent effort will result in completion of designs and approvals, funding and construction by then. Subsequently, it is expected that the implementing agencies will reassess the OBMP around 2030, with an understanding that sea-level rise and other factors will be better defined and revised in the plan at that stage. At that time, agencies will refine the OBMP for the coming decades (2050 to 2100+).
  - **Interim Interventions (3-10 Year Timeframe starting 2018):** This period is likely too early for full implementation of OBMP recommendations, but is critical for establishing the direction and compatibility with future, long-term implementation of OMBP recommendations; detailed designs, pilot studies, regulatory processes and

interim measures will proceed during this period directed toward full implementation. Ongoing management of the shore at SOB during the interim period will be informed by the results of the vulnerability and adaptation feasibility study, as well as the development of a proposed Interagency Coastal Management Agreement.

- **Immediate Needs (1-3 Year Timeframe starting 2015):** The need to minimize short-term risks and other ongoing erosion and/or damage issues is currently being addressed through yearly sand nourishment activities. Additional measures are being considered as part of the *Immediate-Term Coastal Erosion Management Plan* (Immediate Plan) currently under review.

## 1.2 Lake Merced Tunnel Vulnerability Assessment & Triggers

- The long-term trend in shore change at SOB is primarily erosion, dating back to at least the 1850s; Ocean Beach is the visible portion of a much larger coastal sand and sediment system. It is an intensely energetic environment, frequently battered by powerful waves and storm surges; the beach is subject to erosion in which more sand is removed than deposited by waves and currents and the shoreline recedes landward.

- The U.S. Army Corps of Engineers annually dredges a marine shipping channel in the sandbar near Golden Gate Bridge to allow ship access to the San Francisco Bay; the northern end of Ocean Beach has been getting wider since the 1970's because of a combination of sediment management practices and natural changes to the sandbars; conversely SOB is narrowing as erosive forces scour away sand and bluffs leaving less and less buffer between waves and critical SFPUC infrastructure.

- Engineered revetments and sand bags and sand nourishment have partially and temporarily mitigated damage risk to the LMT.

- Although the LMT is below beach level and inland of the existing bluff, the LMT is located too far seaward to be sustained without adaptive actions to protect it from damage.

- The LMT is part of the SFPUC's combined sewer system that protects coastal water quality and therefore damage would likely result in regulatory permitting violations and impacts to the environment.

- Relocation of the LMT is not practical within the vulnerability horizon. Hence, protecting the LMT in place with a structure is the preferred and recommended solution.

- Sand nourishment and other environmentally friendly measures described in the Immediate Plan for SOB are expected to be adequate to mitigate risks over the next few years, provided that extreme storms with approximately a 30-year return period or greater do not occur.

- The varying geometry, geology, armoring and LMT alignment result in a range of vulnerability along the SOB shore.

- Over the interim timeframe, the LMT is most vulnerable along Reach 3, EQR Reach, and Reach 2 (Figure i).

- The LMT is vulnerable along all SOB reaches within the next 50 to 100 years, starting in 2015.

- The LMT is most vulnerable to a reduction of lateral support. In addition, the structural integrity of LMT is at an unacceptable risk level when the bluff toe migrates to within 10 feet of the LMT.

- The LMT is least vulnerable to a reduction in vertical overburden. Based on geo-structural modeling of the LMT by McMillen Jacobs Associates (Appendix 4), the minimum required overburden to counteract potential buoyancy forces has been reduced from 15 feet (in previous studies) to six feet of soil, or an equivalent "hold-down".

- Minimum *Structural Buffers* (horizontal and vertical) are recommended below, beyond which risk levels become unacceptable. The minimum soil dimension around the tunnel to maintain structural stability has been revised based on geo-structural modeling of the LMT by McMillen Jacobs Associates (Appendix 4).

  - *Horizontal Structural Buffer*: 10 feet of soil between the tunnel and bluff toe to provide adequate lateral restraint. Previous studies conservatively stipulated 25 feet.

  - *Vertical Structural Buffer*: 6 feet of soil overburden to counteract potential flotation forces, or equivalent hold-down. Previous studies conservatively stipulated 15 feet.

- An additional *Safety Buffer* of 25 lateral feet is recommended to allow for rapid erosion that could occur during a large winter storm with a 15- to 20-year return period, with the objective of avoiding emergency action or actual damages (Table 1).

- A *Trigger Distance* of 35 lateral feet from the LMT (10-foot Structural Buffer plus 25-foot Safety Buffer) is therefore recommended as the minimum distance between the LMT and the bluff toe before initiating action to protect the LMT.

- Subsurface exploration (e.g. geotechnical) is recommended to better define stratigraphy and engineering properties affecting LMT vulnerability and protection design. The subsurface data should then be used to inform additional geotechnical, geo-structural, structural, and coastal vulnerability analysis of vulnerability and structural protection. Analysis of seismic response is also recommended to further assess the triggers.

**TABLE 1**
**DEFINITION OF BUFFER AND TRIGGER DISTANCES**

| Component | Distance | Note |
|---|---|---|
| Structural Buffer | 10 feet | Horizontal distance that will maintain structural integrity of the LMT |
| Safety Buffer | 25 feet | Allowance for erosion from large winter storm with 15- to 20-year return period |
| Trigger Distance | 35 feet | Trigger for action to protect the LMT using Immediate Plan measures and initial implementation of the long-term project |

## 1.3 Preferred Project Concept

- The LMT would be protected with a low-profile wall structure that is generally described in the OBMP. The feasibility study found that a vertical wall could be constructed from the existing bluff-top surface to protect the LMT along SOB (Figure ii, Plan View Structural Protection):
  - A low-profile vertical wall to provide lateral constraint of the LMT, placed at the minimum structural buffer distances described above, with a top elevation near that of the crown (top) of the tunnel (Figure iii, Typical Sections).
  - In some locations, a horizontal slab, or cap, would be added, providing additional protection, buoyancy resistance and vertical restraint (see Figure iii, Typical Sections).
  - In vulnerable locations, this cap may be constructed along with the vertical wall. In others, it can be added once erosion begins to expose the vertical wall.
  - The vertical wall can be constructed using different materials and approaches, including a pile wall and soil mix wall, which could both be installed from the existing ground surface with minimal excavation.
  - A reinforced concrete secant pile wall is a promising structural technique for the vertical element, especially where bluffs consist of imported fill.
  - A soil mix wall, formed by injection of cementitious grout into the native bluff material, may be suitable for southerly reaches, where bluffs consist of Colma formation sandstone. It would not require a horizontal cap. A soil mix wall is not as strong as reinforced concrete, but could be made stronger by having a greater thickness. When exposed, its appearance is similar to the native material it contains.
  - This structural protection system is a feasible solution to anticipated coastal hazards and is consistent with recommendations in the OBMP.
- The existing "Taraval Seawall"[1] provides a useful example for the proposed structure in its finished condition.
- Construction of the structure will be accompanied by surface restoration, including:
  - Grading of the area to improve site drainage and removal of excess imported materials (i.e. concrete rubble and rock);
  - Placement of sand to nourish the beach and construction of linear back-beach dunes; planting of native coastal dune and bluff plants;
  - Construction of access improvements, including trails and other public amenities.
- During the initial phases, pilot studies (e.g. soft measures such as a dynamic cobble revetment and sand nourishment) could be integrated into the interim interventions, especially to manage erosion processes during approval and mobilization of long-term measures and to test concepts.

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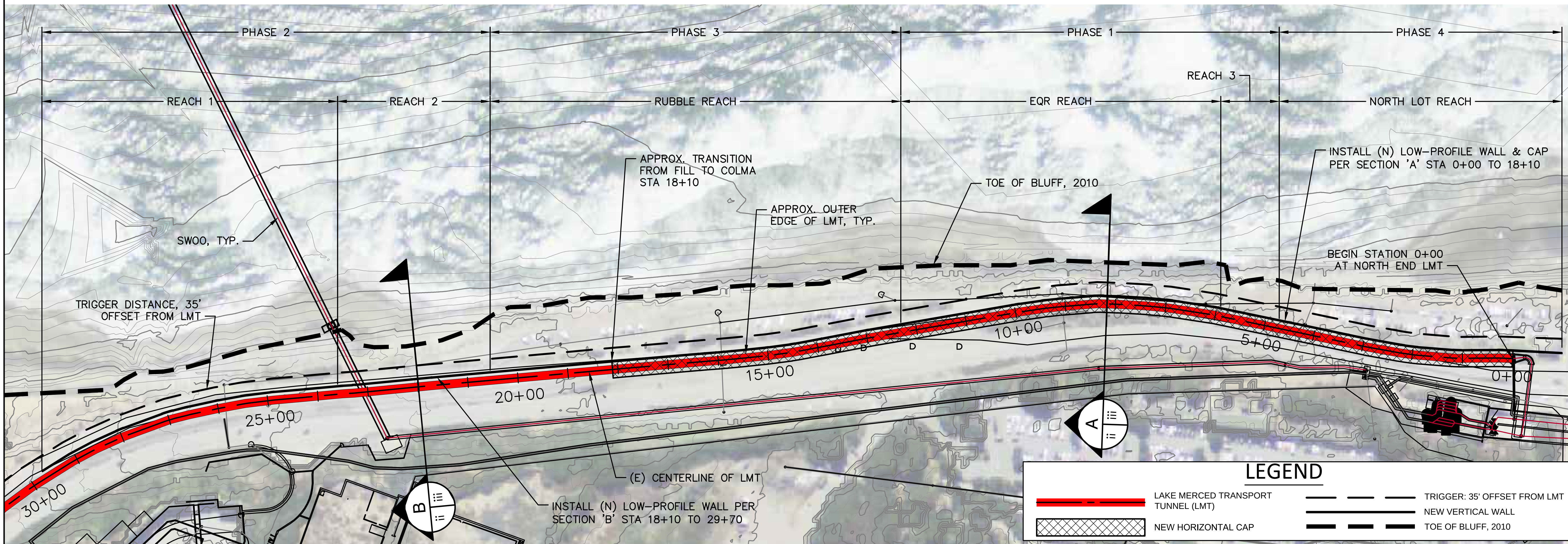
[1] Low profile seawall constructed at the end of Taraval Street in the 1940s. See Sections 6.5 and 6.6.




SUMMARY OF PHASES AND EXTENTS OF CONSTRUCTION

| PHASE | REACHES INCLUDED | APPROXIMATE LENGTH OF PHASE | STATIONING OF NEW VERTICAL WALL | STATIONING OF NEW HORIZONTAL CAP | CROWN ELEVATION OF OUTSIDE OF LMT (FEET NAVD) | REMAINING BLUFF WIDTH SEAWARD OF TRIGGER DISTANCE (NOTE 1) | ESTIMATED TIME UNTIL TRIGGERED (NOTE 2) |
|---|---|---|---|---|---|---|---|
| 1 | EQR & REACH 3 | 800 FEET | 4+70 TO 12+30 | 4+70 TO 12+30 | 9.0 - 10.1 | 24 FEET | 8 TO 24 YEARS |
| 2 | REACHES 1 & 2 | 800 FEET | 20+55 TO 29+70 | -- | 11.2 - 12.3 | 40 FEET | 13 TO 40 YEARS |
| 3 | RUBBLE REACH | 800 FEET | 12+30 TO 20+55 | 12+30 TO 18+10 | 10.1 - 11.2 | 81 FEET | 13 TO 40 YEARS (NOTE 3) |
| 4 | NORTH LOT REACH | 650 FEET | -1+00 TO 4+70 | 0+00 TO 4+70 | 8.5 - 9.0 | 44 FEET | 15 TO 50 YEARS (NOTE 4) |

NOTE 1: REMAINING BLUFF CALCULATED AS DISTANCE FROM TRIGGER TO THE BLUFF TOE, BASED ON USGS BEACH SURVEY FROM JANUARY 2010. VARYING WIDTHS; NARROWEST MEASUREMENT UTILIZED FOR CONSERVATIVE CALCULATION. THE BLUFF TOE THROUGHOUT THE PROJECT AREA FOR 2015 IS FARTHER SEAWARD DUE TO SAND PLACEMENT IN 2012 AND 2014-2015.

NOTE 2: ESTIMATE OF TIME FOR BLUFF TOE TO REACH TRIGGER FOR REQUIRED ACTION IS BASED ON A RANGE OF EROSION RATES FOR UNARMORED BLUFFS OF 1 TO 3 FEET PER YEAR. BECAUSE MOST OF THE BLUFF IS PARTIALLY OR FULLY ARMORED, THE ESTIMATE IS CONSERVATIVE. HOWEVER, UNARMORED BLUFFS OR SAND DUNES (VS. COLMA) COULD ERODE APPROXIMATELY 25 FEET IN AN EXTREME EVENT.

NOTE 3: FOR PHASE 3, TIME WAS REDUCED TO ACCOUNT FOR RUBBLE REMOVAL, WITH SHORELINE ADJUSTMENT OF 40 FEET.

NOTE 4: FOR PHASE 4, TIME WAS INCREASED TO ACCOUNT FOR THE LOWER CROWN ELEVATION OF THE LMT RELATIVE TO BEACH ELEVATIONS.

SOURCE: Existing Grade from USGS; Sewer Infrastructure & Image from City of San Francisco



**Figure ii**
Plan View of Proposed Protection

ESA

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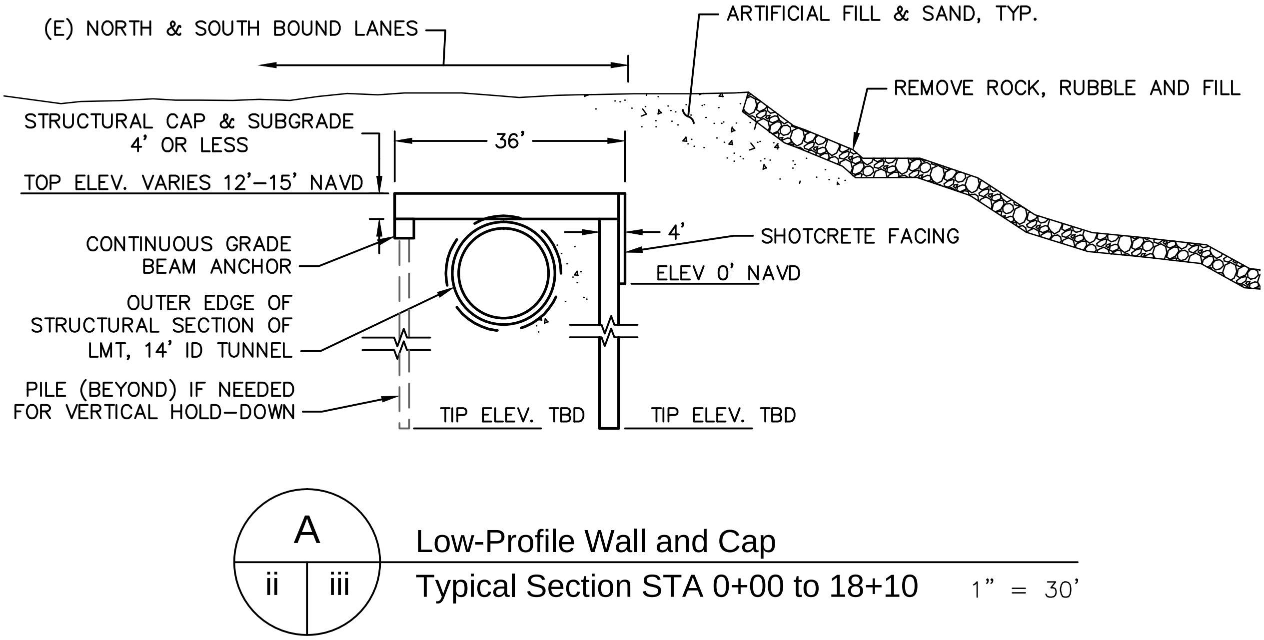
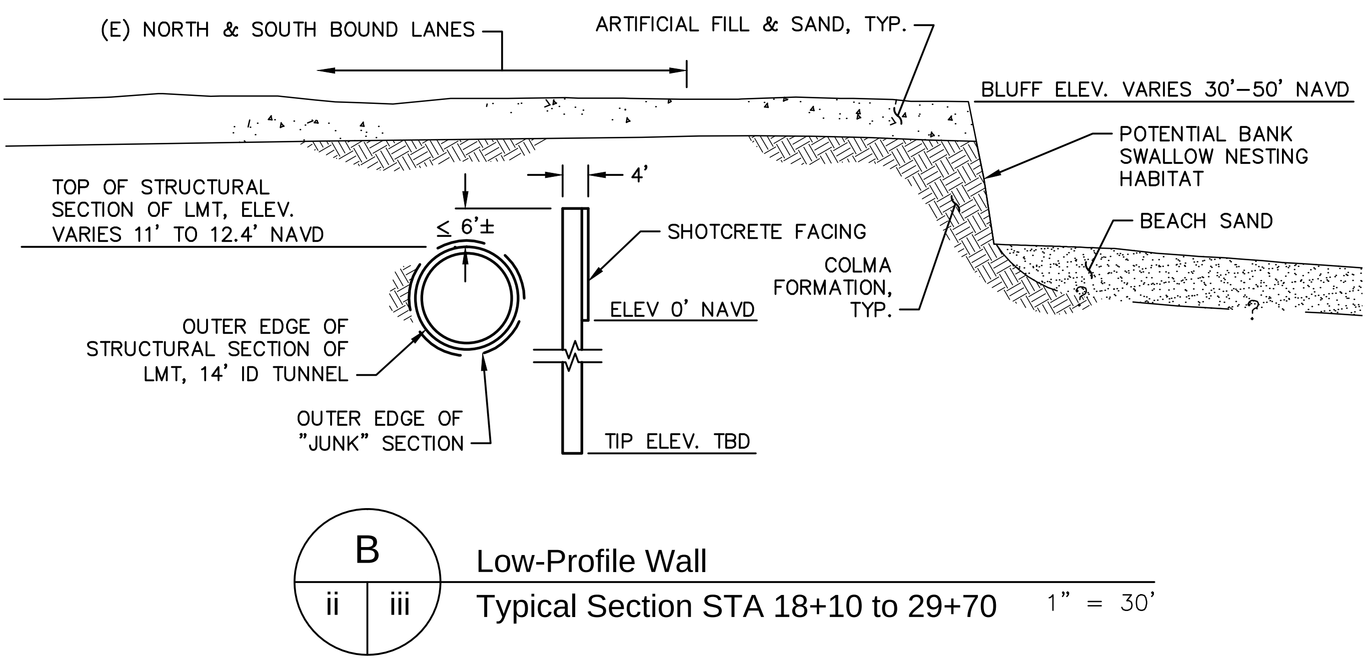



Low-Profile Wall and Cap

Typical Section STA 0+00 to 18+10 1" = 30'




Low-Profile Wall

Typical Section STA 18+10 to 29+70 1" = 30'



**Figure iii**

Typical Sections A and B:
Low-Profile Protection of LMT

ESA

- The project could be implemented in phases to limit the extent of coastal armoring to the minimum required during any particular time frame and to couple rubble removal and shore enhancement with the LMT armoring. Major factors affecting a phasing approach include, recommended further studies (Section 1.4) and:
  - Risk: Address the most at-risk reaches of shore first, with subsequent phases being implemented when monitoring indicates that triggering conditions have been reached
  - Regulatory Compliance: Locations with unpermitted development are likely candidates for early shore enhancement.
  - Mobilization and Regulatory Compliance Cost Efficiencies: Implementing only what is necessary must be weighed against the efficiency of fewer, larger implementation phases, the potential for duplicative environmental impacts, and the uncertainty associated with how quickly natural forces can reduce or eliminate buffers.
- The project site has been divided into four phases (Figure i). This report proposes starting with implementation of Phase 1 and Phase 2.
  - Phase 1: Approximately 800 linear feet at the EQR reach and Reach 3, likely consisting of a secant pile wall, excavation of imported fill, and possibly a horizontal cap.
  - Phase 2: Approximately 800 linear feet at Reaches 1 and 2, likely consisting of a vertical wall only.
- Phase 3 and Phase 4 could be implemented based on the estimated time to reach the trigger (Table 2). However, CCSF may decide to implement the phases differently based on risk, cost efficiencies and regulatory requirements.
- The OBMP "protect-in-place" strategy is considered feasible from engineering and coastal morphology perspectives, and is expected to be accepted by a broader range of stakeholders as long as the shore condition is improved in accordance with overall planning objectives.
- The shore is not sustainable in its existing configuration because it is too far seaward. Based on the shore response analysis, a sandy beach can be maintained at SOB through 2050 with sand nourishment and implementing interim and long-term recommendations which allow for managed retreat (i.e. remove existing rubble and armor and allow shore to realign landward).
- Extreme winter and storm erosion may expose the LMT protection during the project life. Additional sand nourishment can be employed to accelerate recovery from extreme conditions.
- Dredging to maintain the Federal navigation channel to San Francisco Bay generates sufficient sand to accommodate beach maintenance at SOB for the project life.

**TABLE 2**
**SUMMARY OF PHASES AND RECOMMENDED IMPLEMENTATION SCHEDULE**

| Phase | Reaches Included | Approximate Length of Phase[1] | Existing Coastal Armoring | Existing Bluff Width Seaward of LMT[2] | Trigger Distance Seaward of LMT[3] | Remaining Bluff Width Seaward of Trigger Distance | Estimated Time Until Action Required[4] |
|---|---|---|---|---|---|---|---|
| 1 | EQR & Reach 3 | 800 feet | Engineered Revetment & Sandbag Revetment | 59 feet | 35 feet | 24 feet | 8-24 years |
| 2 | Reaches 1 & 2 | 800 feet | Engineered Revetment (Reach 1); Rubble (Reach 2) | 75 feet | 35 feet | 40 feet | 13-40 years |
| 3 | Rubble Reach | 800 feet | Extensive Non-Engineered Rubble | 116 feet | 35 feet | 81 feet | 13-40 years[5] |
| 4 | North Lot Reach | 650 feet | Sand & Non-Engineered Rubble | 79 feet | 35 feet | 44 feet | 15-50 years[6] |

1 See Table 3 for details of bluff toe and top elevations, LMT elevations, depths of cover over the LMT, and horizontal offset of bluff toe from LMT by reach.

2 Existing bluff calculated as distance from LMT to the bluff toe, based on USGS beach survey from January 2010. Varying widths; narrowest measurement utilized for conservative calculation. The bluff toe throughout the project area for 2015 is farther seaward due to sand placement in 2012 and 2014-2015, and confirmed by review of April 2015 survey data provided by Daniel Hoover, USGS.

3 The trigger distance of 35 feet is the sum of the 10 foot structural buffer and 25 foot safety buffer. See Table 1 for definition.

4 Estimate of time for bluff toe to reach trigger distance is based on a range of erosion rates for unarmored bluffs of 1 to 3 feet per year. Because most of the bluff is partially or fully armored, the estimate is conservative. However, unarmored bluffs or sand dunes (vs. Colma) could erode approximately 25 feet in an extreme event.

5 For Phase 3, time was reduced to account for rubble removal, with shoreline adjustment of 40 feet.

6 For Phase 4, time was increased to account for the lower crown elevation of the LMT relative to beach elevations.

## 1.4 Recommendations for Further Analysis

The following analyses are recommended to inform final project criteria, including design:

- Perform additional geotechnical field investigations and evaluation of the subsurface conditions to inform the locations and characteristics of geologic features, including the extent (vertical and horizontal) of the Colma formation.
- Refine geostructural modeling of the LMT updated per the findings of the geotechnical field investigations.
- Evaluate the seismic loading conditions for the LMT to inform the design.
- Refine estimates of storm erosion potential based on actual location of the Colma formation and performance of beach nourishment (e.g. sand backpass efforts and anticipated USACE sand placement).
- Refine beach nourishment program and project design, including triggers, targets (e.g. desired beach widths), storm response, and monitoring.

- Prepare Conceptual Engineering Report using the additional data and analyses.
- Develop and implement a monitoring program to inform implementation and assess performance, as described in Section 6.6.5.

# 2 INTRODUCTION

The OBMP was charged with looking at all major aspects of the beach for 50 years and beyond. By taking a decidedly long view, developing a consensus vision and working backward to arrive at short- and longer-term actions, the master plan provided the framework for decision making. It presented a framework for understanding the wide range of issues and challenges at Ocean Beach and a series of recommendations for balancing the many priorities and objectives identified by local agencies and stakeholders.

The Ocean Beach Coastal Management Framework (CMF) comprises a series of technical studies for the SFPUC and partner agencies. It is developing a framework for implementing the recommendations in the OBMP (OBMP; SPUR 2012) to manage the shore in the short and long term, while recognizing it would likely take years to implement. The focus of this study is South Ocean Beach (SOB), the area south of Sloat Boulevard where erosion hazards are chronic (Figure 1). Its intent is to protect the CCSF's wastewater infrastructure while incorporating ecological and recreational objectives using concepts and ideas formulated in the OBMP.

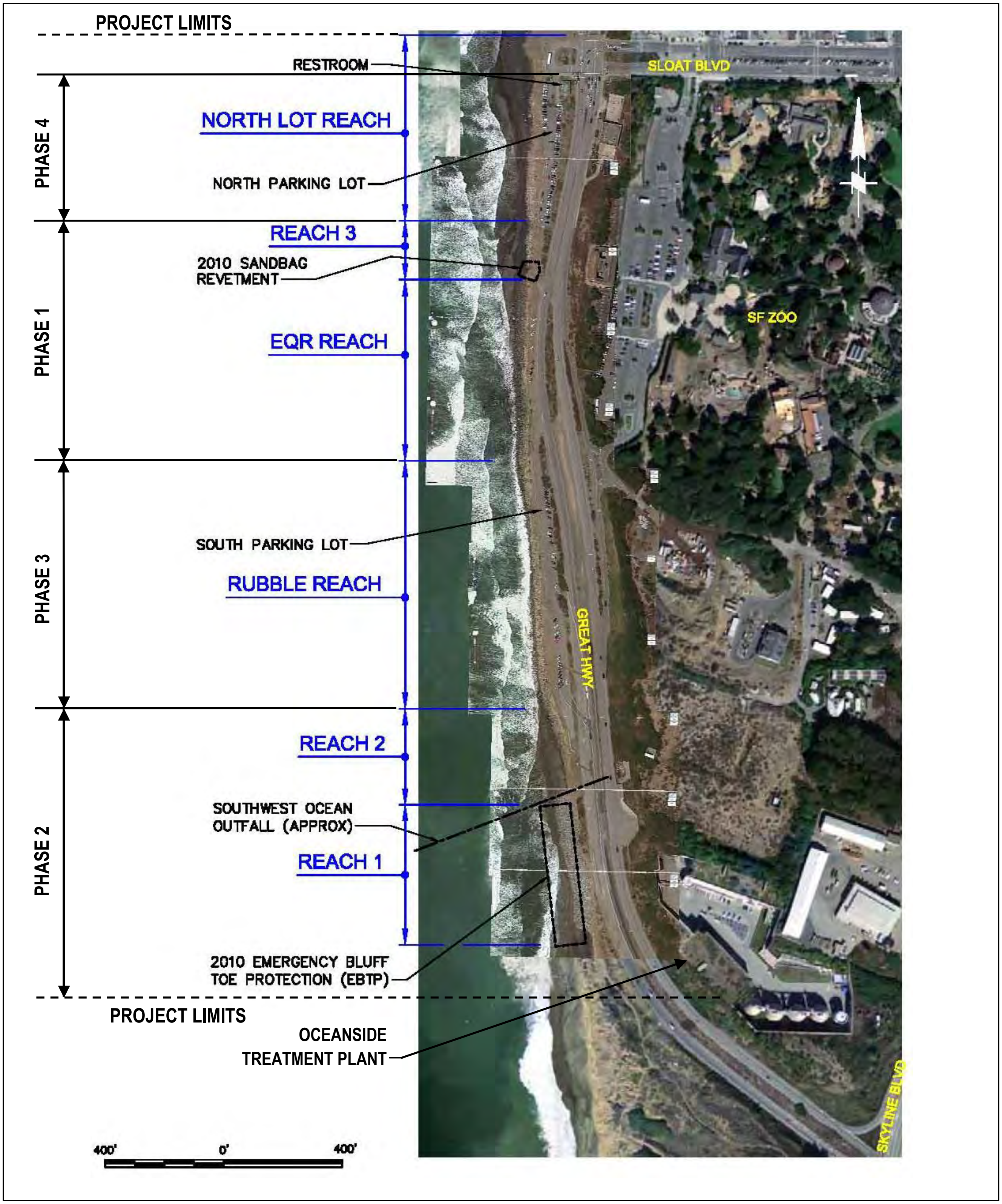
## 2.1 About This Report

This report represents Task 2 of the CMF (Coastal Engineering Feasibility Studies [see Scope of Work, Appendix 6]) that focuses on the interim interventions and long-term implementation of the OBMP for SOB, specifically over the 10-40 year time horizon for protection through 2050. Here, a methodology is presented for assessing the vulnerability of the LMT and evaluating the feasibility of a series of erosion protection approaches.

This report refines the assessment of vulnerability of the LMT tunnel based on a more precise understanding of the subsurface geological conditions and other factors that support a reduction in the lateral and vertical buffers recommended for the LMT. A data gaps analysis was performed to coordinate with local entities, including the CCSF, the U.S. Geological Survey (USGS), the U.S. Army Corps of Engineers (USACE), and the National Park Service (NPS) to identify data sources that would inform development of the preferred project concept and ongoing design and monitoring efforts (ESA 2013). The team also evaluated the feasibility of implementing different measures to protect the tunnel in place while also improving the recreational and ecological conditions of the beach and bluffs south of Sloat Boulevard.

The information presented in this report was developed in coordination with a Technical Advisory Committee (TAC), a select group of technical experts in a range of relevant disciplines, including coastal management, littoral and climate science, infrastructure, and regulatory processes. Three TAC meetings were conducted during the course of this study. The TAC provided advisory feedback and guidance on the project approach and methodology, helping to vet and strengthen the project team's approach, emphasizing technical and methodological issues within the established framework of the study and the broader OBMP implementation process. Appendix 1 summarizes and directly addresses comments provided by the TAC.

The remainder of this report is organized into four major sections that help to frame the project rationale and recommend a project concept based on sound technical analysis. First, the overall project goal and the guiding principles and objectives are presented to frame the problem and constrain the possible solutions (Section 3). Next, the assessment of vulnerability and description of imminent hazards are used as a basis for protection concepts and site improvements (Sections 4 and 5). Section 6 describes the preferred project concept as a first phase of the overall coastal protection program that can be repeated for additional segments in subsequent phases.




SOURCE: Moffatt & Nichol (2012)



**Figure 1**
Project Site and Definition of Reaches

## 2.2 Project Context and Need

Previous studies have broken South Ocean Beach into series of reaches (see Figure 1) corresponding to distinct physical conditions. These include existing armoring, the presence of exposed rubble and debris, the proximity of bluffs to infrastructure and amenities, ongoing coastal processes and the native geological formations. The physical conditions are also a reflection of the management and regulatory history of the shoreline, including:

- Installation of the Great Highway over several decades; including the use of dumping fill to serve as the roadbed. Some of this fill material has become exposed (e.g. rubble) during severe erosion events.
- Installation of the wastewater system (including the LMT) under the Clean Water Program in the 1980s and 1990s. This work included narrowing the Great Highway (8 lanes to 4), constructing parking lots, installation of existing dune-like sand embankments, and considerable planting of vegetation and amenities. This included placement of fill for access facilities. Some of this fill has since eroded and become exposed and parking and other amenities have been severely eroded.
- Emergency coastal armoring in the form of boulder revetments placed in 1997-1998 (EQR reach) and 2010 (Reach 1). These revetments, installed as emergency protection, present ongoing regulatory compliance issues. Because they present barriers to access, cover portions of the beach, and disrupt coastal processes, they are highly controversial among stakeholders and regulators, including the CA Coastal Commission. The safe removal of these revetments and the expected resolution of all Coastal Commission permitting issues is an objective of this project.
- Softer coastal protection measures, which reflect changes in local practices after the Coastal Commission denial of a CCSF permit application in 2011, have been utilized. Newer measures include a sandbag revetment (Reach 3) and significant sand placement projects, which have improved the appearance and access conditions at the North Lot/Reach 3 (2012) and Reach 2 (2014).

In 2012, SPUR completed the OBMP, a vision document that emerged from an in-depth interagency and public planning process. The OBMP recommended a coastal management approach that would protect the existing infrastructure while adapting to climate-induced sea-level rise and improving access, aesthetics, and ecological function.

Other implementation efforts are developing OBMP recommendations. The Ocean Beach Transportation Study includes the traffic analysis necessary to implement in a step-wise fashion the full closure of the Great Highway South of Sloat Boulevard and accommodate related changes in the circulation system. The Ocean Beach Open Space Design project is developing landscape designs that improve public access even as a multi-stage managed retreat process unfolds.

The primary driver of this report is the vulnerability of the LMT to erosion (which is characterized in detail in Chapter 5) but this project takes a comprehensive, multi-objective approach. It develops and tests OBMP coastal management recommendations, and validates the concept of a low-profile structural protection system. This approach minimizes disruption of coastal processes, allows for the removal of intrusive coastal armoring and accommodates surface restoration and enhancement.

The existing revetments, exposed fill, and eroding parking lots have resulted in degraded access and ecological conditions, and reflect decades of *ad hoc* coastal management. With a clear strategy now in place, rooted in an informed, values-based process (e.g. TAC process), Ocean Beach will likely be a model of adaptive, resilient coastal management.

## 2.3 Coastal Management Framework Planning Time Horizons

The purpose of this report is to develop feasible coastal protection solutions consistent with the OBMP. It also identifies interim coastal management measures that are compatible with long-term solutions.

The CMF spans three main timeframes that are the basis for progressive management of the shore at SOB in a manner consistent with the OBMP. A summary of the three time horizons is as follows:

> **Long-term Planning (10-40 Year Timeframe starting 2020):** Continued implementation of OBMP coastal management recommendations; includes phased adaptive management approach; likely to include complex regulatory and implementation processes that will be enacted over the next 10 to 40 years. It is expected that diligent effort will result in completion of designs and approvals, funding and construction by then. Subsequently, it is expected that the implementing agencies will reassess the OBMP around 2030, with an understanding that sea-level rise and other factors will be better defined and revised in the plan at that stage. At that time, agencies will refine the OBMP for the coming decades (2050 to 2100+).
>
> **Interim Interventions (3-10 Year Timeframe starting 2018):** This period is likely too early for full implementation of OBMP recommendations, but is critical for establishing the direction and compatibility with future, long-term implementation of OMBP recommendations; detailed designs, pilot studies, regulatory processes and interim measures will proceed during this period directed toward full implementation. Ongoing management of the shore at SOB during the interim period will be informed by the results of the vulnerability and adaptation feasibility study, as well as the development of a proposed Interagency Coastal Management Agreement.
>
> **Immediate Needs (1-3 Year Timeframe starting 2015):** The need to minimize short-term risks and other ongoing erosion and/or damage issues is currently being addressed through yearly sand nourishment activities. Additional measures are being considered as part of the *Immediate-Term Coastal Erosion Management Plan* (Immediate Plan) currently under review.

## 2.4 Key Acronyms & Abbreviations

The following list contains key abbreviations used throughout this report and this study:

| | |
|---|---|
| CIDH | Cast-in-drilled-hole |
| CCC | California Coastal Commission |
| CCSF | City & County of San Francisco |
| CMF | Coastal Management Framework |
| CY | Cubic Yards |
| ESA | Environmental Science Associates |
| EQR | Emergency Quarrystone Revetment |
| IPCC | Intergovernmental Panel on Climate Change |
| LMT | Lake Merced Transport |
| M&N | Moffatt & Nichol |
| MCY | Million Cubic Yards |
| MLLW | Mean Lower Low Water |
| MHHW | Mean Higher High Water |
| MOB | Middle Ocean Beach |
| NOB | North Ocean Beach |
| NPS | National Park Service |
| NRC | National Research Council |
| OBMP | Ocean Beach Master Plan |
| OTP | Oceanside Treatment Plant |
| OPC | Ocean Protection Council |
| PWA | Philip Williams & Associates |
| RSMP | Regional Sediment Management Plan |
| SFPUC | San Francisco Public Utilities Commission |
| SFPW | San Francisco Public Works |
| SLR | Sea-level rise |
| SOB | South Ocean Beach |
| SPUR | San Francisco Planning and Urban Research Association |
| SWOO | Southwest Ocean Outfall |
| TAC | Technical Advisory Committee |
| USACE | U.S. Army Corps of Engineers |
| USGS | U.S. Geological Survey |

# 3 COASTAL MANAGEMENT FRAMEWORK PRINCIPLES & OBJECTIVES

## 3.1 Project Goal

Develop a coastal management strategy for Ocean Beach using a multi-objective approach that protects critical infrastructure, supports ecological functions, and improves aesthetics and public access.

## 3.2 Guiding Principles

The intention of the CMF is to further develop and refine the coastal management approach recommended in the Ocean Beach Master Plan. To that end, the CMF's guiding principles include the following:

- Multi-objective approach, incorporating
  - Protection of the LMT and associated infrastructure
  - Ecological function
  - Recreation and access
  - Aesthetics and landscape character
- Emphasis on softer solutions wherever possible
- Adaptive management, incremental implementation
- Compatibility with OBMP recommendations

## 3.3 Operational Objectives

Several operational objectives were developed to relate the project outcomes to the overall project goal. The following operational objectives are used to clearly establish that actions will be consistent with the project goal:

- Protect the LMT and address ongoing chronic erosion hazard by:
  - Defining structural buffers, safety buffers, trigger distances required for action, and base them on :
    - Characterization of subsurface conditions
    - LMT structural requirements, and
    - Erosion hazards

- Defining a preferred set of structural interventions
- Planning a sequence of related actions over time
- Addressing interim risk in a manner compatible with long-range vision
- Developing interim solutions that allow for transition into long-term solutions or that are reversible
- Avoiding impacts to sensitive species known within the project area
- Basing shore management decisions on the vulnerability of the LMT rather than the vulnerability of surface treatments, such as the road and parking lots
- Identifying an interim strategy that will be consistent with a proposed interagency management agreement for SOB

- Improve ecological and habitat conditions through:
  - Softer, more reversible physical measures wherever feasible, such as sand placement, sandbags, temporary structures
  - Enabling a wider beach using managed retreat/realignment, beach nourishment, and cleanup
  - Restoring native conditions and processes where possible through cleanup and removal of fill and revetments, native bluff and beach, and vegetation of bluff and dunes
- Improve Public Recreational Access, through:
  - Removal of hazardous, unwelcoming conditions
  - Improved beach width and aesthetic conditions
  - Integration of clearly defined coastal access points
  - Integration with ongoing open space and circulation planning
  - Facilitation of coastal trail and other recreational improvements
- Develop regulatory/adaptive management regime that:
  - Allows proactive planning, funding, and regulatory clearance of management actions
  - Could be implemented incrementally in response to defined triggers
  - Provides for improved interim conditions at every stage

# 4 METHODOLOGY & APPROACH FOR ASSESSING LMT VULNERABILITY & ADAPTATION FEASIBILITY

Assessment of the LMT vulnerability is based on an analysis of subsurface conditions and projected bluff recession, including existing geotechnical conditions, existing structural condition of the LMT, lateral and vertical cover requirements over the LMT, and assumed wave conditions. The project team has reviewed available subsurface and geotechnical data and reassessed bluff recession rates and corresponding trigger distances required for protection of the LMT. The vulnerability assessment is based on two primary lines of analysis, integrated and updated to the extent practicable:

1. Long-term geomorphic response modeling in response to sea level rise, sand supply and shore management actions (Appendix 1; Appendix 2); and,

2. Episodic erosion, along with geomorphic change, that violates minimum dimensions of burial for structural stability of the LMT (M&N 2012; Appendix 3).

The team evaluated the feasibility of several alternatives to protect the LMT in place based on project objectives (see Section 3), including protecting the LMT with an approach that is compatible with the long-term Ocean Beach Master Plan vision (e.g. low-profile structure, trigger-based implementation, reversibility, etc.), ecological improvements to habitat conditions, and improvements to public access.

The following sections (4.1-4.4) provide a description of the methodology by discipline. Each of the interrelated analyses considered the shore as a series of "reaches" (see Figure 1) distinguished by differing existing shore conditions (e.g. armored, unarmored, etc.), geological formations, and tunnel proximity to the bluff edge.

## 4.1 Coastal Erosion and Response to Sea-Level Rise

This study uses existing data and assumptions in the OBMP to address sea-level rise (SLR) over the planning horizon of the structural elements. This approach is based on translating the beach profile landward as it adjusts to SLR, with accelerated shore and bluff retreat. This approach, referred to as geomorphic response modeling, and assumed sea level rise scenarios are consistent with the OBMP studies, and further described in Appendix 1. The approach is based on California State Guidance for Sea Level Rise (OPC 2013), and is consistent with CCSF Guidance on how to incorporate SLR into capital planning (CCSF SLR Guidance 2014).

The vulnerability of the LMT was evaluated using an erosion-based approach defined by trigger distances (M&N 2012; Appendix 3). The trigger distance is the sum of the buffer distance needed to provide adequate restraint for the tunnel (e.g. Structural Buffer, Table 1) to maintain its structural integrity, and an additional buffer (e.g. Safety Buffer, Table 1) for action in response to rapid and episodic bluff erosion. Updates to these buffer and trigger distances were based on the review of available information, including prior studies, erosion estimates, and additional technical analysis including tunnel modeling. The buffer distance was reduced based on updated geostructural modeling of the tunnel (see Section 4.1.2). However, insufficient data were available to provide a complete analysis for updating the additional trigger distance beyond the buffer. Additional geotechnical field data is recommended to fill key data gaps on geologic variability of the bluff materials, including the presence of the relatively stiffer Colma formation within which the LMT is embedded, concrete rubble and debris that is present on the beach and in the bluff fill, and the interaction of existing coastal structures that serve as protection to the bluff with the local wave climate, hydrology, and geomorphology. Additional subsurface exploration, which is required for subsequent design, can be organized to fill these data gaps and improve the assessment of erosion and future vulnerability.

## 4.2 Tunnel Vulnerability

McMillen Jacobs Associates (Appendix 4) has incorporated existing information from original design and construction records to refine the criteria used to define the vulnerability of the LMT, including possible alternatives for protecting the LMT in place, such as structural wall concepts and structural reinforcing of the pipe. McMillen Jacobs Associates evaluated the LMT behavior based on a range of short-term and long-term solutions to understand the interaction of the LMT and surrounding bluffs.

McMillen Jacobs Associates modeled the LMT and surrounding soils in a three-dimensional finite element model (Appendix 4). Input parameters for modeling the LMT support were derived or obtained from existing literature, including existing engineering data that describes how the tunnel support was designed and constructed. McMillen Jacobs Associates derived applicable geologic parameters from the literature using information on the existing soils conditions surrounding, and above, the LMT. Using the tunnel and ground support model, evaluation of the LMT's response to reduced lateral restraint and vertical overburden informed the short-term and long-term solutions. The model is also used to run parametric studies to assess the LMT's sensitivity to movement by varying engineering parameters within the model. These studies are used to establish boundaries within which the external changes can take place and still maintain the LMT within acceptable levels of safety.

## 4.3 Subsurface Conditions

AGS reviewed existing information from the project vicinity and considered the geologic variability of the sand, Colma formation, concrete rubble, and fill areas. AGS provided geotechnical parameters to McMillen Jacobs Associates for use in their tunnel modeling.

Interpretations of boring records were reviewed with the objective of refining the short-term and long-term erosion rates and soil properties in the vicinity of the LMT. Previously collected and analyzed borings indicate existing areas of higher strength Colma formation relative to adjacent fill and dune sands. However, some boring logs do not distinguish between Colma and Merced formations, which represent an important interface for assessing bluff failure mechanisms.

Furthermore, AGS reviewed records of geologic and geotechnical conditions that were encountered during construction of the LMT. The tunneling records indicate that the northern 1,800 linear feet was wholly or partly in sand deposits and imported fill that were not as strong as the expected Colma formation, which then transitioned into the stiffer Colma formation. The bluffs at the southern end of the reach are composed of the Colma material, and stand vertical, compared to the northern 1,800 feet which is characterized by eroding imported fill and concrete rubble.

Additional geotechnical field data collection is recommended for subsequent study and design stages of the project. Updates to the vulnerability and triggers rely on accurate understanding of the spatial distribution of materials and their varying properties. Likewise, structural design of the protection device may be impacted under different types of physical conditions. Therefore, the team recommends that a geotechnical field data collection program be implemented to improve the understanding of the geologic conditions at the site.

## 4.4 Protection Measures Under Consideration

A range of bluff protection solutions were developed in the OBMP process. These included soft solutions, such as beach nourishment and dynamic cobble revetments, as well as structural measures. The protection measures evaluated for the long-term protection of the LMT include:

- **Taraval-type wall** consisting of a wall seaward of the tunnel and a structural cap that would provide structural hold down of the tunnel that is equivalent to soil requirements, resulting in a low-profile finished surface (similar to the existing wall on the beach at the end of Taraval Street). The cap element provides resistance to LMT uplift, armors the LMT against wave action and scour, and provides a reasonable surface for access. The cap also minimizes the height of the low-profile protection, but may entail a cost premium. This report considers two wall alternatives: Pile wall and Soil Mix wall.
    - **Pile wall** constructed by drilling piles from the top of the bluff and located in front of the tunnel. The piles can be augured and cast in place up to the desired elevation. For a low profile armoring, the wall would not extend to the top of the existing bluff but rather to an elevation that provides minimum vertical and horizontal cover over the tunnel, and could be modified to include a cap in the future.
    - **Soil Mix wall** embedded in bluff constructed by in-situ grouting that can be converted by adding a “cap” in the future. This method could also be used in lieu of the “cap” to maintain adequate ballast (hold-down) and armoring above the LMT.
- **Toe wall**, or soldier pile and lagging wall, at the seaward edge of bluff that would hold the toe of the bluff and maintain a slope that provides at least the minimum vertical and horizontal cover over the tunnel. This wall would be farther seaward but would be removed as erosion progressed in favor of another landward wall or other approach. This structure is

considered primarily as a temporary measure if other measures are not available within the time frame required.

- **Other hold-down methods**, such as saddles, that would replicate the vertical and horizontal loading of the soil, but would allow for a low-profile finished surface.

- **Structural modification of tunnel** that would include structural strengthening to modify the cross section of the tunnel to withstand reduced cover vertically and horizontally. This measure was culled from detailed consideration due to the longer lead times associated with the potential requirement to modify discharge permits if tunnel capacity is reduced.

- **Relocation of the tunnel** was assessed previously, and the results were summarized.

# 5 COASTAL VULNERABILITY OF THE LAKE MERCED TUNNEL

This section presents the findings of the vulnerability assessment of the LMT to coastal erosion in SOB. The vulnerable reaches (see Figure 1) are described and characterized according to existing and future erosion hazards. At present, the most vulnerable section of the LMT is where its alignment is the farthest seaward, in the vicinity of the EQR, a rock revetment constructed in 1997-1999. As a result of the analysis, updates to the Structural Buffer (horizontal and vertical), Safety Buffer and Trigger distance are included (e.g. Structural Buffer was reduced from 25 feet to 10 feet after geostructural modeling of the tunnel and bluff characteristics were analyzed). Although the LMT is least vulnerable to a reduction in the vertical overburden, approximately six feet of overburden (e.g. vertical structural buffer reduced to 6 feet from 15 feet) is required to counteract buoyancy forces in the absence of other structural measures to hold the tunnel down.

## 5.1 Vulnerable Reaches

The vulnerability of the LMT varies by reach along the shore at SOB as well as over time. All of SOB has strong potential for future erosion and therefore, over the long-term, most of the coastal segments of the LMT are vulnerable to coastal erosion. Interim-term vulnerabilities, however, are located in distinct areas where the bluff toe is closest to the LMT and is relatively unarmored. Moffatt and Nichol (2012) divided the study area of SOB into reaches that are distinguished by variable types and timing of erosion protection (see Figure 1). Concrete rubble and quarry stone revetments armor most of the shore, except for an area protected by the 2010 sandbag revetment and another in Reach 2 just north of the Southwest Ocean Outfall (SWOO).

The vulnerable reaches along the shore were grouped into four phases (see Section 6.4) to be implemented according to their relative degree of vulnerability:

- **Phase 1:** Reach 3 with the 2010 sandbag revetment and along the EQR Reach at which a rock revetment is showing some deterioration.
- **Phase 2:** Reach 1 where the 2010 Emergency Bluff Toe Protection, a large quarrystone revetment, was constructed, and Reach 2 where rubble is limited to lower beach levels.
- **Phase 3:** Rubble Reach, which is a stretch of shore that is protected by a large quantity of concrete rubble, is effectively an un-engineered shoreline protection structure.
- **Phase 4:** North Lot Reach where the NPS facilities, including public parking and restrooms, and primary SOB public access are located.

## 5.2 Horizontal Structural Buffer and Trigger Distance

The distance between the LMT and the shore is an indication of the available time before coastal erosion reaches the LMT. However, there is a minimum distance of earth cover needed to maintain lateral support for the tunnel. Also, erosion often occurs episodically, in short storm and winter periods, more quickly than the time needed to properly implement long-term adaptive actions such as structural armoring.

The distance required for structural integrity, often called lateral cover, is herein called the *Structural Buffer*. The additional distance beyond the *Structural Buffer* to allow for adequate reaction time is called the *Safety Buffer*. The sum of the *Structural Buffer* and *Safety Buffer* establishes a *Trigger Distance* for action and is located seaward from the LMT (Figure 2).

Previously, during the response to 2010 erosion, a structural buffer distance of 25 feet was estimated as a provisional minimum to provide adequate support to the LMT (M&N 2012). Based on the potential erosion of unarmored bluff that is associated with an approximate 15- to 20-year recurrence interval, an additional 25-foot safety buffer was defined. This resulted in a trigger distance 50 feet seaward of the LMT. However, the provisional safety buffer did not account for existing armoring, which greatly reduces the risk of bluff erosion, and, in effect, suggests that the safety buffer distance of 25 feet is conservatively high along armored bluffs. Similarly, the presence of erosion-resistant Colma formation would also reduce the required safety buffer to less than 25 feet.

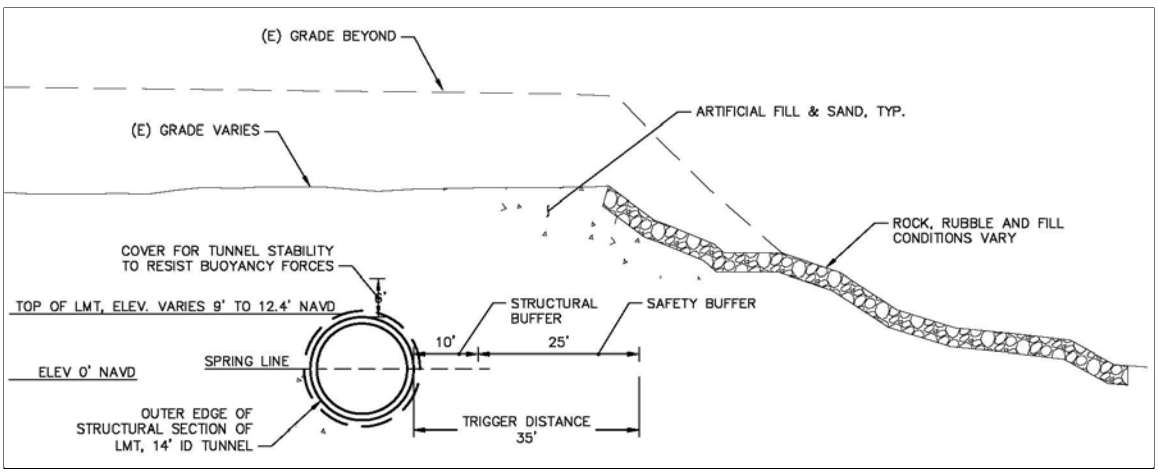



**Figure 2**
Typical Section of LMT and Bluff:
Structural Buffer and Trigger Distances

Based on the results of tunnel modeling, the lateral restraint is adequately maintained as long as the bluff is intact at least 10 feet seaward of the LMT (Appendix 4). The modeling analyzed three conditions where the bluff erosion occurred:

- Over the full vertical height of the tunnel,

- For the bluff toe located near the spring line of the tunnel, and
- For the bluff toe located near the elevation of the crown of the tunnel.

The existing bluff toe is typically located at an elevation near the crown of the tunnel and fluctuates vertically to as low as 2 feet MLLW[2], approximately the spring line of the tunnel (see Figure 2). The condition when the beach lowers is considered a temporary condition, and one that is typical of armored locations. Further, structural damage to the LMT is unlikely in the event that the bluff toe encroaches into the 10-foot buffer because the structural design parameters were derived using conservatively high factors of safety.

**The recommended *Trigger Distance* for LMT protection is a 35-foot horizontal distance between the bluff toe and the LMT**, based on a 10 foot Structural Buffer plus the 25 foot Safety Buffer (see Table 1; Figure 2). This Trigger Distance provides the basis for evaluating the vulnerability of the LMT (section 5.4 below). Note that coastal armoring can replace the Safety Buffer.

In the future, new geotechnical data and updated bluff retreat estimates, which better account for the apparent cohesion of the near-vertical Colma formation bluffs and existing armoring, may result in a revised safety buffer distance.

## 5.3 Vertical Structural Buffer

Prior studies identified a provisional minimum vertical burial depth of 15 feet to prevent deformation of the LMT due to buoyancy forces (M&N 2012). Buoyancy forces are incurred when the LMT is not flowing full (e.g. it may be nearly empty during dry weather operations), and the ground water level high. The maximum buoyancy force occurs if the LMT is empty and the groundwater level is above the LMT. However, based on the results of recent tunnel modeling, a reduction of the Vertical Structural Buffer to 6 feet above the LMT crown does not have an adverse effect on the structural integrity of the LMT (Appendix 4). The tunnel modeling used a high groundwater elevation, at the crown (top) of the LMT, and a maximum buoyancy force based on an empty tunnel. Therefore, the Vertical Structural Buffer of 6 feet is recommended. In practice, these conditions are unlikely to occur simultaneously, so these assumptions are conservatively high.

Alternatively, vertical restraint can be provided through structural means with no overburden. The 6-foot distance may be reduced based on consideration of soil strength based on additional data collection and analysis. This updated assessment allows us to focus on the lateral buffer, and is consistent with the vision of the OBMP to protect the LMT in place with a low-profile structure.

[2] MLLW refers to mean lower low water, the average of the lower low water height of each tidal day observed over the National Tidal Datum Epoch.

## 5.4 Vulnerability of LMT

Vulnerability is a measure of risk, and can be expressed as the product of the probability of damage and the consequence of damage (IPCC 2001; NRC 2012):

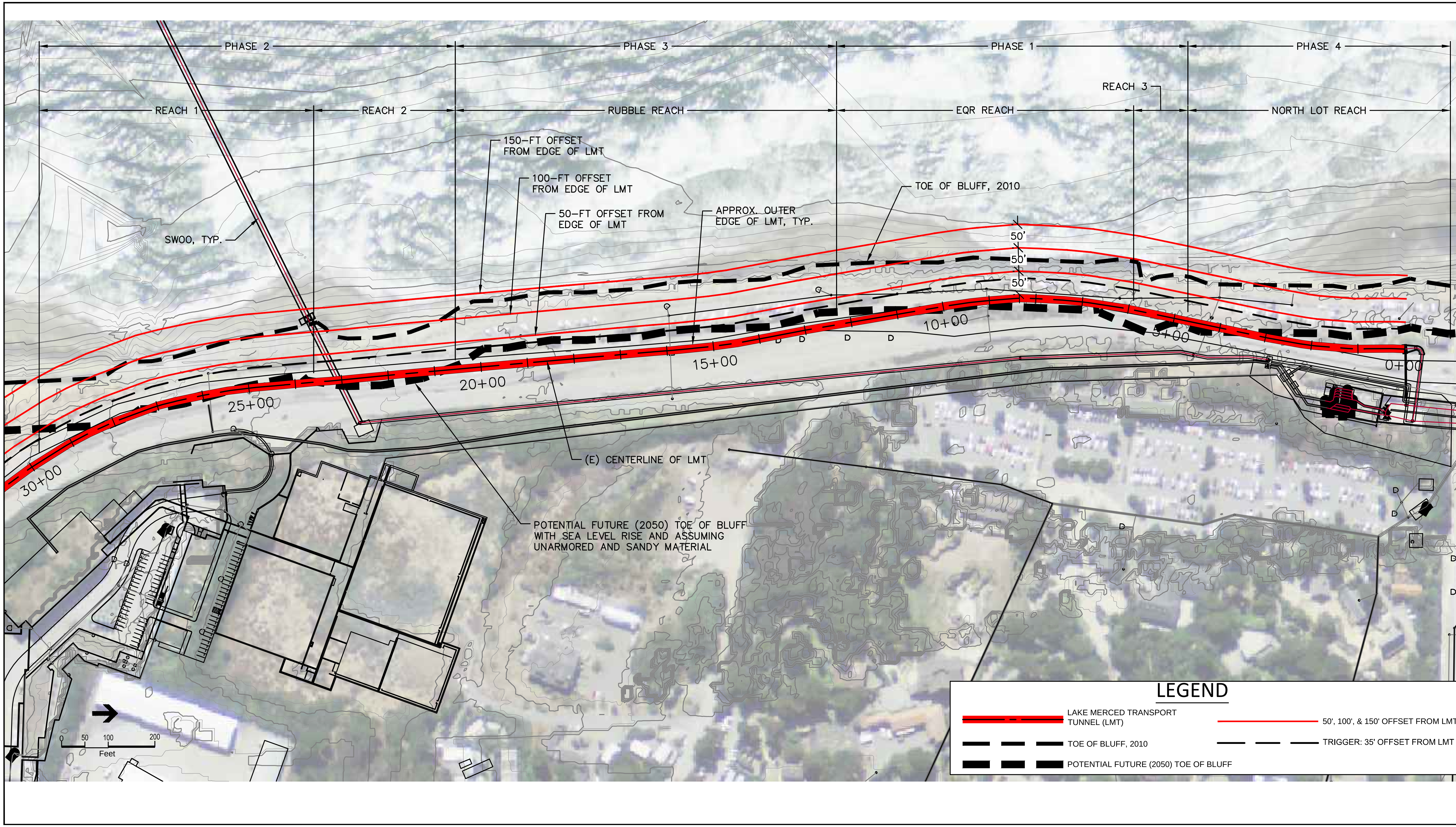
$$\textit{Vulnerability} = \textit{Probability} \times \textit{Consequence} \qquad (1)$$

Evaluation of vulnerability of the LMT under this framework then relies on defining the probability that damage occurs, and the consequence of the damage. The SFPUC and San Francisco Public Works (SFPW) anticipate that damage to the LMT would have potentially severe consequences, noting that LMT integrity relies on the surrounding soils to limit differential pressure from the soil on the tunnel exterior. The consequence of damage, conceptualized as structural failure directly resulting from exposure of the LMT, could be a combination of disrupted wastewater treatment operations, a decrease in storage of the Oceanside Treatment Plant (OTP), and a possible sewage spill. Therefore, because the consequence would be expected to be very high for any damage scenario, vulnerability has defaulted to the potential for damage which is described as erosion penetrating the structural buffer distance. Therefore, the potential for erosion to cause the bluff toe to penetrate the Horizontal Structural Buffer (e.g. 10 feet) has been used to assess LMT vulnerability.

### 5.4.1 Interim Vulnerability

The LMT is most vulnerable at discrete reaches along SOB, as described above in Section 5.2. The LMT is low, located below typical back beach elevations, and the bluff toe in all locations is farther seaward of the previous 50-foot trigger distance, and much of the shore is armored (Table 3; Figure 3). These dimensions were extracted from LiDAR[3] and aerial photography georeferenced relative to the LMT location, and the elevations were derived from USGS beach surveys. The horizontal and vertical location of the bluff toe is actually variable, moving down and seaward when beach conditions lower in the winter and spring, and moving up and landward when conditions include a high and wide sandy beach (see Figure 18 for example of how the bluff toe is affected by seasonal beach fluctuations). The dimensions in Table 3 are indicative of the conditions during the 2010 winter. Conditions since 2010 have not resulted in additional bluff toe recession, while sand placement has temporarily moved the toe seaward.

[3] LiDAR refers to Light Detection and Ranging, which is a remote sensing method that uses light in the form of a pulsed laser to measure ranges (variable distances) to the Earth, and is primarily used in topographic mapping to produce several types of digital elevation products for various applications.




SOURCE: Existing Grade from USGS; Sewer Infrastructure & Image from City of San Francisco

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**Figure 3**
Horizontal Offset of LMT to Bluff Toe

ESA

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**TABLE 3**
**TABULATION OF EXISTING BLUFF PARAMETERS IN RELATION TO THE LMT BY REACH**

| Reach Description | Reach Length (ft) | Horizontal Offset to Bluff Toe (ft) | Toe Elevation (ft NAVD)[1] | Bluff Top Elevation (ft NAVD)[2] | Tunnel Top Elevation (ft NAVD)[2] | Depth of Cover (ft) | Armoring at Toe |
|---|---|---|---|---|---|---|---|
| North Lot Reach | 650 | 79-142 | 13 | 30 | 8.5-9.0 | 20.9-21.4 | Sand berm; buried rubble |
| Reach 3 | 120 | 59-94 | 5 | 30 | 9.0-9.3 | 20.7-20.9 | Sandbag structure; buried rubble |
| EQR Reach | 640 | 75-120 | 4 | 30 | 9.3-10.1 | 19.9-20.7 | Rock revetment; constructed 1998 |
| Rubble Reach | 820 | 116-139 | 6 | 30 | 10.1-11.2 | 18.8-19.9 | Extensive amount of concrete rubble |
| Reach 2 | 300 | 75-120 | 12 | 30-40 | 11.2-11.6 | 18.8-28.4 | Relatively unarmored; limited concrete rubble |
| Reach 1 | 500 | 86-119 | 10 | 40-50 | 11.6-12.3 | 28.4-37.7 | Extensive rock revetment; constructed 2010 |

[1] Toe elevation estimated from USGS beach survey, January 28, 2010
[2] Basis of elevation from Table 1, Appendix 3, and converted from MLLW to NAVD88 and rounded

Although erosion of the bluff top has progressed, erosion of the bluff toe has been limited by rock revetments and concrete rubble. Sand placement along the North Lot Reach and Reach 3, and construction of large rock revetment at Reach 1, has also limited bluff recession. The 2012 sand backpass project improved the conditions at the North Lot Reach by constructing a wide sand berm with over 70,000 cubic yards of sand from North Ocean Beach. The erosion at SOB has not progressed substantially since the 2010 storms, and the beaches have recovered in width and elevation. Immediate actions, including sand placement and sandbag revetments can be constructed in response to the Trigger Distance to prevent migration of the bluff toe to within 10 feet of the LMT. Additional actions, such as emergency armoring, may be needed if segments of bluff erode within 10 feet of the LMT under extreme conditions during large winter storms (See Section 6.6.5 Monitoring).

### 5.4.2 Seismic Vulnerability

The LMT relies on support from the surrounding earth. Conceptually, ground accelerations during earthquakes may increase loads on the tunnel. Seismic loading was **not** addressed in this study, but should be considered in the design of the LMT protection, including a review of the trigger distances for future phases.

### 5.4.3 Long-term Vulnerability

Based on long-term estimates and projections of erosion, the LMT is located within the horizontal footprint of erosion hazards. The long-term geomorphic response of the shore at SOB was modeled to understand the potential for the LMT to be impacted by erosion due to SLR (Appendix 1; Appendix 2). Appendix 1 summarizes the SLR assumptions and geomorphic profile response used in the OBMP; Appendix 2 provides a separate description for shore response to SLR produced for the San Francisco Littoral Cell Regional Sediment Management Plan (RSMP). These approaches suggest that the bluff could recede between 110 and 190 feet by 2050 without any intervention. The upper range includes higher long-term recession rate of 2 feet per year (fpy) and an allowance for at least one severe El Niño winter or other extreme bluff erosion event in excess of that represented by the average annual erosion rate. However, uncertainties remain with regard to the rate of retreat for the bluff (composed of variable layering of Merced and Colma formations, ancient dune sand, artificial fill and rubble) versus erosion of the sandy shore. The bluff recession potential of 110 to 190 feet is likely conservatively high because it includes uncertainty that could result in higher recession and excludes factors that are known to potentially reduce recession, such as armoring, erosion-resistant subsurface conditions, and interventions. This "conservative" approach is consistent with prior work characterized by a very risk-averse approach to infrastructure damage and limited consideration of ecology and recreation impacts of proposed armoring. In contrast to prior work, shore management decisions are now based on the vulnerability of the LMT rather than the vulnerability of surface treatments, such as the road and parking lots. Therefore, protect-in-place alternatives that are lower profile, and hence less damaging to the beach function and services, are feasible.

## 5.5 Summary of LMT Vulnerability

Overall, implementation of one or a combination of alternative protection approaches identified in the OBMP will reduce the vulnerability of the LMT. However, significant data gaps remain, including:

- Subsurface information in the vicinity of the LMT, including the horizontal and vertical extents of the Colma formation and its erodibility relative to dune sands;
- Reduced bluff erosion rates that consider the presence of exposed Colma formation and existing armoring; and,
- Refinement of the action trigger distance to reflect the above.

The original design of the LMT considered and targeted the Colma Formation, while subsequent vulnerability estimates assumed that the material surrounding the LMT was composed of loose sand material, leading to increased tunnel vulnerability and greater trigger distances. These assumptions appear appropriate for the northern 1,800 feet of the LMT, where construction records indicate that the LMT is not encased in Colma formation. However, the available information does not allow high confidence in interpretations (see Appendix 5). Refinement of the action triggers requires additional studies, including a subsurface geotechnical field data collection program, and interpretation of the data to update the geostructural understanding of the LMT and its necessary horizontal and vertical buffers, as well as the additional distance for triggering actions.

The recent geostructural modeling of LMT tunnel deformation did consider the apparent cohesion of the Colma formation found in the bluff (Appendix 4). Results of parametric tunnel modeling show that reduction in lateral restraint provided by the bluff to less than 10 feet from the LMT could compromise the structural integrity of the tunnel. Therefore, the interim actions and treatments should maintain adequate bluff distance of 10 feet until a long-term protection strategy is installed closer to the LMT. Reduction of the vertical overburden to 6 feet, however, is feasible. Armoring of the shore can reduce or replace the safety buffer distance of 25 feet.

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# 6 PREFERRED PROJECT CONCEPT

The recommendations of the OBMP serve as the basis for identifying the possible adaptation approaches to protecting the LMT. The OBMP envisions a low-profile structure (Figure 4) that protects the LMT in place, and allows landscaping and surface treatments to improve the conditions of the beach and bluffs to a more natural state.

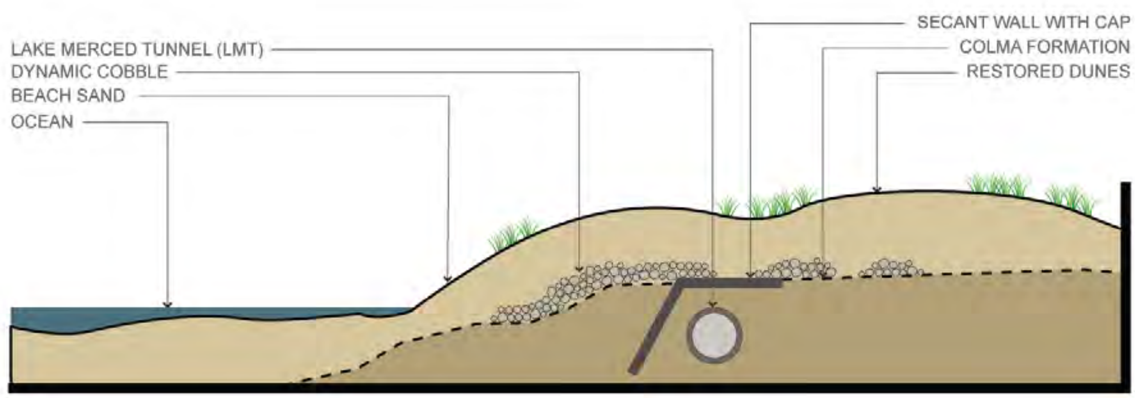



**Figure 4**
Ocean Beach Master Plan Vision and
Low-Profile Protection of the LMT In-Place

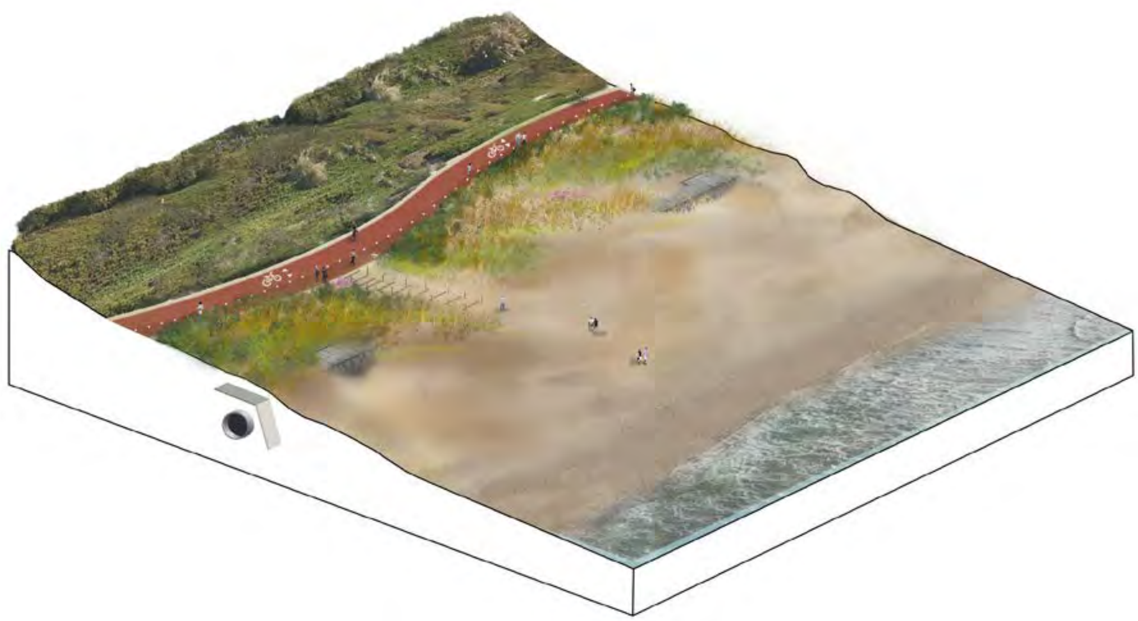
In order to achieve the operational objectives that are presented in Section 3, implementation of LMT protection is subject to the following criteria:

- Protects the LMT in place
- Cost efficient
- High degree of implementation adaptability
- Low degree of intrusiveness
- Compatibility with surface treatments, including ecology and recreation

Criteria will be refined as the concepts become more detailed and are further developed in subsequent design, environmental review and permitting.

The OBMP recommends a low-profile structure located in the elevation range of the beach, with less impact to sand movement and wave dissipation than traditional coastal armoring structures (Figure 5). This approach results in less environmentally-damaging protection scheme, in keeping with contemporary public works practice.

Surface treatments could include a dynamic cobble revetment to dissipate incident wave energy in a controlled manner, and provide a grade transition at the armoring, when exposed. Sand placement will be required to maintain a beach over the long term. Sand will be placed directly on the beach face to raise and widen the beach, and a linear back beach dune will be constructed that will "sacrificially" erode, thereby absorbing wave power and supplying sand to the beach. Native vegetation and access amenities are included in the open space design criteria.



Source: AECOM

**Figure 5**
Axon of the Ocean Beach Master Plan Long-Term Vision for LMT Protection and Improved Access and Ecology

## 6.1 Overview of Preferred Project Concept

The preferred project concept for coastal protection at Ocean Beach consists of a low-profile structure protecting the LMT, from which natural dunes and beach extend seaward. This requires the initial construction of the low-profile structure, followed by removal of various types of existing armoring (rubble, rock, and sand bags) and removal of existing site improvements (parking areas, landscaping, etc.). With the existing revetment removed, there are two options for the exposed bluff:

- Let the bluff erode until the low-profile wall is exposed, then place sand to re-establish the beach to the pre-eroded condition.
- Place sand immediately to create a widened beach and prevent landward bluff migration.

In either case, the placement of sand would be needed to form dunes and beach; in addition, landscape treatments including planting and public access elements would also be incorporated.

The low-profile armoring configuration is innovative but it has been successfully implemented at Ocean Beach, between Taraval and Santiago Streets. The Taraval Seawall is described in Section 6.6 of this report.

**Summary of proposed project – Phases 1 & 2**

- Renovate approximately 800± linear feet of shore for Phase 1, and approximately 800± linear feet of shore for Phase 2 (Figure 6);
- Install low-profile seawall to protect the LMT (Figure 7);
- Remove all armoring from beach and bluff;
- Remove all imported earth and rubble located 40 feet landward of bluff (primarily for Phase 1 areas that were filled);
- Place "beach sand" to backfill areas of excavation, and widen bluff and beach;
- Plant to stabilize sandy bluff and dunes; and,
- Install public space amenities, including vertical and lateral access and interpretive elements.

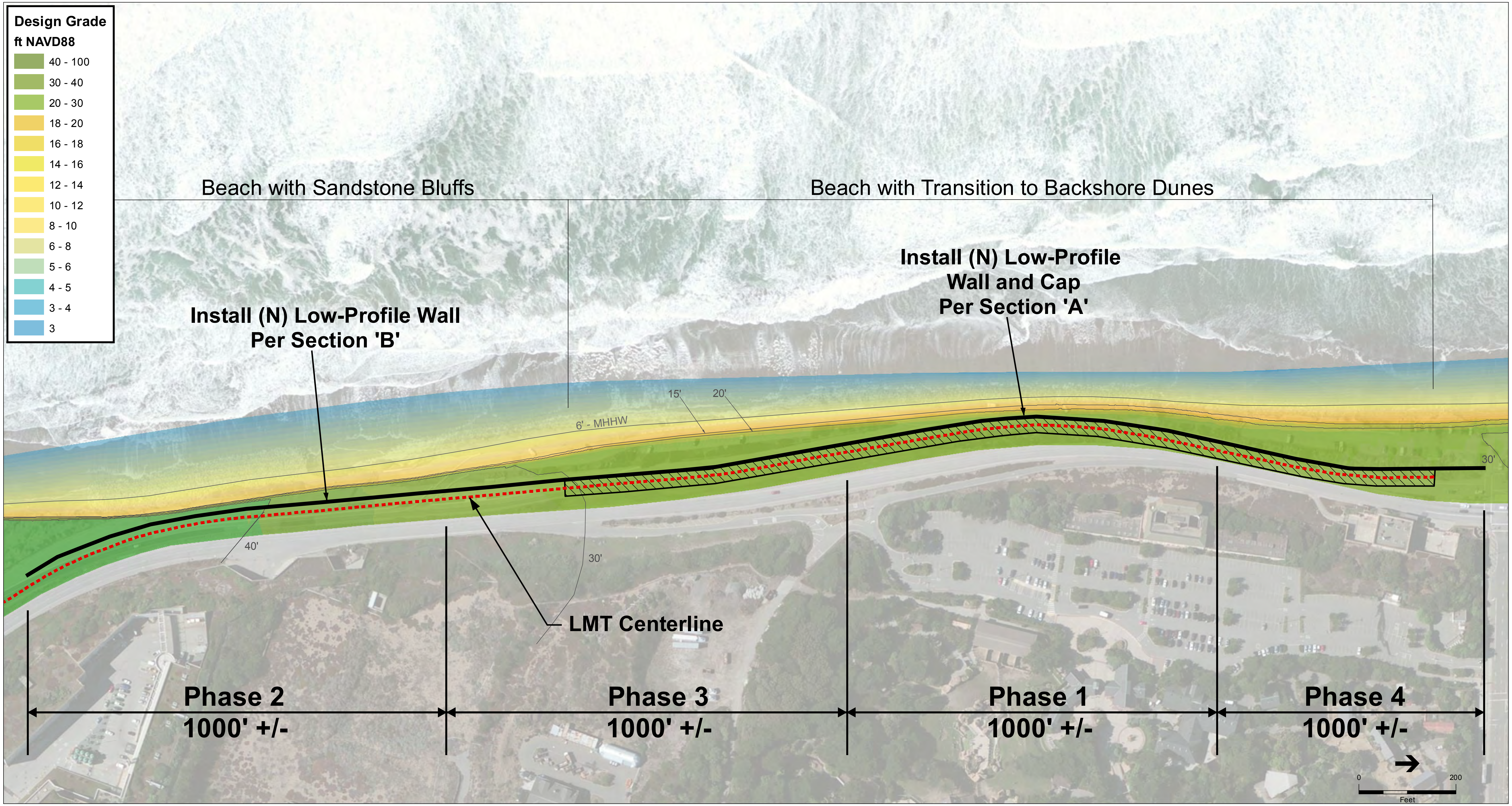
The implementation of the low-profile structure and sand placement should be phased due to the varying levels of risk to the LMT along the shore, and the flexibility of the project concept. For instance, portions of the LMT are becoming threatened in Reach 3 and the EQR Reach as the erosion process continues to degrade the existing bluff, and measures to protect these reaches may become necessary in the immediate future.

The preferred project concept presented here represents Phase 1 (EQR and Reach 3) and Phase 2 (Reaches 1 and 2) of an adaptive coastal management program, and includes:

- Installation of structural protection of the LMT and removal of rock and rubble from the beach
- Initial surface restoration actions, including ecological and access improvements
- Other additional ongoing or long-term management approaches, such as periodic beach nourishment.

The phasing of the project (see Section 6.4) is related to the hazard, and implementation of each phase is based on triggering conditions evaluated through monitoring (see Section 6.6.5 Monitoring). Implementation of the phases can be conducted separately, or phases can be combined if additional benefits related to construction costs and funding, permitting, and environmental reviews are identified.

The fully implemented, long-term project is illustrated by Figures 4 (Cross Section), 5 (Axon View), and 6 (Plan View). These are visionary figures which portray with-project site conditions with full restoration and landscaping at a conceptual level of design detail. The LMT protective structure is below grade, all rubble and rock armor has been removed, and the shore is primarily natural open space with access and interpretive amenities.




SOURCE:
ESA 2014
ESRI Imagery

Ocean Beach Master Plan . D1209025.00

**Figure 6**
Ocean Beach Master Plan
Vision

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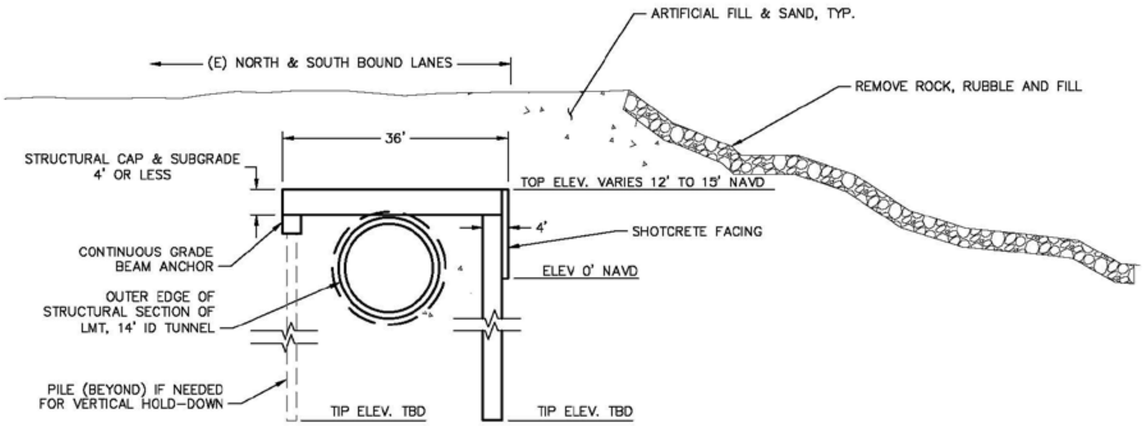



**Figure 7**
Typical Section "A" – Low-Profile Protection of LMT

## 6.2 Structural Protection of the LMT

The LMT protective device is a reinforced concrete structure with vertical and horizontal elements (see Figure 7). The design allows for the vertical section to be constructed first, without excavation except directly above the structure. The vertical structure protects the LMT from wave forces and provides lateral confinement for the tunnel wall. The horizontal element provides a similar function as the vertical element, but also resists buoyancy forces, essentially holding the pipe down. The horizontal portion also serves as a lateral constraint to the vertical section (tie-back), with load transferred to the soil at the landward grade beam. Vertical piling deadmen[4] can be used instead of the grade beam[5], as used for the Taraval Wall (Figure 8), and there are several other structural options discussed later. Excavation of the overburden is required to construct the horizontal section, and to apply an aesthetic facing onto the vertical section.

4 Tie-back anchors attached to the wall or structure typically with a rod or cable and placed in the soil beyond the potential failure plane in the soil.

5 Grade beam footing is a component of a foundation that consists of a reinforced concrete beam that transmits the load from a bearing wall into the ground directly or via additional elements such as piling.

**Figure 8**
Taraval Seawall in Fall 2011 (left; © Elena Vandebroek)
and Winter 1998 (Right; © Bob Battalio)

A different structural section is proposed for the southern section, where the shore is composed of the Colma formation and the bluff reaches a higher elevation. This second typical section is shown in Figure 9, and has a taller vertical element and no horizontal element. Figure 10 shows the plan view alignment and the location of the transition between the typical sections. This would be the typical structural configuration for implementation of the Phase 2 (Reaches 1 and 2) project.

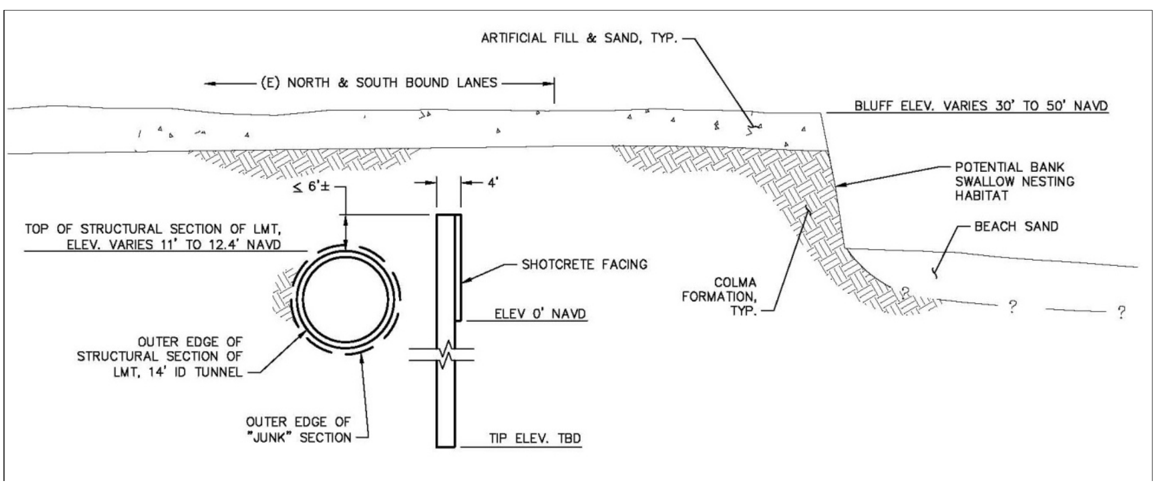
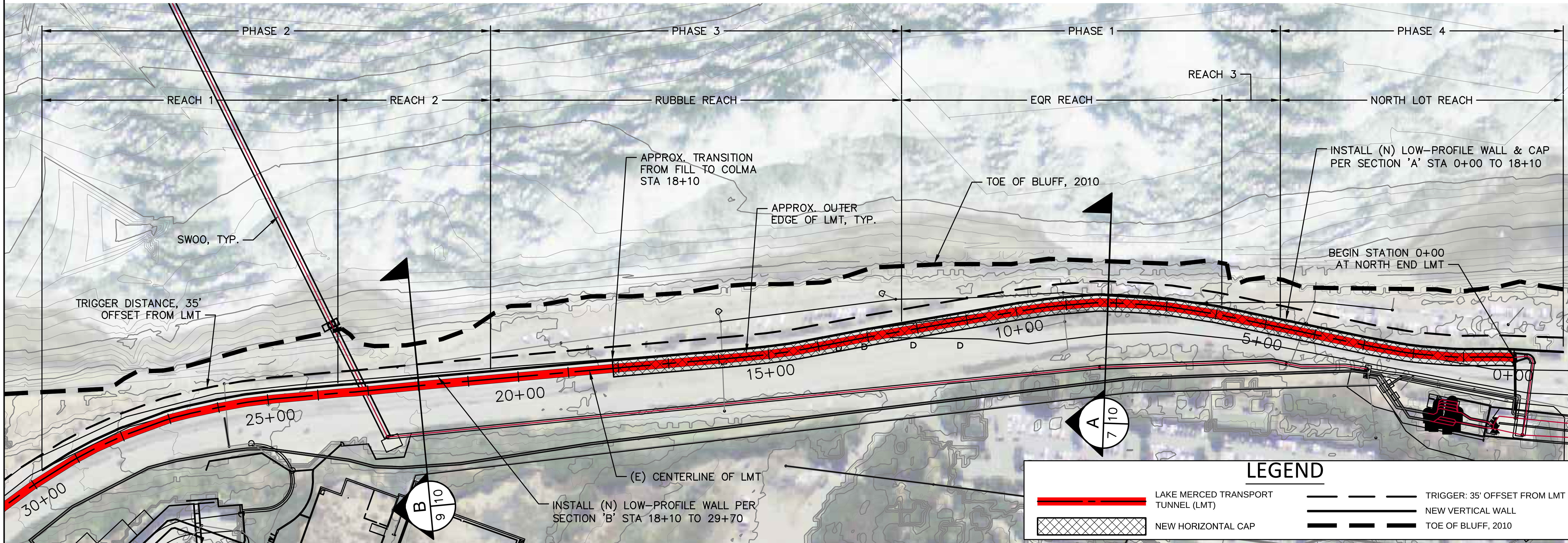



**Figure 9**
Typical Section "B" – Low-Profile Protection of LMT




0 50 100 200
Feet

SUMMARY OF PHASES AND EXTENTS OF CONSTRUCTION

| PHASE | REACHES INCLUDED | APPROXIMATE LENGTH OF PHASE | STATIONING OF NEW VERTICAL WALL | STATIONING OF NEW HORIZONTAL CAP | CROWN ELEVATION OF OUTSIDE OF LMT (FEET NAVD) | REMAINING BLUFF WIDTH SEAWARD OF TRIGGER DISTANCE (NOTE 1) | ESTIMATED TIME UNTIL TRIGGERED (NOTE 2) |
|---|---|---|---|---|---|---|---|
| 1 | EQR & REACH 3 | 800 FEET | 4+70 TO 12+30 | 4+70 TO 12+30 | 9.0 - 10.1 | 24 FEET | 8 TO 24 YEARS |
| 2 | REACHES 1 & 2 | 800 FEET | 20+55 TO 29+70 | -- | 11.2 - 12.3 | 40 FEET | 13 TO 40 YEARS |
| 3 | RUBBLE REACH | 800 FEET | 12+30 TO 20+55 | 12+30 TO 18+10 | 10.1 - 11.2 | 81 FEET | 13 TO 40 YEARS (NOTE 3) |
| 4 | NORTH LOT REACH | 650 FEET | -1+00 TO 4+70 | 0+00 TO 4+70 | 8.5 - 9.0 | 44 FEET | 15 TO 50 YEARS (NOTE 4) |

NOTE 1: REMAINING BLUFF CALCULATED AS DISTANCE FROM TRIGGER TO THE BLUFF TOE, BASED ON USGS BEACH SURVEY FROM JANUARY 2010. VARYING WIDTHS; NARROWEST MEASUREMENT UTILIZED FOR CONSERVATIVE CALCULATION. THE BLUFF TOE THROUGHOUT THE PROJECT AREA FOR 2015 IS FARTHER SEAWARD DUE TO SAND PLACEMENT IN 2012 AND 2014-2015.

NOTE 2: ESTIMATE OF TIME FOR BLUFF TOE TO REACH TRIGGER FOR REQUIRED ACTION IS BASED ON A RANGE OF EROSION RATES FOR UNARMORED BLUFFS OF 1 TO 3 FEET PER YEAR. BECAUSE MOST OF THE BLUFF IS PARTIALLY OR FULLY ARMORED, THE ESTIMATE IS CONSERVATIVE. HOWEVER, UNARMORED BLUFFS OR SAND DUNES (VS. COLMA) COULD ERODE APPROXIMATELY 25 FEET IN AN EXTREME EVENT.

NOTE 3: FOR PHASE 3, TIME WAS REDUCED TO ACCOUNT FOR RUBBLE REMOVAL, WITH SHORELINE ADJUSTMENT OF 40 FEET.

NOTE 4: FOR PHASE 4, TIME WAS INCREASED TO ACCOUNT FOR THE LOWER CROWN ELEVATION OF THE LMT RELATIVE TO BEACH ELEVATIONS.

SOURCE: Existing Grade from USGS; Sewer Infrastructure & Image from City of San Francisco

ESA



**Figure 10**
Plan View of Proposed Protection

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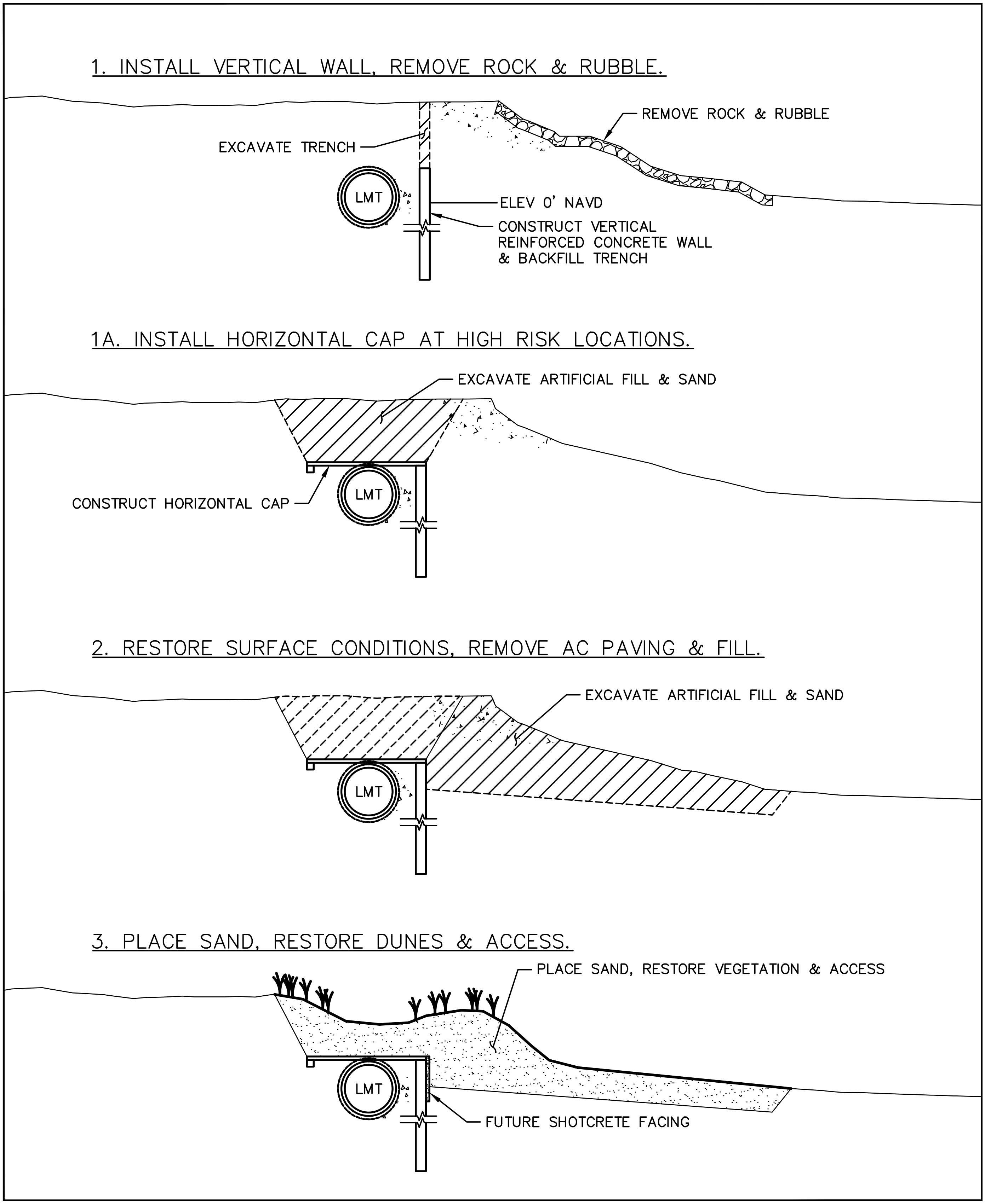
## 6.3 Sequencing

Each phase of implementation will be installed in a defined sequence of actions to construct the structural protection and surface restoration. The structural protection concept is intended to facilitate removal of rock and rubble and allow the ongoing natural erosion of the shore. The general construction sequence is shown schematically in Figures 11 and 12 for Typical Sections A and B, respectively, and consists of the following:

1. Construct LMT vertical protection element
   - For high risk area (see *Phasing*, below), construct horizontal element
2. Removal of armoring (rock revetment, sand bags), rubble, pavement, drainage system, loose fill;
3. Place sand to form linear dune and beach;
4. Install landscaping including planting, access trails (vertical and horizontal) and other open space elements.

It is important to include restoration of the shore (rubble removal, etc.) with the wall construction in order to achieve a balanced project appropriate for the setting, unlike the existing degraded condition. Steps two through four should therefore be closely coupled with step one, and would likely be a condition of regulatory approval.



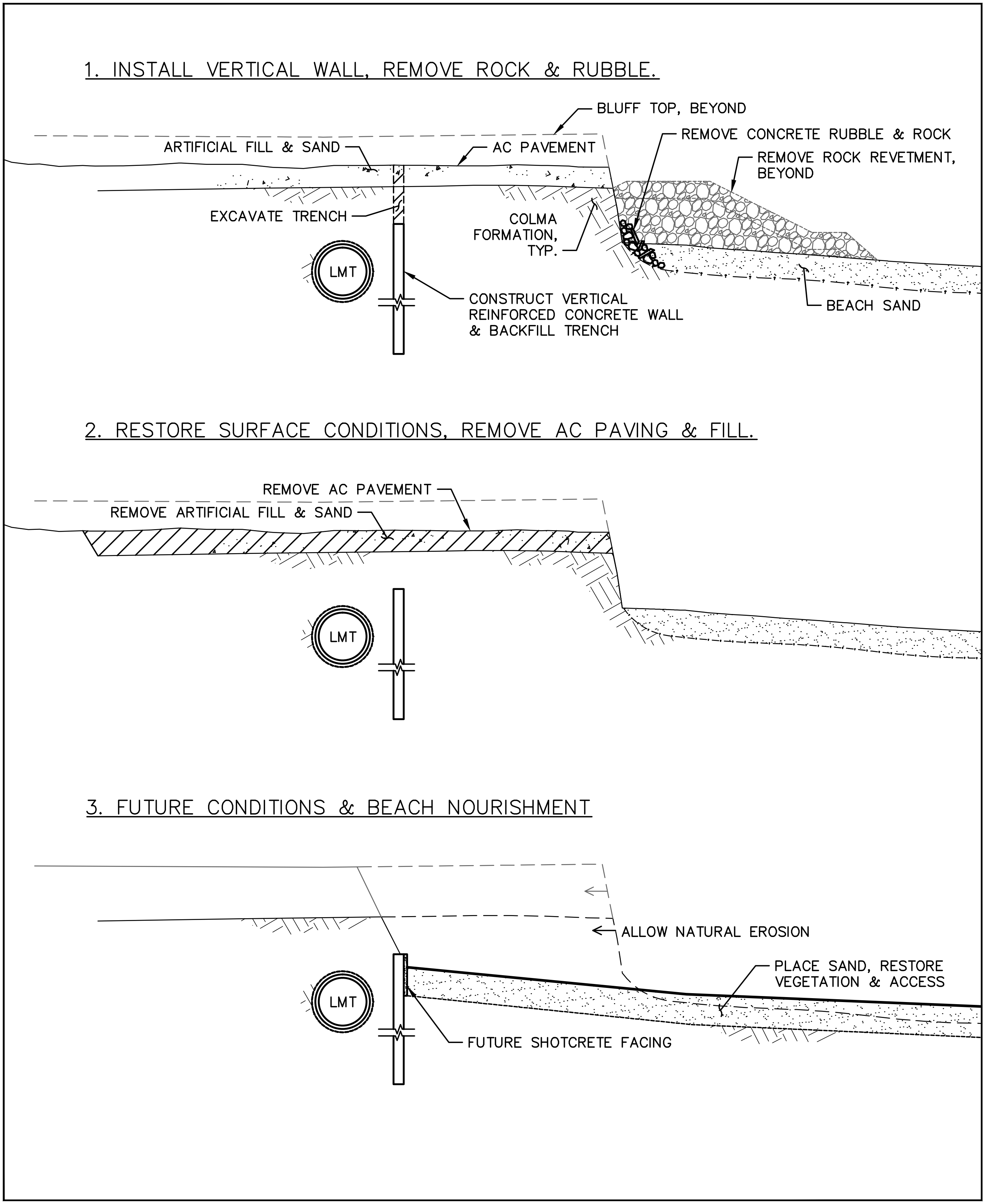

SOURCE:

ESA

Ocean Beach Master Plan . D120925.00

**Figure 11**

Sequence for Typical Section A
Low-Profile Protection of LMT




SOURCE:

ESA



**Figure 12**
Sequence for Typical Section B
Low-Profile Protection of LMT

## 6.4 Phasing

The conditions along the SOB shore vary with location, partly because of varying geography (geology, geometry and conditions), but also due to the location of the LMT relative to the exposed shore face. A phased project implementation is proposed so that the wall is constructed only where needed and other project elements (e.g. rubble removal) can be completed in a similar time frame. The construction will be accomplished first for the shore reaches exposed to the worst hazard or subject to regulatory concerns due to armoring and degraded conditions. Subsequent phases will be constructed when necessary based on actual erosion. Phasing is discussed here to clarify priority areas, and to inform the management team. While phasing is recommended, there may be benefits to more accelerated and comprehensive implementation which can be evaluated during design and environmental review.

The LMT is most vulnerable in the EQR Reach and Reach 3 (location of 2010 sandbag revetment). Along limited sections of these reaches, the bluff toe has retreated to approximately 60 feet from the edge of the LMT. The beach in this area is significantly degraded by the seaward extent of the rock armor. Therefore, Phase 1 should be constructed at the EQR and Reach 3 section of shore. Construction would include the LMT armoring and the backshore restoration (rubble removal and sand placement). Phase 1 would be followed by construction of the subsequent phases as listed below:

- Phase 2: The southern areas, including Reaches 1 and 2;
- Phase 3: The rubble reach; and,
- Phase 4: North Lot Reach.

Note that there are multiple phasing options subsequent to the initial recommended Phase 1. As stated in 6.1, the Preferred Project entails construction of Phases 1 and 2 as soon as practicable.

### 6.4.1 Phasing Considerations

Phasing of the project may be affected by the following factors:

- Imminent hazard of erosion impacts to the LMT will accelerate the need for action;
- Beach conditions and degraded state of habitat is an existing driver for action;
- Permitting and regulatory factors require modification of existing conditions, and also affect lead time activities and construction windows;
- Capital planning and resources availability are important considerations for implementation of construction projects : and,
- Other activities, such as sand placement by the US Army Corps of Engineers will affect conditions and access.

## 6.5 Structural Considerations & Materials

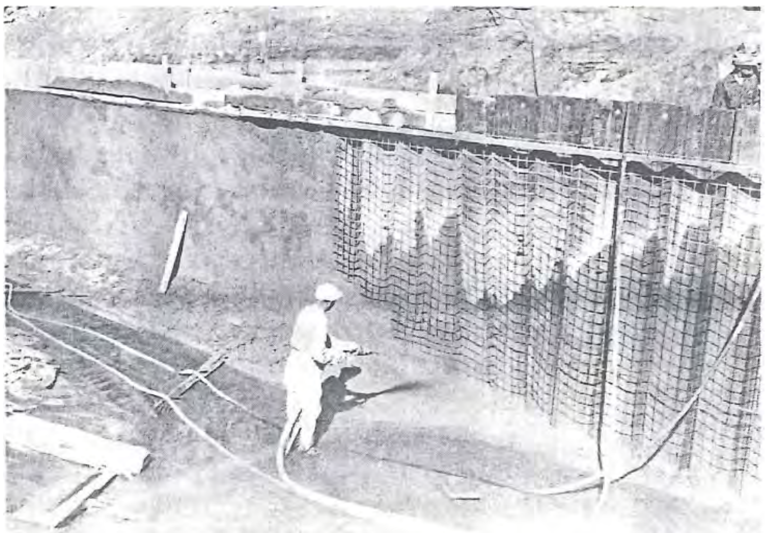
The proposed concept for the structural protection of the LMT is a low-profile configuration that resembles an existing seawall in the vicinity of Taraval Street (called the “Taraval Seawall” – See below case study). At SOB, structural protection consists of a low-profile wall seaward of the tunnel and a cap over the tunnel that provides the required hold down within six feet vertically of the LMT. The cap provides protection against wave-induced scour and provides a better walking surface than other structures (similar to an existing wall on the beach at the end of Taraval Street; see Figure 8). The cap can also provide structural hold-down of the tunnel that is equivalent to the required overburden forces, thereby lowering the top of the structure to its minimum feasible elevation, resulting in a low-profile finished surface. This is consistent with the long-term recommendations of the OBMP.

The following sections describe alternate construction materials and methods to construct the LMT protection. The actual dimensions of the low-profile structure will be defined during subsequent stages of the design process.

***Case Study: The Taraval Seawall Low-Profile Shore Armoring at Ocean Beach, San Francisco***

The low-profile armoring configuration is innovative but it has been successfully implemented at Ocean Beach, between Taraval and Santiago Streets. The so-called “Taraval Seawall” was constructed in the 1940s after several prior armoring schemes had failed (Barrigan, 1985a;b). The structure consists of a steel sheet-pile wall with tie-backs, covered with reinforced concrete (Figure 13), and at one time served as the foundation for a pedestrian tunnel under the Great Highway. The wall emerged from the beach in the El Niño winters of 1982-1983, 1997-1998, and 2009-2010, and is intermittently exposed or slightly buried by accreted sand at present (Figures 14 and 15). The wall has not required maintenance over the 70 years it has been in place, locally reduces dune erosion, and does not appear to impede deposition of sand when driven by natural coastal processes. During erosive periods, scour occurs at the face of the wall resulting in direct wave attack, nearshore morphologic responses including rip current formation, and reduced lateral access (Figure 15).

The  proposed LMT structural protection has a structural configuration similar to the Taraval Seawall, constructed in the 1940s and still in place over 70 years later. Here, the structure is under construction, with vertical steel sheet piling being finished with reinforced concrete, prior to burial below the beach.
Source: Berrigan, Paul D., The Taraval Vertical Seawall, Shore & Beach, January, 1985.



A close up photograph of a reinforced concrete facing being applied by spraying concrete on a steel reinforcing mesh arranged on the surface of the steel sheets. This application technique is often called “shotcrete” and is still frequently used for seawall and retaining wall construction. The term “gunnite” was used historically, but now connotes a cheaper and less strong version of shotcrete.
Source: Berrigan, Paul D., The Taraval Vertical Seawall, Shore & Beach, January, 1985.

The Taraval Seawall circa winter 1982-83, with the Betty-L barge grounded in the background and the headwall for the access tunnel in the foreground. The barge was installing the South West Ocean Outfall (SWOO) when its moorings gave way during the 1983 El Nino storms. Pictured standing on the horizontal slab component of the wall structure  are UC Berkeley Professor Joseph Johnson and construction instructor Sanford.
Photograph: Robert L. Wiegel, 1983.

SOURCE:



**Figure 13**
Taraval Sea Wall
Construction in 1941, conditions in 1983

South end of the Taraval Seawall, near the end of Santiago Street. Photograph: Bob Battalio.

North end of the Taraval Seawall, near the end of Santiago Street, June 2010.
Photograph: Bob Battalio.

SOURCE:



**Figure 14**
Taraval Sea Wall
Conditions in 1990s and 2010

South end of the Taraval Seawall, near the end of Santiago Street, circa late 1990s, eroded conditions eroded conditions following the 1997-98 el nino winter. Photograph: Bob Battalio.

North end of the Taraval Seawall, near the end of Santiago Street. Photograph: Bob Battalio.



**Figure 15**
Taraval Sea Wall
Eroded Conditions Post 1997-98 El Nino winter, and in 1990s

### 6.5.1 Pile Wall

This alternative includes a pile wall constructed by drilling cast-in-drilled-hole (CIDH) piles from the top of the bluff and located in front of the tunnel (Figure 16). The piles could be constructed up to an elevation that provides minimum vertical and horizontal cover over the tunnel, and could be modified to include construction of a cap in the future.

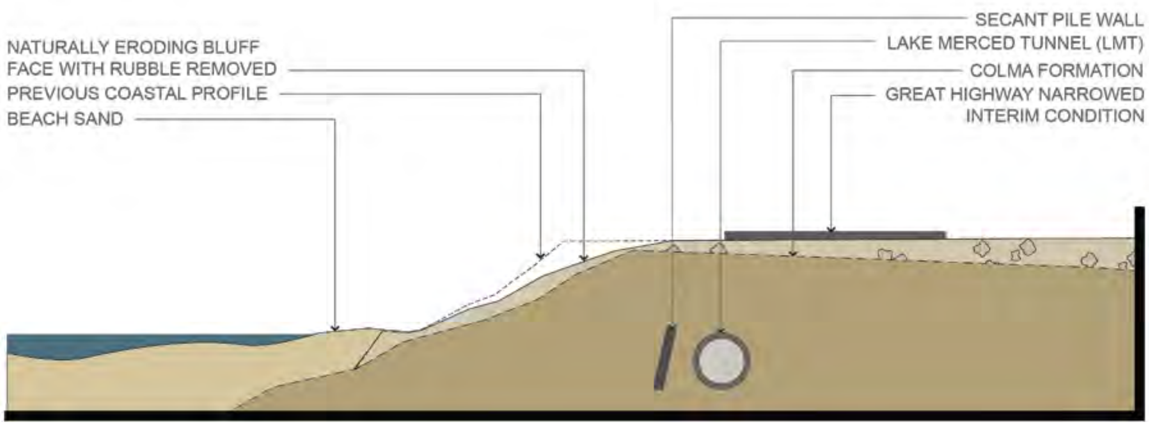



**Figure 16**
Conceptual Schematic of Secant Pile Wall Measure

The benefits of the pile wall are many – it has a small footprint and can be designed to specific structural criteria; it can be constructed from the existing ground surface and built up to an elevation near the crown of the tunnel; it uses conventional construction methods that can be performed by a wide variety of contractors. Since construction would occur on CCSF property outside of the beach and surf, this alternative may be easier to permit, and would not be susceptible to seasonal construction restrictions.

Construction of the pile wall would occur as close to the LMT as possible, preferably within 10 feet of the seaward edge of the LMT. Once constructed, armoring on the bluff and beach could be removed, and the bluff could be allowed to migrate landward naturally. Figure 17 presents an isometric view of the pile wall concept.

The cost of this protection measure is expected to be quite high, but would facilitate the low-profile structure that is described in the OBMP. In addition, a cap could be constructed over the top of the LMT and would be founded and structurally connected to the vertical wall on the seaward side of the LMT, and on additional piles or grade beam located on the landward side of the LMT. Areas with sufficient cover and natural bluffs could remain in place and would not require a cap until deemed necessary at some point in the future.

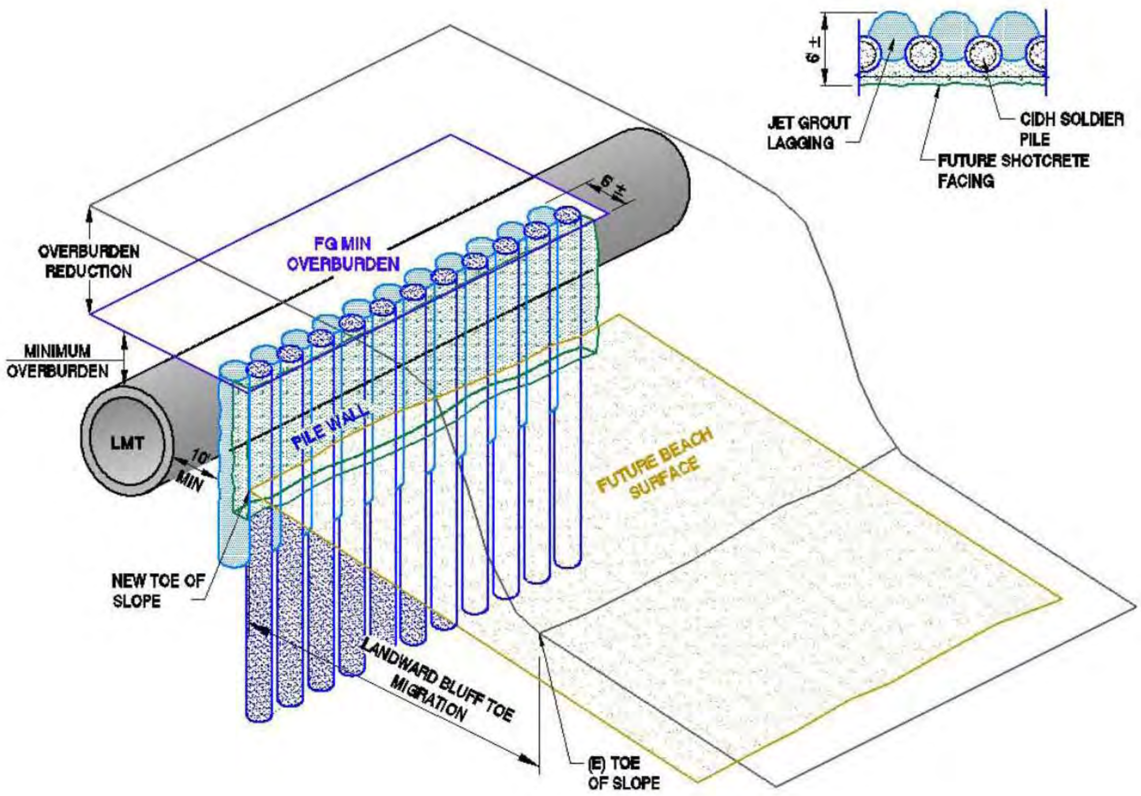



**Figure 17**
Isometric View of Pile Wall Concept

## 6.5.2 Soil Mix Wall

This alternative is a low wall embedded within the bluff constructed by the mixing of cementitious material with the in-situ material to create a low-strength (relative to reinforced concrete structures) soil-cement matrix (Figure 18). The in-situ soil grouting is entails mechanically mixing the existing in-situ material while injecting cementitious grout, resulting in a continuous wall that is constructed prior to erosion.

This wall could be constructed up to a specified height near the crown of the LMT, similar to the secant pile wall. This alternative could be modified in the future to include a cap over the LMT. In this case, the cap would function more as a protective apron than a hold-down because the grout may not provide sufficient structural capacity to resist potentially large buoyant forces of the LMT during special circumstances (e.g. high groundwater and empty tunnel). It may be possible to stabilize the soil above the LMT with grout as well. Figure 19 presents an isometric view of the soil mix wall concept.

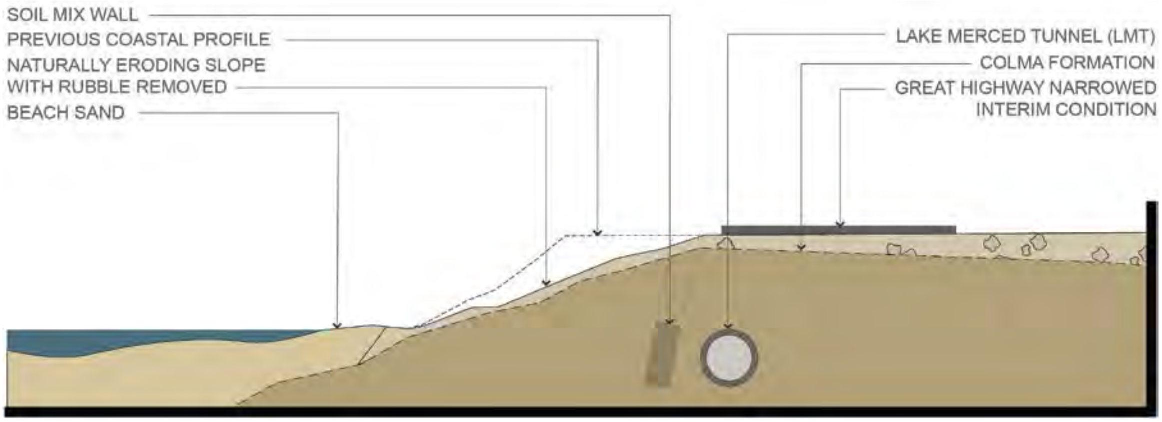



**Figure 18**
Conceptual Schematic of Soil Grout Wall Measure

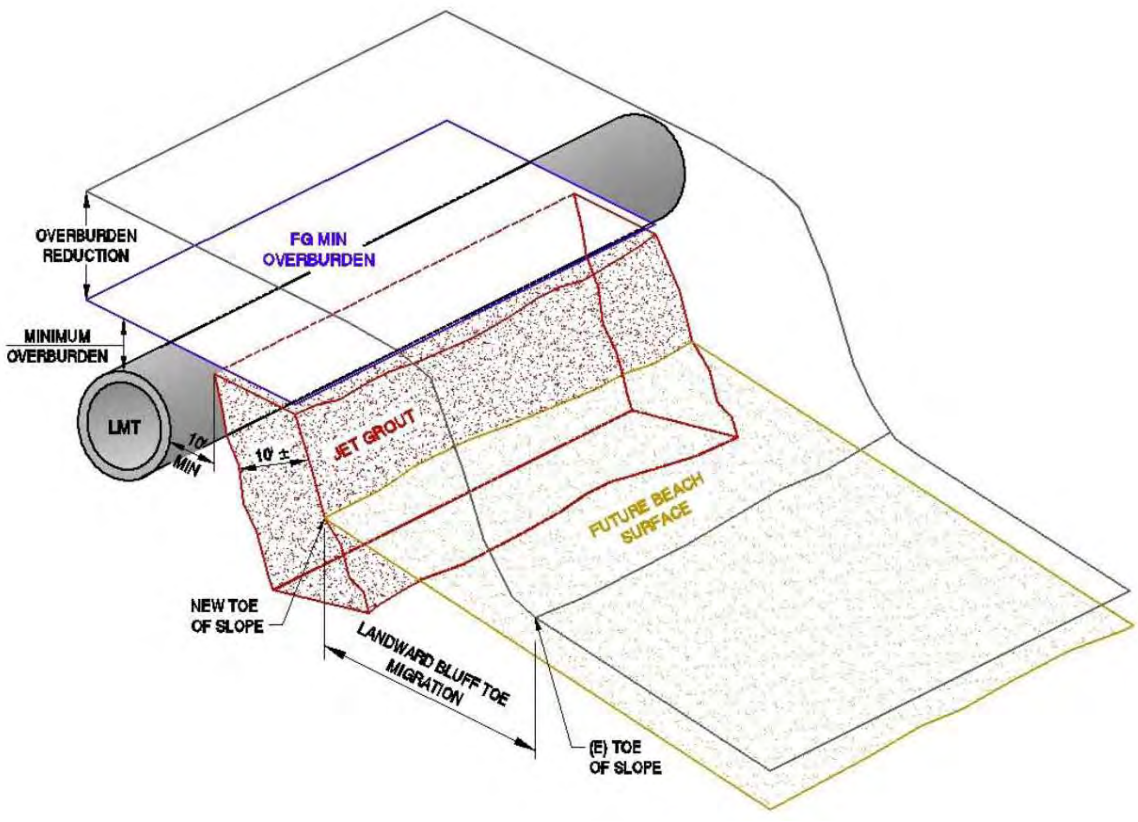



**Figure 19**
Isometric View of Soil Mix Wall

A limitation of the in-situ soil grout wall is that sufficient existing bluff materials between the LMT and the bluff face are required, as the process of injecting the grout could induce bulging at the toe of the bluff and additional geotechnical issues. Similarly, added lateral pressure on the LMT due to the soil grouting process should be minimized. A benefit of this approach is that the appearance of the wall, when exposed, appears more natural than a wall constructed entirely of concrete. The exposed face will be undulating and irregular, with a color similar to the native bluff materials. Compared to the secant pile wall measure described above, the soil grout wall is much cheaper, but would not be as strong and long lasting.

### 6.5.3 Comparison of Materials and Construction Methods

Table 4 presents a comparison of the pile wall and soil mix wall for several categories to evaluate the associated pros and cons and to facilitate selecting the appropriate approach. Based on the comparison of alternative approaches, the pile wall concept is the preferred method for constructing structural protection in the northern half of SOB: from the North Lot Reach through most of the Rubble Reach (see Figure 7, Typical Section A). This area is characterized by imported fill, concrete rubble and rock revetments. A pile wall or a soil mix wall can be used to construct the vertical wall along the southern half of SOB, from the southern portion of the Rubble Reach south through Reach 1 and the 2010 emergency bluff toe protection.

**TABLE 4**
**COMPARISON OF ALTERNATIVE WALL MATERIALS**

| | Pile Wall | Soil Mix Wall |
|---|---|---|
| Performance | • High-strength reinforced concrete wall will be designed to resist loads due to future exposed height, waves, rocks (cobbles), and seismic forces.<br>• The slender wall (3-ft thick) will be cantilevered and will be subjected to high shear and moment loads.<br>• Strength is dependent on reinforcing steel, so it is imperative that steel is protected from corrosion.<br>• A horizontal cap can be constructed that will protect the surface from erosion, and serve as a structural hold-down that will counteract buoyancy forces. | • Low-strength unreinforced concrete will act as a gravity wall.<br>• The thick wall (assumed to be 10-ft to 20-ft thick) will be subjected to high shear forces but low moment loads.<br>• No potential for corrosion due to absence of steel reinforcement.<br>• A horizontal cap cannot be structurally attached to the grout wall due to the absence of reinforcing. |
| Construction Complexity | • May require pre-construction of a drilling template.<br>• Will require casing during drilling, which is typically a full-length steel pipe. Casing will be advanced down the hole during drilling and extracted as concrete is placed in the hole.<br>• Two cranes will be required to construct the piles, plus a concrete pump truck.<br>• Will generate significant spoils (drill cuttings) but may be able to dispose of on-site (as beach nourishment) if material characteristics are acceptable.<br>• Future construction of shotcrete facing may require temporary cofferdam to construct. | • Pre-construction of trench is required to contain return slurry spoils. Return fluid cannot be reused and will be disposed of off-site.<br>• Will generate similar volume of spoils as generated by the thinner pile wall; however, spoils cannot be used in the project.<br>• Requires substantial staging area for on-site batch plant to prepare grout. |
| Appearance (after exposure) | • The face of the pile wall will be vertical, with the face consisting of a series of arcs (scalloped). The concrete is typically gray in color but can be tinted to blend with the bluff by addition of colorant into the wet concrete mix.<br>• Optional future covering with shotcrete facing to improve the aesthetics and resistance to impact. | • The face of the soil mix wall can be vertical (deep soil mixing or DSM) or sloping (jet grout).<br>• The DSM face will be roughly planar; jet grout wall could be highly irregular and undulating. |
| Permit Issues | • Concrete spillage (low probability)<br>• Need for future cast-in-place facing (shotcrete). | • Grout Spillage (low probability)<br>• Larger footprint of completed structure in comparison to pile wall. |
| Initial Cost | • High, possible range of \$5000/lf to \$7500/lf. | • High, possible range of \$4000/lf to \$6000/lf. |
| Future Costs | • Guardrail may need to be constructed once top of wall is exposed.<br>• Shotcrete wall will be constructed to cover exposed wall face. | • Guardrail may need to be constructed once top of wall is exposed |
| Service Life | • Assume minimum 75-year service life.<br>• May be governed by rate of saltwater intrusion into concrete to initiate corrosion of reinforcing steel.<br>• When initially exposed due to bluff landward migration, the wall will be subject to direct wave action. A shotcrete facing is proposed to be constructed once this occurs, to provide protection from wave erosion and to improve the appearance of the wall.<br>• After construction of shotcrete facing, pile wall will not be subjected to damage by wave action | • Assume minimum 75-year service life.<br>• Will be governed by rate of erosion due to wave action. Assume face of soil mix wall will be exposed to wave action 1-year after removal of existing shoreline protection. |
| Public Safety | • Vertical wall may require a guardrail to be constructed once top of wall is exposed (although no such railing is in place at the Taraval Street seawall). | • Near-vertical wall may require a guardrail to be constructed once top of wall is exposed (although no such railing is in place at the Taraval Street seawall). |
| OBMP Compatibility | • Conforms to OBMP.<br>• Allows future cobble berm and reduced cover over LMT.<br>• Minimum footprint of structure. | • Conforms to OBMP.<br>• Allows future cobble berm and reduced cover over LMT.<br>• Larger footprint of structure may not meet OBMP goals of using the “minimum” footprint possible. |

### 6.5.4 Potential Regulatory Framework

Permitting a seawall is likely to entail a significant effort. However, this would be the least environmentally damaging alternative: Reconstruction or structural modification of the LMT itself may have fewer beach impacts but could degrade wet weather sewage treatment performance and associated regulatory complexities and require a longer planning and implementation period. Although the construction of a pile wall would not occur on the beach, we expect the same or similar level of regulatory scrutiny and environmental review associated with beach and bluff-face structures. For example, construction within 300 feet of the bluff is within the zone of appeal by the California Coastal Commission (CCC). Therefore, this measure, and all subsequent measures presented below, will require a Coastal Development Permit and is appealable to the CCC. Including rubble and other armor removal and improved ecology, access and aesthetics, and designing for sea level rise are all important benefits of the integrated plan intended to directly respond to CCC and community concerns.

In addition to acquiring a Coastal Development Permit, the project will require significant regulatory and environmental review process, including permits from the U.S. Army Corps of Engineers, California Department of Fish and Wildlife, and involvement of Federal, State, and Local agencies. Table 5 presents a conservative overview of environmental and regulatory agency approvals or consultations, depending on final project design and specific resources affected. The jurisdiction and permit requirements of environmental and regulatory agencies will depend on the specific project location and the resources potentially affected. A detailed inventory of resources at the project site has yet to be completed. The agencies and requirements described in Table 5 may change once the project is progressed, including finalizing the project location and completing evaluations of resources.

**TABLE 5**
**POTENTIALL AFFECTED AGENCIES, ENVIRONMENTAL REGULATIONS, REQUIREMENTS, AND PROJECT APPLICABILITY**

| Agency | Governing Regulation | Potential Requirements | Applicable Project Component |
|---|---|---|---|
| Federal | | | |
| U.S. National Park Service - Golden Gate National Recreation Area (Potential Lead Federal Agency) | National Environmental Policy Act (NEPA) and National Park Service Act | NEPA Compliance, Special Use Permit | All project activities, if NPS lands are used |
| U.S. Army Corps of Engineers | Clean Water Act | Section 404 Authorization | Project activities below the mean high tide line |
| U.S. Fish and Wildlife Service | Endangered Species Act, Migratory Bird Treaty Act (MBTA) | Section 7 Consultation | Project activities affecting species listed by the federal government as threatened or endangered |
| State | | | |
| California Coastal Commission | California Coastal Act; Coastal Zone Management Act | Coastal Development Permit | Project activities below the mean high tide line |
| State Water Resources Control Board | Clean Water Act | General Construction Permit | Project activities affecting disturbing more than 1 acre of land |
| San Francisco Bay Regional Water Quality Control Board | Clean Water Act | Section 401 Water Quality Certification | Project activities requiring a Section 404 permit |
| California Department of Fish and Wildlife | Fish and Game Code Section 2080 | Incidental Take Permit | Project activities that result in Take of a species listed by the State as threatened or endangered |
| California Office of Historic Preservation | National Historic Preservation Act | Section 106 Consultation | Project activities requiring federal action (e.g., issuing a permit) |
| Local* | | | |
| City and County of San Francisco, Lead Agency, (California) | California Environmental Quality Act (CEQA) | CEQA Compliance | All project activities |
| San Francisco Planning Department | Local Coastal Program (LCP) | Coastal Development Permit | Project activities occurring within San Francisco's LCP jurisdiction |

* Additional non-environmental local regulatory approvals would be required for project implementation. Other local agencies whose approval may be required include SFPUC, SFDPW, SFDPT, and SFRP.

## 6.6 Surface Restoration Elements

This section addresses interrelated actions to improve the beach and bluff conditions for ecology and recreation, as well as the monitoring strategy for the SOB area. Environmental conditions will be improved by removing the armor and the rubble that is located throughout the shore, including two rock revetments, concrete rubble placed as fill and on the beach, and eroded asphalt pavement that has fallen on the beach.

### 6.6.1 Grading and Bluff Removal

As landward bluff migration progresses, the top of the low-profile wall will begin to be exposed. The wall will act to prevent further bluff toe migration, but the upper bluff above the wall will be allowed to continue to erode. As this material erodes and the slope flattens, there is opportunity to grade the area landward of the low profile wall down to a shallower plane. The specific amount of excavation that can be done is dependent upon maintaining the minimum cover over the LMT to prevent buoyancy forces from damaging the LMT.

Removal of artificial elements in the beach and at least 40 feet landward of the bluff edge is recommended to support natural coastal processes and a dynamic, resilient shore. The dimension of 40 feet landward of the bluff edge was identified during the shore modeling for the OBMP. It is based on the seaward protrusion of the existing constructed bluff profile relative to a more natural sand beach and dune profile to the north. This adjustment distance is the estimated initial response to removal of rubble and other non-native materials and placement of sand. This will require excavation and off-haul of a large volume of material (Figures 11, 20 & 21).

The extent of existing rubble in this area is not well documented, and therefore the actual volume of rubble versus sand is not known. In its existing configuration, the rubble holds the bluff farther seaward than a sandy dune and native bluff system, which results in a lower and narrower beach. Alternatively, the rubble could be removed from the beach and bluff as needed over time, but this would likely result in an increased overall cost and reduced performance relative to project objectives. Removing some of the rubble will allow a similar rubble bluff and beach to form somewhat landward, which would be an incremental improvement but not a complete restoration. A bluff partially armored by remaining rubble will also have a lower sand storage and storm-response volume, increasing the sand placement needs to achieve an appropriate beach width (see Appendix 2 for description of modeled changes to the beach width over time). Note also that the proposed low-profile protection in this location includes a cap that requires excavation of the area anyway.

**Figure 20**
Photographs of rubble on back beach and in fill (© Bob Battalio)

**Figure 21**
Photographs of concrete rubble on beach and in fill at Reach 2, South Ocean Beach. From left to right: June 2012 (Left); March 2014 (Middle); July 2014 (Right) (© Bob Battalio)

Native beach sand or similar should be placed to restore the bluff grades, or otherwise comply with the open space plan. This backshore restoration is a key element of shore enhancement, as learned in other projects such as Surfers Point (Ventura, CA) and Pacifica State Beach (Pacifica, CA).[6] The backshore restoration provides space for a more gradual and natural dissipation of wave power, which reduces wave reflection and results in a higher and wider beach. The placement of beach sands in the bluff also provides a sand source that directly and immediately counters beach erosion, dissipating energy by moving sand, which in turn "feeds" the beach.

These types of sand placements have proven effective over the short term, with three placements in the early 2000s (Figure 22), one in 2012 (Figure 23) and another in 2014. Realigning the beach farther landward increases the volume of sand that can be stored in the back shore, thereby increasing the buffering capacity during a rough winter, when it is needed and construction is not practical.

6 Description of managed retreat projects at Ventura and Pacifica State Beach: http://coastalmanagement.noaa.gov/initiatives/shoreline_ppr_retreat.html

**Figure 22**
Photographs of Sand Placement in 1999-2000 at Reaches 1 and 2, between the South Parking Lot and the SWOO (© Bob Battalio)

**Figure 23**
Photos of sand embankment and wind-blown sand transport on April 30, 2013, after the 2012 sand backpass project (© Bob Battalio)

### 6.6.2 Removal of Revetments

The removal of the existing revetments is a basic objective of the recommended project. This would have two primary physical results: First, it would restore the beach surface to a more natural state. Second, it would allow natural erosive processes to occur. Although unsightly and dangerous to beach users, the rock revetments and exposed rubble have limited bluff erosion for decades, and it must be noted that the unprotected bluff will undergo erosion once the revetments are removed.

The removal of the revetments would also help resolve important regulatory challenges. Because they do not have Coastal Development Permits for permanent structures, the constructed revetments are not permitted beyond the duration of the emergencies they were meant to address. The EQR constructed in 1997 and 1999[7] has degraded and is expected to degrade further. It was constructed under emergency conditions and is not presently in compliance with the Coastal Act. The 2010 emergency bluff toe protection was designed to protect the road and the LMT. Because

[7] EQR was initially constructed in 1997 and augmented in 1999.

the southbound lanes are in process of closure, the structure is only needed to protect the LMT. A much smaller structure could provide adequate protection to the LMT, and hence, some rock could be removed immediately. Construction of a wall would allow complete removal of the rock revetment, which would have an immediate beneficial effect of exposing beach area.

### 6.6.3 Rubble Removal

The extent of rubble along the shoreline south of Sloat Boulevard is not accurately known. During a low-beach condition in May-June 2012, the rubble was observed to extend far out into the ocean on a relatively consistent slope. As the beach rose back to normal elevations, the rubble was again concealed from sight. This episode is instructive when discussing plans to remove the existing rubble, because the effort involved in this undertaking is largely dependent upon the conditions present at the time. If the beach is at normal elevations, only a small fraction of the rubble is accessible, and the lower levels of rubble (typically hidden) will possibly migrate upslope and become visible. If the beach is at a low point, a large percentage of the rubble can be removed and the restoration of the beach will be far more complete.

Removal of rubble for public safety, access, ecology and aesthetics is recommended. Exposed rubble can be removed with land based equipment when dry beach and exposed rubble exist. Rubble at lower elevations will require additional construction effort, either via a long-reach from the bluff top or by constructing a platform along the beach using the rubble itself as it is removed. Both methods are feasible. The rubble will likely be off hauled and disposed. Ideally it will be recycled for reuse in concrete, road base or trails. It is possible that some of the rubble could be reused on site.

While rubble placement was widespread in the early to mid-1900s, it is currently not permitted by the CCC and the San Francisco Bay Conservation and Development Commission (BCDC – SF Bay CZM agency).

### 6.6.4 Pilot Studies: Dynamic Cobble Revetment & Wind Blown Sand Mitigation

The OBMP identifies several erosion mitigation and open space elements that have promise in terms of helping to achieve project objectives, but are innovative and hence require special attention. Pilot studies are recommended to help assess feasibility and performance prior to full implementation. Pilot studies would consist of limited construction that would be monitored and evaluated, and could be removed.

The three primary pilot studies proposed for SOB are:

- Cobble revetment,
- Wind-blown sand mitigation, and
- Native dune restoration

The Cobble Revetment was described in the OBMP. It consists of rounded rock, larger than gravel and approaching boulder sizes on the back beach at and below typical beach levels. The structure is also known as a "dynamic revetment" because the individual rocks are moved by waves and the overall rock mass geometry is malleable. The cobble dynamic revetment simulates natural lag deposits that exist along many California beaches and are exposed only when the beach is eroded in severe winters. Cobble berms are naturally exposed at river and creek mouths. The benefits of the cobble revetment is wave dissipation, backshore erosion prevention, resilience with sea level rise and erosion, and a transitional slope between a vertical bluff and flat beach.

Wind-blown sand mitigation consists of a suite of measures that limit inland migration of sand from the dune nourishments. Placement of dead vegetation and surface roughening elements can modify the near-surface wind characteristics, limiting sand entrainment and transport. Straw-like stalks are partly embedded in the sand surface by hand or equipment in a process called "straw punching" and larger tree segments can also be used. Another element is planting with native dune plants, which are naturally able to thrive in dunes partly by way of their stabilizing influence on sand transport that allows a more sustainable growing environment. Also, coarser sediments, which are not easily entrained by wind, can be used for nourishment: This approach was used in the dune placements in the early 2000s and wind-blown sand was not problematic (See Figure 22 and Appendix 1). Coarser materials, including sand or shell lag can also be used as a surface treatment. Finally, visitors should be channeled into designated beach access points to limit disruptions of surface treatments.

### 6.6.5 Monitoring

Monitoring is necessary to ascertain when erosion has progressed to the point that action is needed. Monitoring of shore change is also needed to better understand how the shore responds to interventions, so that environmental reviews and designs are well informed and future interventions are more likely to be successful.

In addition to monitoring that is required as part of standard regulatory procedures, there are several other groups that collect data at Ocean Beach for research and management purposes, such as the U.S. Geological Survey, San Francisco State University, and the National Park Service. Integration of monitoring efforts with these groups can yield more complete data sets that will help to develop understanding of the physical processes and shoreline response to interventions. Hence, the monitoring plan could be formulated to complement other efforts and provide coordinated results. This will provide the lowest cost and most effective results, by leveraging the capabilities of scientific and academic institutions to help address specific project needs.

The key data that are currently being collected at Ocean Beach are beach elevation and nearshore bathymetry. Topographic surveys are conducted several times per year. These surveys are used to create topographic maps, which are then compared to discern changes. The maps and changes are used to establish an understanding of the seasonal fluctuations of the beach, the overall movement of sediments in the sand budget, and interactions of wave energy and beach morphology. USGS researchers collect beach elevations over the beach face along transects, and also collect the backshore elevations along the bluff toe. However, the degraded condition of the back beach with concrete rubble and rock often prevents collection of the data due to lack of access.

Researchers from San Francisco State University periodically collect elevation transects of the beach near Sloat Boulevard, use kite-based photogrammetric methods to map surface conditions,, and gather information on wave caustics using radar technologies. Undergraduate students collect transects of the beach and dunes annually as part of coastal geology curriculum. Other special monitoring has occurred, including use of land-based radar to analyze nearshore wave patterns.

The National Park Service collects biological information at Ocean Beach, primarily related to birds. Shorebird counts are periodically conducted to understand the locations of the beach with highest use, as well as the seasonality, number of birds, and the effects of human and dog activity on the birds. Bank Swallow nests are found in the Colma bluffs above the emergency 2010 revetment in Reach 1. The nests are made from burrows in the bluffs and are located strategically to minimize disturbance by predators and to be located close to Lake Merced where the Bank Swallows feed.

The long-term monitoring program should include the coordination elements, regular survey locations and control points that can be reoccupied, and collaboration between efforts. The collected data should be used to help the managers of South Ocean Beach understand when and where interventions should take place, as well as the anticipated effects of the interventions on the beach.

The next step is to develop a monitoring plan. Discussions with the TAC have provided useful input toward future monitoring plan development. The monitoring plan could include:

- Reporting:
  - The Monitoring Plan: A report that describes the monitoring and analysis.
  - Reports: Annual or as otherwise determined.
  - Data Archiving: Data, reduced data and metadata.
- Surveying of grades in South Ocean Beach:
  - Survey control
  - Topography
  - Nearshore Bathymetry (underwater topography)
  - Maps of elevation changes
  - Sand volume changes
- Photography:
  - Re-photography: Repeated photographs from same land vantage
  - Aerial
  - Digital time lapse
- Environmental Conditions:
  - Publically available (ocean waves and water levels, winds, etc.)
  - Surf observations (breaking patterns, currents, sediment sizes, etc.)

- Ecological Conditions:
  - Species use
  - Habitat metrics
- Other data:
  - Events (e.g. sand placement)
  - Public use
  - Other
- Assessments:
  - Geomorphic changes and implications
  - Effectiveness of shore management actions
  - Validation of modeling predictions
  - Need for management action(s).

### 6.6.6 Large-Scale Beach Nourishment

Large-scale beach nourishment is recommended as an additional management element of the preferred project concept. Maintaining a sandy beach over time is an important objective of the OBMP, and would greatly improve the ecological and recreational value of the site. A wider beach also reduces the landward extent of wave runup and coastal hazards.

Sand for beach nourishment is available offshore of Ocean Beach. Maintenance dredging of the San Francisco Bar Main Ship Channel occurs every one to three years, and the rate averages about 300,000 cubic yards per year (Battalio 2014). The timing of dredging is also affected by other factors such as funding and equipment availability. There are indications of dredged sand sizes becoming increasingly smaller, but the sand is considered large enough for direct placement on the beach and surf zone (Battalio 2014). In addition, sand has accumulated in other areas, such as the offshore disposal site and intermediate waters and these areas could be sources of sand for beach nourishment.

The OBMP vision relies on an ongoing large-scale beach nourishment program over the next 50 years and beyond that will be needed to maintain a sandy beach after the effects of sea-level rise are realized. The OBMP called for placing approximately 2 MCY every 10 to 30 years: about 1.5 MCY would be placed at MOB and about 0.5 MCY would be placed at SOB. However, based on recent modeling by the USACE, sand placed at SOB may not persist as long as previously estimated, and an increased nourishment rate of 0.5 to 1.0 MCY every 10 to 20 years is recommended (see Appendix 1). The effectiveness of sand placement at SOB is not known and success criteria are not well defined. All sand placement activities at SOB should be monitored to collect information of the persistence and fate of sand over time that could be used to calibrate and validate existing numerical models for the site.

## 6.7 Other Adaptation Options for Coastal Protection

The following three alternative adaptation approaches are presented as potential protection measures that are discussed but were identified as inconsistent with OBMP principles and objectives or not considered feasible for regulatory and cost implications.

### 6.7.1 Toe Wall

This alternative is a soldier pile and lagging wall at the seaward edge of bluff (i.e. “toe wall”) that would hold the toe of the bluff and maintain a slope that provides at least the minimum vertical and horizontal cover over the tunnel (Figure 24). This wall would be located farther seaward than a pile wall, but would be removed as erosion progressed in favor of another landward wall or other approach. The toe wall represents an interim approach to limiting erosion of the bluff and thereby reduces the vulnerability of the LMT. This alternative would be an interim protection measure that could be maintained into the long-term horizon. The presence of rubble in the subgrade could create a difficulty for constructing the toe wall. Rubble on the beach and throughout the reaches is several feet thick. Construction of the toe wall would occur on the beach and would be affected by waves. This measure would be difficult to construct due to limitations on seasonal constructability, restricted habitat zones, and permitting effort. It could also present access and safety issues.

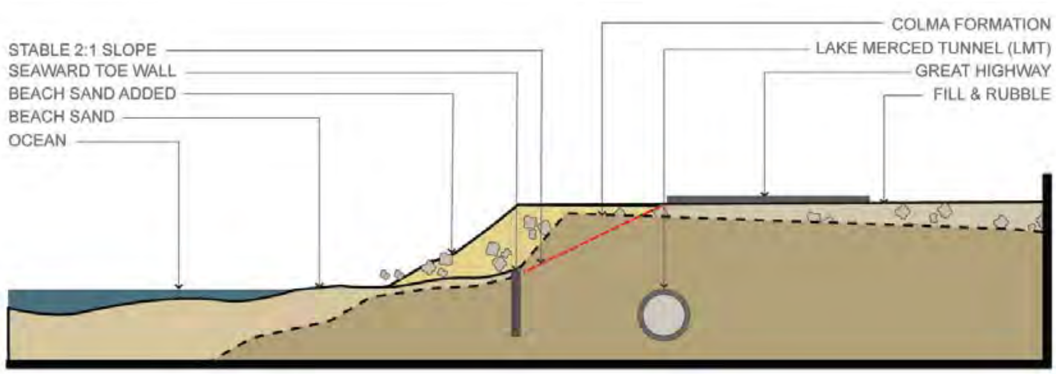



**Figure 24**
Conceptual Schematic of Toe Wall Measure

### 6.7.2 Structural Modification of the LMT

Internal, or structural, strengthening to modify the cross section of the tunnel to withstand reduced vertical and horizontal cover is another possible approach. However, implications to the storage and LMT functions are considered a fatal flaw of this alternative by the SFPUC. The LMT functions include dry weather conveyance of wastewater flows to the OTP and wet weather conditions during which the tunnel provides up to 10 million gallons of combined sewer and stormwater storage. Reduction of the cross section of the tunnel would reduce the volume of the system, and therefore would violate regulatory requirements for its operation and function, and could increase the number of combined sewage-stormwater overflow events that occur typically.

This concept has not been evaluated in detail and is not being considered as an alternative at this point.

In principle, the volume lost from structural modification of the LMT could be relocated to an alternative location. Assuming that the structural modification along 3,000 linear feet of the LMT south from Sloat Boulevard increases the structural section of the tunnel wall by 2 feet, and hence reduces the inside diameter of the tunnel from 14 feet to 10 feet, the static storage volume reduction is approximately 1.7 million gallons, which is about 5 acre-feet. Consideration of the dynamic storage volume and overall treatment system impact is beyond the scope of this study.

### 6.7.3 Relocation of Facilities

The cost of reconstruction of a new LMT is estimated at approximately $100 million for the tunnel only, and approximately $160 million with related infrastructure (source, SFPUC). Relocation will require significant lead-time to determine how and when this would occur from both physical and regulatory standpoints. Since protection of the LMT in-place may provide some protection to the treatment plant and associated facilities located landward of the LMT, relocation of the LMT may increase future costs.

Given that the LMT is early in its service life and its storage capacity, protection in place is the preferred option. Relocation may be considered in the future, especially if storm water management progresses to the point of reduced treatment requirements. In such a case where storage during wet weather conditions is not a primary driver, a smaller, pressurized pipe may be feasible for transporting sewage flows that are currently routed through the LMT. For example, prior studies have indicated that a 3-foot diameter pipe and a pump station is sufficient to handle sewage flows only (no wet weather storage), and this was called "Alternative B" in the preliminary feasibilities studies for the LMT (HLA 1981).

Conditions and operational requirements may change in the future, and therefore revisiting relocation and facilities modification alternatives is anticipated in the OBMP as part of the update anticipated to occur by 2030.

## 6.8 Synopsis

Adaptation will most likely comprise of constructing a low-profile wall as close to the tunnel as is practical, and then, where needed, constructing a lid or cap that acts as a structural hold-down of the tunnel. A secant pile wall is the most promising structural approach because it can be constructed from the existing bluff top, without compromising the required soil buffer. The crest of the wall can be limited to an elevation close to the top of the LMT, in order to accommodate a structural cover at the desired elevation consistent with the low-profile protection envisioned in the OBMP. The reinforced concrete cap can be installed later after bluff erosion, or the bluff could be excavated to complete the protection immediately. This low-profile protection will allow the beach to transgress over the top of the structure, and is most appropriate in the northern part of SOB, where the Colma formation is less extensive and vertical beach access is most important. Farther south, where the Colma is exposed and the bluffs rise near vertically to elevations around

40 to 50 feet above the beach, the cap slab may not be desired. Once the secant pile wall is exposed, the surface can be improved with additional concrete, including sculpting and coloring to resemble native bluff materials. This is compatible with additional beach management measures, such as nourishment and installation of a dynamic cobble revetment.

Of course, other options likely are available and implementation may vary by reach and apparent vulnerability. There are several structural options that can be better evaluated during design when more detailed information about subsurface conditions is to be available. For example, a soil grout wall could be constructed in place of a secant pile wall where a structural connection to a cap is not required, such as along the southern-most reach. There are several means of countering the buoyant uplift potential of the LMT without a structural cap, which may be acceptable in the very northerly portion where the LMT is low and farther inland. While cost is typically used as a metric for alternatives evaluation, it is important to implement the low-profile armoring that achieves the OBMP principals and objectives (see Section 3) and avoid the traditional, single-objective armoring design process which would result in further degradation of the shore.

Implementing these structural and shore restoration actions in combination with placing cobble and sand on the beach would lower the vulnerability of the LMT and extend its useful life. Placing the cobble on the beach as a dynamic revetment will allow for a steeper natural beach face that would increase the amount of protection over the structure while naturalizing the beach. Large-scale beach nourishment operations in partnership with the U.S. Army Corps of Engineers are assumed to be in place, depositing at least 500,000 CY of sand every 10 to 20 years.

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# APPENDIX 1

## South Ocean Beach Shore Recession Estimates, Ocean Beach Master Plan, with Consideration of TAC Input

Prepared by
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January 2014; revised February 2015

# 1. Introduction

The purpose of this appendix is to describe and evaluate the erosion extents calculated for South Ocean Beach (SOB) as part of the Ocean Beach Master Plan (OBMP[1,2]), and identify recommended changes. First, a summary of new information and recommended changes is provided. The calculation methodology is summarized and evaluated relative to factors identified by members of the Technical Advisory Committee (TAC). The original work was accomplished in 2010-2012. The TAC meeting occurred on October 31, 2013 at SPUR offices in San Francisco. Comments were provided by TAC members during November and December 2013. (Section 1.1)

This document was revised in February 2015 based on comments received from the TAC and San Francisco Public Utilities Commission (SFPUC). Review and comment was facilitated by TAC meetings on January 22, 2014 and November 14, 2014. (Section 1.2).

## 1.1 Recommended Changes to OBMP Erosion Projections Based on TAC input following October 31, 2013 Meeting

In general, TAC comments indicated that greater erosion may occur at SOB than projected in the OBMP. A summary of factors brought up and found in this review are listed below and are addressed in Sections 2 Summary of Erosion Calculations for SOB OBMP; and Section 3 Estimates of Future Erosion.

- Shore erosion has increased: Shore erosion in SOB has increased from about 1 foot per year (fpy) for the period 1930-1990 to about 2fpy when the time frame is extended to 2010. While this erosion rate is for the shoreline (beach), the back beach (bluff) has not receded as rapidly (that is, the beach has narrowed), and there is a concern that bluff erosion rates will also increase.

- Sediment supply will decrease: Recent study by the USGS (Barnard et al, 2012) indicates that sand supply to Ocean Beach is likely to diminish. Hence, future erosion rates may be greater than historic erosion rates.

- El Niño may intensify with climate change: Recent studies indicate a complex and uncertain relationship between climate change and climate fluctuations (Pacific Decadal Oscillation (PDO) and El Niño Southern Oscillation (ENSO, aka El Niño)), which are known to induce severe erosion events (Bromirski, 2011 and 2013; Santoso et al, 2013). Hence, there is a risk severe events could increase erosion extents to levels greater than the long-term average rate before 2050.

- Event erosion can be large relative to the average erosion rate: Several reviewers have noted that bluff recession of 10 to 50 feet can occur in one winter, and the projected 110 feet of recession by 2050 seems relatively small.

[1] San Francisco Planning + Urban Research Association (SPUR), 2012; Ocean Beach Master Plan. http://issuu.com/oceanbeachmasterplan/docs/obmp_document_full/11, last visited December 2013.
[2] ESA PWA, January 2012, Scenario Analysis – Coastal, Appendix 1, Ocean Beach Master Plan.

- Sand placement may be less effective than assumed: Recent modeling by the Army Corps of Engineers indicates that sand placed at SOB may not persist as long as previously estimated.

- Erosion resistance of Colma, Merced and buried rubble are not quantified: Available information is not sufficient to account for the erosion resistance of the Colma and Merced Formations in future erosion projections (subsurface data are evaluated elsewhere in this Methodology report).

Based on the above factors, we propose to include the possibility of greater erosion in the OBMP projections of shore evolution and costs. Implications to the OBMP are proposed as follows:

- An increase in the long-term shore recession rate from 1fpy to 2fpy adds 40 feet of recession by 2050, and an allowance for at least one severe El Niño winter or other extreme bluff erosion event would increase shore recession by another 40 feet. Therefore, an additional 80 feet of erosion could occur by 2050, and a range of 110 to 190 feet is proposed.

- Sand Placement: A higher rate of sand placement may be needed and hence we propose a range of 0.5M cy to 1.0M cy placed every 10 to 20 years[3].

- Subsurface exploration is recommended to ascertain the extent of Colma and Merced formation under the beach and within the bluffs. This information would be used to verify or refine the high-end erosion estimates, better assess LMT vulnerability, and assist with alternative feasibility analyses of actions to mitigate erosion hazards.

- Conduct more detailed analysis of sand transport at SOB, using multiple methods (e.g. the USACE modeling, monitoring of backpassing, model calibration) to better inform future sand placement needs and designs[4].

- Fast track interim treatments and implementation of OBMP elements so that erosion hazards can be mitigated.

[3] The SPUR technical report relies of a large volume of sand from the Main Ship Channel being placed onto the stretch of Ocean Beach south of Sloat Blvd. Although the SPN is developing a plan for one onshore placement (i.e., the CAP 204 plan), USACE has no authorization or appropriation for subsequent beach placement. Consequently, if the periodic sand-placement element of the SPUR plan is to be realized, someone outside of USACE will have to ensure that the authorization and appropriation are in place. (Source: John Dingler, US Army Corps of Engineers).

[4] If another party is interested in using the existing CMS grid to run further model simulations, then it is recommended that they follow up with the San Francisco District and/or Engineering Research Development Center (ERDC) to determine if a technology transfer of the model is feasible. The points of contact are James Zoulas (James.G.Zoulas@usace.army.mil)

## 1.2 Recommended Changes to OBMP Erosion Projections Based on TAC Input following TAC meetings on January 22, 2014 and November 14, 2014

The TAC comments pertinent to future erosion projections were limited to the following:

- Monitoring to inform actions and understanding: Monitoring of the shore response to sand placements and generally was emphasized by TAC members, who also noted substantial monitoring activity being accomplished by others such as the United States Geological Survey (USGS) and the California State University, San Francisco (SFSU). This comment was addressed by pursuing more extensive monitoring plans and coordination, and is not otherwise addressed in this Appendix.

- Update for new sea level rise guidance: Consider updating the Plan to consider recent guidance on sea level rise and coastal hazards. These Topics are addressed herein in Section 4 Sea Level Rise Policy and Guidance, and in Sections 3.5 SFPUC Maps and 3.6 FEMA Maps.

- Concern about damage risk: Concerns were expressed about the timing required to implement the project and the management of risks in the interim time frame. These concerns are being addressed by progressing with immediate management actions and implementation of the OBMP and therefore are not addressed by updating erosion estimates or otherwise addressed further in this Appendix.

# 2. Summary of Erosion Calculations for SOB OBMP

An equilibrium profile and sand budget methodology was employed for the OBMP. Conceptually, the shore profile shape is presumed to be fundamentally a response to wave dissipation, while the location of the profile shifts according to changes in sand volume and sea level[5]: If sand is added to the shore reach, then the shore will migrate seaward and vice versa; If sea level rises the shore migrates landward and up. This results in the following conceptual shore profile migration equations:

Shore horizontal translation = (Shore Movement Rate) * (time) - Sea Level Rise Recession; (1)

Sea Level Rise Recession = Sea Level Rise * ( Profile Width / Profile Height); (2)

Shore vertical translation = Sea Level Rise; and, (3)

Shore Nourishment Width Increase = Volume / Profile Height / Shore Length. (4)

These equations are similar to the volume equations in Figure 1, reproduced from Everts (1985). These methods utilize the “Bruun Rule” concept that the shore response to sea level change can be geometrically predicted based on the overall slope of the shore profile (Figure 2). The method is considered approximate but is simple to apply and responds to sea level change and sediment supply.

## 2.1 Bruun Rule

The Bruun Rule indicates that the lateral shore recession resulting from sea level rise is approximately equal to sea level rise divided by the slope of the shore face. Figure 3 shows shoreface profiles surveyed at Ocean Beach, San Francisco. The red lines show the slope used for this study. Note that selection of the shore face limits is ambiguous as demonstrated in Table 1 for a range of values. Hence, Table 1 indicates that the shore recession (landward movement) response to sea level rise could vary by +40% to -15% based on uncertainty in geometry selection. More recently, ESA PWA computed beach slopes for SOB using bathymetry provided by the United State Geologic Society (USGS) as part of the San Francisco Littoral Cell, Coastal Regional Sediment Management Plan[6]. The estimated slopes were 1:50.

5 Everts, Craig, Sea Level Rise Effects on Shoreline Position, J. Waterway, Port, Coastal, Ocean Eng. 1985.111:985-999.
6 ESA PWA, 2013, Plan Formulation Technical Appendices – Draft, SF Littoral Cell CRSMP, prepared for the Coastal Sediment Management Workgroup, via contract with the USACE, Los Angeles District.

**TABLE 1**
**RANGE OF SHORE FACE SLOPES POTENTIALLY SELECTED FROM FIGURE 3**
**(units are feet, elevations relative to Mean Lower Low Water)**

| Top Elevation | Bottom elevation | Height | Width | Slope (h:w) | Recession 2050 | Notes |
|---|---|---|---|---|---|---|
| 0 | 33 | 33 | 2000 | 1:60 | 70 | Used |
| 15 | 33 | 48 | 2500 | 1:50 | 60 | steeper |
| 15 | 35 | 50 | 3000 | 1:85 | 100 | Flatter |

NOTE: The 2050 recession is computed assuming a sea level rise of 14" and application of the Bruun Rule.

Another uncertainty in application of the Bruun Rule is the volume of material required to build the landward part of the profile (Everts, 1985: Rosatti, Dean and Walton, 2013[7]). Conceptually, the wave runup will extend farther landward with sea level rise, and deposit sand to build the profile up and landward, and require greater shore recession to generate the additional sand volume (see $V_g$ and $V_l$ in Figure 1). However, the shore at Ocean Beach is backed by constructed linear dune or bluff that extends well above the beach elevation indicating that this landward transport term is not significant.

Another uncertainty with the Bruun Rule is the availability of sand. In this study, it is assumed adequate sand is available to compensate for sea level rise, and that historic erosion rates continue in addition to the recession induced by sea level change. Also, the OBMP includes placement of sand to widen the shore and temporarily compensate for the ambient erosion and sea level rise induced recession (see $V_o$ in Figure 1). If net sand supply is less than required to satisfy the increased demand caused by sea level rise, then erosion could accelerate more than predicted by this method. However, because the large sewer culverts buried in the backshore and the extensive armoring already constructed along the shore, the erosion is likely to manifest in a reduction in the beach width and a change in profile. Therefore, improvements to this method might best be focused on assessing the effect of "squeezing" the beach between the rising sea and existing armoring and infrastructure. This loss of beach will conflict with ecology and recreation objectives, and also increase loadings on erosion control measures by increasing the height of the bluff and allowing larger waves to impact the bluff.

## 2.2 Historic Erosion Rates

The selection of shore change rates is an important factor in the profile response modeling. For SOB, a shore recession rate of -1 foot per year (fpy) was selected based on the rate from 1930 to 1995 (Battalio and Trivedi, 1996[8]; M&N, 1995[9]; M&N, 1994[10]). The historic shore position data

7 Rosati, Dean and Walton, 2013; The Modified Bruun Rule Extended for Landward Transport, Marine Geology 340 (2013) 71–81
8 Battalio RT, Trivedi D. Sediment transport processes at Ocean Beach, San Francisco, CA; Proceedings of the International Conference on Coastal Engineering, 1996 September 2-6, 1996; Orlando, CA. ASCE. p 2691-2704.
9 Moffatt and Nichol Engineers. 1995. Sediment Transport Processes Study, Ocean Beach, San Francisco, CA.
10 Moffatt and Nichol Engineers.1994. Shoreline Mapping for Ocean Beach, San Francisco, CA. Prepared for: U.S. Army Corps of Engineers, San Francisco District.

are shown in Figure 4, with more recent shore position data added as part of the SF Littoral Cell CRSMP (see Appendix 2 for projections of future shore erosion). The more recent data indicate that shore erosion rates have increased in the last two decades. This is consistent with studies by others as previously reported in the OBMP. However, Figure 4 shows that the shore position has fluctuated several hundred feet in the past. These reversible fluctuations are likely related to climate oscillations and possibly prior beach nourishment. Regardless, it is apparent that using the higher, short-term erosion rates would have over-predicted historic shore recession. Hence the longer term rate of about -1 fpy was selected for the bluff-controlled SOB area. If the higher rate of -2 fpy were to have been used, the net change would have been additional recession of 40 feet by 2050. These are shore recession rates which are not necessarily the same as bluff recession rates. The bluffs have retreated less rapidly than the shore, indicating the beaches have narrowed.

## 2.3 Bluff Recession

The methods applied in the OBMP do not explicitly consider the erosion resistance of bluff materials (except to use the lower, long-term recession rate of 1 fpy, as described above). The historic shore erosion rates are for beach sand, and the use of an equilibrium profile and Bruun Rule presume that the back shore is loose sand similar to that in the beach and surf zone. The historic shore erosion rates are for the shore fronting the sandy beach. The implication is that the beach has narrowed while the bluffs have been eroded more slowly and or been armored. Consequently, the SOB bluff face was adjusted by 40 feet landward in our OBMP analysis in order to account for profile response to armor removal.

Figure 5 shows pictures of the bluffs in SOB. The native materials are sandy, consisting of recent or ancient sand dunes, and uplifted lithified silty sand deposits of the Colma and Merced formation. The Colma and Merced formations are erodible but can stand vertically due to apparent cohesion. SOB has been modified by placing dune sand, earth and rubble to extend the bluff seaward to support development by the Government. Hence, the bluff is protruding boldly onto a low beach, and is frequently impacted by waves, with recession of the bluff wherever unarmored.

One of the complexities associated with SOB is the uncertainty about subsurface conditions and their erosion resistance. This uncertainty is partly due to variability in the site geology and partly because the area has been extensively modified since the late 1880s with almost no documentation of the changes. Therefore some review of history is potentially useful in forming judgment about the existing conditions and hence future shore response. Figure 6 shows a map from the mid 1800s with interpretation by Dr. Peter Baye. Note that the shore planform was different historically with a landward indentation and lower grades in the vicinity of the existing zoo and Sloat Boulevard. This low lying, cove-like formation was associated with an ephemeral channel that would carry water overflowing from Merced Lagoon. The channel was blocked in the late 1800s, as shown in Figure 7. A recent picture (Figure 8) shows a headwall exposed due to bluff erosion in 2010: It is logical to conclude that most of the bluff “erosion” is fill since the date stamped on the headwall is 1926 and it had been buried since at least the 1980s. This is further supported by the extensive rubble in the bluff (Figure 5). Figure 9 shows the filled and landscaped area after completion of the Clean Water Program sewer facilities in the early 1990s, and the subsequent eroded conditions in the early 2000s.

As indicated by Figures 5 through 9, the bluffs at SOB are neither homogeneous nor uniform. The following is a provisional conceptual description based on extensive research and judgment, pending better subsurface information. The existing SOB starts in the north near an old drainage swale from a back-barrier freshwater lagoon, now called Lake Merced, and extends south to the bluffs at Fort Funston. This area can be characterized as littoral sands and fill consisting primarily of dune sand. The shore has been filled seaward about 200 to 300 feet (Olmsted and Olmsted, 1979[11]: Battalio and Trivedi, 1996), and the beach has narrowed as the shore has receded while the bluffs have receded less. Some rubble, debris and earth exist from prior construction and dumping by San Francisco in the period 1880 to 1985, and from erosion of parking and access facilities damaged since 1990. The bluff top in this north region is about 30 feet above Mean Lower Low Water (MLLW). In contrast, the bluff in far southern end of SOB, near the sewer treatment plant, is about 50’ MLLW. The bluff continues to rise toward the south into Fort Funston. This area is uplifted due to seismic activity, and consists of Colma formation underlain by older Merced Formation. The bluff tops in Fort Funston are loose sand from ancient and recent wind transport, while the bluff tops in SOB have been filled and graded for road and other construction. The lithified Colma and Merced dip northward and eastward, forming a subsurface ridge generally paralleling the shore. At very low tides during the winter, outcrops of weak sedimentary rock can be observed in the nearshore. Hence, we think there is a wave-cut terrace and bluff overlain by littoral sands and fill. The SOB transitions between the low, sandy north and the more cliff-like south. While much of the erosion on the north side is in fill, most of the erosion on the south side is now in the native bluffs (Figure 5). A key data gap is the location of the Colma and Merced formations and their erosion resistance where not exposed. Of note, the Lake Merced Tunnel was designed to be tunneled into the Colma, and may explain why the expensive infrastructure was located so far seaward. This raises the question, if the Colma was substantial for the preferred tunnel substrate, should we account for the likelihood that it can withstand erosion potential better than loose sand? The erosion analysis accomplished for the OBMP also neglects this variability of materials except to the extent represented by historic erosion rates. Similarly, in contrast to the studies for the LMT design, the recent studies assessing the vulnerability of the LMT assume loose dune sand and neglect apparent cohesion. Our present study is focused on filling this data gap to account for the presence of materials in the bluff and shore that are more erosion resistant than loose dune sand.

Bluff recession is estimated in the Regional Sediment Management Plan (see the section on the San Francisco Littoral Cell Coastal Regional Sediment Management Plan and Appendix 2) in terms of an historic rate which is increased as the fronting beach narrows and decreased as the fronting beach widens. The no-action bluff erosion at SOB was computed to be nearly 70 feet by 2050 and nearly 100 feet by 2100. With adaptation based on the OBMP, the computed bluff

[11] Olmsted, 1979. Olmsted, R. and Olmsted. N. (Feb 1979) Ocean Beach Study: A Survey Of Historic Maps And Photographs.

recession was reduced by about half, to about 30 feet by 2050 and 50 feet by 2100. Event erosion was added to these recession estimates in the RSM (Appendix 2).

## 2.4 Event Erosion

Event Erosion refers to short term erosion events associated with severe storms, clusters of storms and severe winters such as those that can occur during an El Niño climatic condition. The OBMP did not explicitly compute event erosion, but rather used long-term erosion rates averaged over time.

The potential bluff erosion has been estimated previously for SOB using statistical analysis of historic erosion (USACE, 1992[12]; M&N, 2010[13]). Figure 10 shows the estimated bluff top recession extents versus return period. The figure indicates that a bluff recession of 40 feet is likely to occur about once in 30 years. Figure 11 shows the bluff top recession that occurred during the 2009-2010 winter. Based on these data and analysis, it was concluded that 40 feet of additional bluff top recession was likely within the next 7 years. Significant concerns were raised by the Coastal Commission and other environmental groups that the emergency armor placement and proposal to construct a 30 to 50 foot tall wall along the shore to prevent bluff recession would cause adverse effects on the fronting beach, including impacts to recreation and ecology. The OBMP includes an expected recession of the bluff top of 40 feet, and limits armoring to that needed to protect the LMT below the toe of the bluff.

Bluff erosion in response to sea level rise was first estimated by PWA in 2008 (See the section on The Impacts of Sea Level Rise on the California Coast). The short-term bluff recession due to extreme storm conditions was estimated as two times the standard deviation of long-term bluff recession along the shore for the length of time forecasted. For bluffs at SOB this amounted to 49 feet of landward recession of the bluff by 2050 in addition to the long-term trend of bluff recession. Conceptually, the additional 49 feet provides the landward envelope of extreme erosion that may occur in some places but not everywhere.

Regardless of how the bluff recession is estimated, or how actual recession occurs, by nature or design or both, at least 40 feet of bluff recession is expected by 2050, if not in the next decade.

A driver of event erosion is the climatic condition called El Niño, which is a periodic climate occurrence affecting winds, water levels and storm tracks pertinent to the California coast (Komar and Allen, 2004[14]; FEMA, 2005[15]). Water levels can increase by 2 feet for short periods and elevate average water levels over the entire winter by about 1 foot. These higher water levels increase erosion because larger waves break at higher elevations on the shore (Revell et al, 2011).

12 USACE, 1992; reported by M&N, 2010.
13 M&N, 2010, Great Highway Emergency Repairs, Response to 2009 / 2010 Storm Wave Erosion, Draft, Prepared for City and County of San Francisco, Department of Public Works, September, 2010.
14 Komar and Allen, 2004 Komar, P.D. and Jonathan C. Allan. 2004. El Nino Processes and Erosion of the U.S. West Coast, *Technical Memo*, August 9, 2004.
15 FEMA, 2005; Guidelines for Pacific Coast Flood Studies. http://www.fema.gov/media-library-data/20130726-1541-20490-9741/cg_pacific.pdf

Also, El Niño conditions can increase storm intensity and modify storm tracks with the effect of increasing the wave power incident to the California coast. Erosion during an extreme El Niño winter can amount to the same net erosion over several decades. For example, the values for bluff erosion estimated in the last three events are on the same order of magnitude (10 to 40 feet, Table 2) as the long term recession at 1 fpy for 40 years between 2010 and 2050 (40 feet). Note the bluff was not filled as far seaward in 1982-83, which was prior to completion of the Clean Water Program development of the bluff top. The bluff position relative to the surf is a consideration: Obviously, a bluff constructed in the surf zone would erode more quickly than one set back and rarely impacted by waves and hence a more seaward location should erode more quickly.

**TABLE 2**
**BLUFF RECESSION DISTANCES AT SOB, 1997-2010**

| Winter | Bluff Recession (feet) | Severe El Niño Winter (yes/no) |
|---|---|---|
| 1997-1998 | 20’ | Yes |
| 2002-2003 | 10’ | No - moderate |
| 2009-2010 | 20’ to 40’ | Yes |

SOURCE: Moffatt & Nichol Engineers, 2010.

Whether the total recession forecast for SOB is sufficient to account for occurrences of El Niño events is not known. The uncertainty is exacerbated by the potential for climate change to increase frequency and or intensity of El Niño events (Santoso, 2013; Bromirski, 2011). Since water levels are elevated during strong El Niño winters, these conditions can provide some insight into the conditions expected with future sea level rise. However, the duration of high water levels during El Niño conditions (e.g. one winter season or year) is probably not sufficient to have the same effect of permanent sea level rise on shore change. Erosion is work done by wave power[16], and hence time duration is important. For example, the erosion of a single storm is much less than the theoretical maximum erosion that can occur for a storm of infinite duration (FEMA, 2005). More specifically, observations at SOB indicate bluff recession during El Niños of 10 to 40 feet (Table 2), whereas the shore recession from sea level rise of 1.2’ is estimated to be 70 feet.

When considering event erosion, it is useful to consider backshore conditions. Figure 12 shows the substantial change in beach elevation between surveys in 2004 and 2010 (Source: USGS, 2010, provided by Frank Filice, SFDPW). The greatest net change is near the sewer outfall pipe (location also shown in Figure 12). The armoring protecting the outfall pipe is frequently exposed, as shown in Figure 13. The exposed features are steel sheetpiles and quarry stone armor / ballast. Erosion in the vicinity of this location has been a problem since 1999. In the early 2000s, a sacrificial sand berm was placed north of the outfall and a bypass road was constructed in anticipation of bluff erosion. In 2010, erosion occurred south of the outfall, resulting in

16 Wave power is characterized as wave energy per time, and is proportional to $H^2T$, where H is the wave height, and T is the wave period.

revetment construction under emergency declaration. The proximity of the outfall to this erosion area has been discussed since the early 2000s by the Ocean Beach Task Force, based on observations of rip current formation, scour parallel to the outfall and the loose materials in the bluff at this location associated with construction. More recently the USGS has hypothesized that the outfall has affected offshore conditions, incident waves and sand transport (Hansen et al., 2013).

Figure 14 shows that the beach elevation can change markedly in just a few months (Source, Moffatt and Nichol Engineers). These pictures are taken from the north parking lot, and show the sand bags placed in 2011 and the rock revetment placed in 1999. The beach elevation changes are partly attributed to the erosion resistance of the bluff, which increases wave reflection and thereby modifies the nearshore profile by lowering it.

Continuing to construct armoring and attempting to maintain the bluff in the present location will likely exacerbate the extensive vertical beach fluctuations, cause the beach to narrow, and increase the potential for episodes of large bluff erosion, all of which will cause the cost of “holding the line” to increase. Armoring the bluff also degrades the beach width and elevation to the point where the beach ecology and recreation are limited to non-existent.

## 2.5 Beach Nourishment and Sacrificial Dune Placement

The OBMP calls for about 2 million cubic yards (2Mcy) of sand to be placed along SOB and MOB every 10 to 30 years. The presumed source is offshore of Ocean Beach, either as a modification of the ongoing navigation dredging of the San Francisco Bar Channel or other offshore mining. The existing navigation dredging produces about 300,000 cubic yards of sand per year (average of a fluctuating dredge volume accomplished every one to two years). This is about 3M cubic yards per decade, and therefore seemingly enough to satisfy the OBMP. However, the dredging requirement to maintain the navigation channel is expected to decrease in the future, and the sand is on average (median grain size) finer than desired for beach nourishment, and more consistent with dune-sized (wind-blown) sand. Also, the frequency of dredging, equipment and funding for the navigation dredging will require modification to accomplish the proposed beach nourishment.

The OBMP presumes that the 2Mcy will be placed to form a sacrificial linear embankment (aka “dune”) about 50 feet wide and also extend the entire shore face (aka surf zone profile) another 50 feet offshore. About 1.5Mcy would be placed in MOB and about 0.5Mcy would be placed at SOB. This is a “straw man” conceptual sand placement plan that is not intended to be prescriptive but rather a starting point for subsequent feasibility analysis and design.

Sand has been placed at SOB previously, with the recent significant placements as follows:

- Rubble and sand from MOB was placed at SOB as part of the Clean Water Program in the 1980s and 1990s. The placed sand was vegetated. Visitor parking and restrooms were constructed on the fill. These facilities became damaged as the fill eroded, and damages continue to progress. See Figure 9.

- Sand was placed to mitigate an erosion hot spot in front of the sewer plant just north of the sewer outfall pipe three times between 1999 and 2002. The sand was placed to form a sacrificial embankment in lieu of armor. The sand source was the central Bay shoals, and was coarse with shell fragments, was effective but was expensive. See Figure 15.

- Sand was placed along the northern section of SOB in 2012. About 70,000 cubic yards of sand were excavated from NOB, trucked to SOB, and placed to form a sacrificial sand berm. This activity was called “sand backpassing” because the sand placement was generally “updrift” of the borrow site, presuming wave-driven, along-shore sand transport is from south to north. See Figure 16.

Recently, the US Army Corps of Engineers (USACE) Engineering Research and Development Center (ERDC) and the San Francisco District conducted a modeling analysis of sand berm placement at SOB. A description of the study and results are provided in Exhibit 1[17]. The study looked at a 0.3Mcy placement along SOB, and concluded that the sand would migrate away from the placement location at the rate of about 0.08 to 0.1Mcy per year, indicating that the sand placement may persist for as short as 3 to 4 years. Observations of the 2012 backpassing indicate that the sand will not persist beyond two winters, which so far have been fairly mild. This implies a transport rate of about 0.03 to 0.04Mcy per year in mild years. Based on these rates of transport, 0.03 to 0.1Mcyy, the 0.5Mcy placement proposed in the OBMP would persist for about 5 to 17 years.

Sand placement is being studied as a component of Regional Sand Management (RSMP) for the Ocean Beach Littoral Cell (see Appendix 2, Figure 6). This analysis indicates that a shore widening of 50 feet every 20 years would result in a shore recession of about one foot per year, but would maintain an average beach width greater than the existing. The estimated volume is 0.77Mcy per placement. While the beach width is maintained, the location of the back beach is maintained only for about 15 of the 20 years because the backshore (bluff) is predicted to recede as well. This analysis therefore indicates the beach widths can be maintained more effectively if the backshore (bluff) is allowed to recede, versus the traditional concept of using beach nourishment as “soft armoring” to prevent back shore erosion and shore migration. This is perhaps the difference between the Corps’ modeling results and the RSMP and OBMP projections: The former implicitly presumes that the purpose of the sand placement is to prevent backshore erosion whereas the latter explicitly attempts to maintain a beach while only slowing backshore erosion. Correcting for this difference in performance criteria, a review of Figure 6 of Appendix 2 indicates a loss rate of about half that projected by the USACE modeling (0.77Mcy persists about 15 years, indicating a loss rate of about 0.05Mcyy). This is attributed to the RSM and OBMP using average erosion rate values while the USACE study used more extreme erosion rates. However, the analysis methods are very different and hence significant method uncertainty may also exist. The 0.05Mcyy loss rate from the RSMP does compare favorably with the observed loss rate from the 2012 placement of 0.03 to 0.04Mcyy, however.

[17] Personal communication, Frank Wu, USACE SF District.

In summary, the effectiveness of sand placement at SOB is not known and success criteria are not well defined. It is unlikely that sand placement can prevent shore (bluff) recession, whereas allowing some bluff recession will increase the feasibility of maintaining a beach. The available information at this time indicates that the OBMP presumption of 0.5Mcy placement every 10 to 30 years should be revised to entail more frequent placement, say every 10 to 20 years, and or a greater volume per placement of closer to 0.8Mcy to 1.0Mcy.

# 3. Estimates of Future Erosion

Future erosion has been estimated by several groups. The following sections summarize these estimates.

## 3.1 Ocean Beach Master Plan

The Ocean Beach Master Plan (OBMP) included evaluation of potential coastal erosion and flooding for existing conditions and future conditions through 2100 with one sea level rise scenario, for one "no action" scenario and for five adaptation scenarios (SPUR, 2012). An equilibrium shore profile translation model was used to predict shore change, as described elsewhere in this Appendix 1 and ESA PWA (2012). The shore recession values are shown in Table 3. For south Ocean Beach (SOB), it was estimated that the shore would recede 70 feet by 2050 and 275 feet by 2100 due to sea level rise, plus an additional 1 fpy due to continuation of historic erosion, yielding total recession distances of 110 feet by 2050 and 365 feet by 2100. Also, the bluff was expected to recede about 40 feet more because its existing location is not considered sustainable and is lagging the erosion potential. Therefore, bluff recession was estimated to be 110 feet by 2050 and 365 feet by 2100, plus the 40' landward adjustment in bluff top. These erosion estimates are potential. For the selected adaptation strategy, which included retreat, sand placement and armoring, and the recession was re-estimated. However, because SOB is already out of equilibrium, we maintained the 110' bluff recession estimate for 2050 and focused on restoring beach width via sand placement, including a sacrificial sand bluff. After 2050 and probably not later than 2070, we expect that additional armoring or infrastructure realignment will be required. A lower rate of sea level rise may result in less erosion, given that a high sea level rise scenario (+55") was used for the OBMP. Figures 17 and 18 show the future erosion in section and plan, respectively. Note that we expect that the coastal bluff will migrate landward of the LMT by 2050, even with a million cubic yards of sand placement.

**TABLE 3**
**CUMULATIVE SHORE TRANSGRESSION (TOP) IN RESPONSE TO SEA LEVEL RISE AND CUMULATIVE SHORE RECESSION DUE TO SEA LEVEL RISE AND PROJECTION OF ESTIMATED EROSION RATE**

| | | Cumulative Shore Transgression | | |
|---|---|---|---|---|
| Date | Years | Relative Sea Level Rise | Profile Lift | Profile Recession |
| 2030 | 20 | 0.6′ (7″) | 0.6′ | 35′ |
| 2050 | 40 | 1.2′ (14″) | 1.2′ | 70′ |
| 2100 | 90 | 4.6′ (55″) | 4.6′ | 275′ |

| | | Cumulative Shore Recession (erosion plus rslr transgression) | | |
|---|---|---|---|---|
| Date | Years | NOB (+1 fpy) | MOB (-1 fpy) | SOB (-1 fpy) |
| 2030 | 20 | -15′ | -55′ | -55′ |
| 2050 | 40 | -30′ | -110′ | -110′ |
| 2100 | 90 | -185′ | -365′ | -365′ |

## 3.2 San Francisco Littoral Cell Coastal Regional Sediment Management Plan (SF-CRSMP)

Coastal erosion and adaptation strategies are being investigated as part of a regional study called the San Francisco Littoral Cell Coastal Regional Sediment Management Plan (RSMP). ESA PWA is conducting this work for the Coastal Sediment Management Workgroup via contract with the US Army Corps of Engineers, with assistance from the Association of Bay Area Governments (ABAG). The methods and results are described in Appendix 2. Figure 19 shows the projected bluff location for future years assuming actions similar to the OBMP.

A higher rate of sea level rise was modeled, amounting to 1.6 feet (19 inches; based on the "high curve" from USACE 2011 guidance) for the RSMP versus 1.2 feet (14 inches; based on the OPC 2011 interim guidance) for the OBMP. A steeper shore face slope of 1:50 was used for the RSMP than the 1:60 slope used for the OBMP. The analysis was more detailed and included modeling of beach width and the effects on bluff recession. The results are similar to those developed for the OBMP. See Appendix 2 for more detailed explanation.

## 3.3 The Impacts of Sea Level Rise on the California Coast

The State of California funded an assessment of the scale of potential effects of sea level rise on the coast of California. The project was accomplished in 2008 and published in early 2009, and used in the State-wide climate change vulnerability assessment and adaptation strategy (Heberger et al, 2009[18]; PWA, 2009[19]; Revell et al, 2011[20]). The erosion modeling was accomplished by PWA, in support of The Pacific Institute who was the lead investigator. Substantial assistance was provided by Scripps with output from global climate modeling and the Coastal Data Information Program with wave time series. The USGS provided analysis of San Francisco Bay response. This work was the first to quantify the amount that coastal erosion will accelerate with increasing rates of sea level rise, and found that coastal high ground was not beyond the reach of climate change. Figure 20 shows the hazard zones estimated for the year 2100 assuming 55" of sea level rise. These estimates were not intended for local planning purposes, but are widely used and available via the Pacific Institute. The erosion hazard zones shown in Figure 21 are the same shown in the CCAMP Discovery Map for San Francisco published by FEMA, and referenced to the Pacific Institute (Figure 20)[21].

[18] Heberger M, Cooley H, Herrera P, Gleick PH (2009) The impacts of sea level rise on the California Coast., California Climate Change Center. CEC-500-2009-024-F

[19] PWA, 2009. California Coastal Erosion Response to Sea Level Rise - Analysis and Mapping. California Climate Change Center. PWAOPC-1000-2009-013.

[20] Revell, D.L., Battalio, B., Spear, B., Ruggiero, P, and Vandever, J. 2011. A Methodology for Predicting Future Coastal Hazards due to Sea level Rise on the California Coast. Climatic Change (2011) B.V. 2011 109 (Suppl 1):S251–S276, DOI 10.1007/s10584-011-0315-2, 10 December 2011 # Springer Science+Business Media

[21] http://www.bakeraecom.com/wp-content/upLoads/2010/02/SF_Discovery_Map_v1a1.pdf

## 3.4 Our Coast Our Future

The USGS, NOAA, and others have developed hazard mapping for the area as part of the Our Coast Our Future project[22]. Figure 22 is a screen shot taken from the OCOF interactive web site, and shows the projected erosion through 2100 for SOB. The erosion for 2050 was based on 50cm of sea level rise (vs 35 cm for the OBMP) and 150 cm of sea level rise was used for 2100 (vs 140 cm for the OBMP). Storm induced erosion is not mapped separately.

## 3.5 SFPUC Maps

Inundation mapping as part of the SFPUC's SSIP Westside Sea Level Rise Mapping (SFPUC, 2014) used a time series analysis of a 50-year synthetic record of water level and wave data to estimate the 100-year dynamic water level (DWL), which includes a combination of an extreme still water level (SWL) and wave setup. However, there are two major omissions that render the Mapping of limited use to the OBMP: wave runup and erosion. The technical memorandum (SFPUC, 2014) summarizes their estimates of the differences between SWL, DWL and Total Water Level (TWL) which includes wave runup. The mapping uses the DWL which does not include wave runup. Figure 23 is shows portions of the maps for low (12") and high (66") sea level rise for the South Ocean Beach study area. Based on these maps, the risk of coastal inundation extends from the sea to the dune or bluff face for existing and future conditions. These hazard zones are generally consistent with the concern that coastal hazards will extend inland over time if the shore erodes, but under-predict future hazards by ignoring erosion and wave runup.

## 3.6 FEMA Maps

The Federal Emergency Management Agency (FEMA) maps coastal hazards for communities participating in the National Flood Insurance Program (NFIP). FEMA has developed provisional maps for San Francisco but these maps have not yet been adopted and are therefore not "effective." Figure 24 shows a publically available version of the provisional maps which may be representative of the maps to be publically released (FEMA, 2013). The maps represent the extent of Total Water Level (TWL) with an annual recurrence probability of 0.01, also called the 1% annual chance event, and also more commonly called the 100-year event. The maps do not include future conditions such as erosion and sea level rise. The map shows wave runup on the dune face and reaching the roadway in one location. Updated information indicates that these maps (FEMA, 2013) have been revised to show lower wave runup (see below, FEMA, 2014).

In addition to these "standard" FEMA maps, FEMA has conducted a study of sea level rise effects on flood risk at Ocean Beach as part of a national pilot project called the West Coast Sea Level Rise Pilot Study[23]. Figure 24 includes a slide from this presentation indicating that the 100-year runup does not reach the bluff crest (FEMA, 2014) at South Ocean Beach under existing conditions. In addition, the pilot study looked at two ways of adjusting FEMA maps for sea level

22 http://data.prbo.org/apps/ocof/
23 FEMA 2014. http://www.floods.org/Files/Conf2014_ppts/E7_Curtis.pdf last visited February 2015.

rise: (1) adding sea level rise to the TWL elevation and (2) recalculating the TWL with the ocean level increased by sea level rise. The study finds that method (2) results in a much larger increase in computed TWL than method (1). They found that both methods would compute TWL exceeding the bluff crest at SOB in the future. The study also considered coastal erosion based on the TWL and sea level rise computations. Note that the TWL was not updated with the eroded profile. Figure 25 is a slide from the presentation showing the results for one foot of sea level rise and 3 feet of sea level rise. These erosion extents are greater than the extents predicted by most prior studies summarized here and are not for planning purposes.

Perhaps the most pertinent element of the FEMA pilot study is as stated in the presentation *"Shoreline retreat mitigates impact of SLR as shoreline adjusts to new equilibrium position."* This finding is consistent with the conceptual framework of the OBMP: Coastal hazards can be mitigated by retreat from the shoreline which incrementally restores (to the extent retreat is adequate) the coastal floodplain and increases wave dissipation seaward of the setback development.

## 3.7 Summary

All the published erosion estimates for SOB with sea level rise are similar, except for the FEMA pilot study which shows larger theoretical erosion extents. We expect that differences are within method and other uncertainty and hence are not worthy of further evaluation for the OBMP given that the hazard is already considered significant. More accurate estimates of future erosion are expected in the future, both due to monitoring, improved methods and better estimates of likely climate change effects such as sea level rise rates and storm effects. However, all methods indicate that coastal hazards are intensifying at SOB, and that action is required to prevent further degradation to infrastructure, ecology and recreation.

# 4. Sea Level Rise Policy and Guidance

Several recent advances in the policy and guidance of sea level rise have been developed since publication of the OBMP and its supporting technical analyses. Of note, completion of the National Research Council's (NRC) report *Sea-Level Rise for the Coasts of California, Oregon, and Washington: Past, Present, and Future* presented sea level rise projections over the next century, and extended these further to approximate ranges that account for vertical land motion (NRC 2012). Subsequently, the Ocean Protection Council (OPC) adopted sea level rise guidance based on the NRC (2012) ranges, and specifically requires that State agencies plan for sea level rise and coastal impacts (OPC 2013). Further, the California Coastal Commission (CCC) developed draft policy guidance for addressing sea level rise and coastal impacts for development along the California coast, which is based on the NRC (2012) ranges and which also includes coastal erosion (CCC 2013). The OBMP coastal analysis is consistent with these new guidance documents with a couple of exceptions, primarily:

1. The OBMP used the OPC 2010 interim guidance in effect at the time, which had slightly different sea level rise values;

2. The OBMP used only the high OPC 2010 curve (16" by 2050 and 55" by 2100).

The City and County of San Francisco (CCSF) has developed additional guidance called "One SF" that addresses sea level rise policy[24]. This document provides guidance regarding selection of the sea level rise scenarios in NRC (2012), identifies the mid-range curve as the most likely "projection" and identifies the high range as "outer bound" values. This mid range curve predicts 11" by 2050 and 36" by 2100, which are lower than the high range values used in the OBMP. However, the high range values in NRC (2012) (24" and 66", respectively) are higher than used in the OBMP.

> *Regarding selection of a sea level rise scenario, the guidance is generally consistent with the OBMP use of high-range sea level rise projections given the critical importance of the sewer system at SOB (Page 13):*
>
> *"They may choose to plan now for the high end of the uncertainty range (i.e., 66 inches by 2100) – particularly for critical assets that must maintain their function if inundated – or it may be appropriate to plan for the most likely scenario by 2100 (i.e., 36 ± 10 inches) while completing sensitivity testing and developing appropriate adaptation strategies that could accommodate higher sea level rise estimates. This latter approach accommodates uncertainties in the science and allows for flexibility should the higher-end of the sea level rise projections become more likely. "*

[24] OneSF, CCSF, http://onesanfrancisco.org/sea-level-rise-guidance/ last visited, February, 2014.

The guidance states that wave action and coastal erosion should be considered (page 14):

> *"The Westside inundation maps leverage Federal Emergency Management Administration (FEMA) water level and storm surge data and coastal hazard analysis methods that consider shoreline types (i.e., sandy beaches, dunes, bluffs), presence of coastal structures, and erosion potential. The inundation maps include a range of sea level rise estimates from 12 inches to 66 inches, and account for the dynamic overland water levels associated with sea level rise-driven changes to the 100-year coastal storm surge and wave hazards. These maps were published in June 2014 and are available through the SFPUC and the Sea Level Rise Committee. "*

Note however that the above characterization of the Westside inundation maps is incorrect, because these maps do not include wave runup or coastal erosion (see Section 3.5 SFPUC Maps). Fortunately, the OBMP did include coastal erosion and wave runup hazards in its development, and appears to be generally consistent with the OneSF guidance.

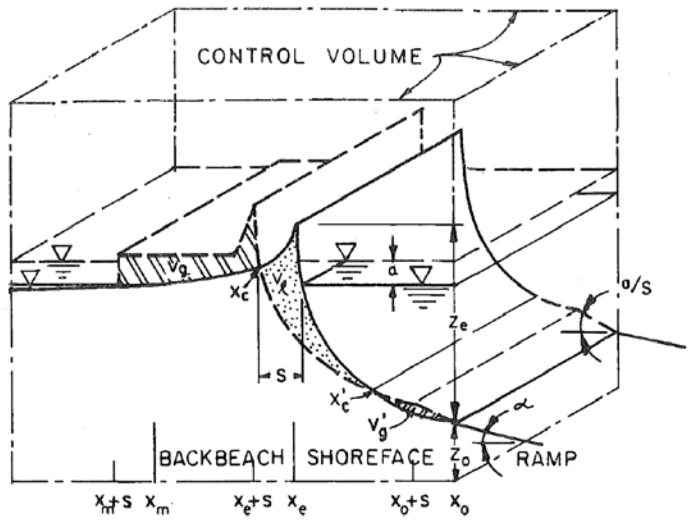



**FIG. 3.—Definition Sketch**

$$kV_l + V_o - (V_g + V_g') = 0 \ldots\ldots (4)$$

**Figure 1:** Definition sketch and equation for profile migration modeling according to Everts (1985):

$V_l$ = the erosion volume

k = the percentage of sand coarse enough to be stable in the depositional zones

$V_g$ = the inland deposition volume

$V_g$' is the offshore deposition volume

$V_o$ is the net supply of sand from outside the control volume, typically along shore transport or beach nourishment, or a deficit which is due to offshore transport or manifested by historic erosion rates

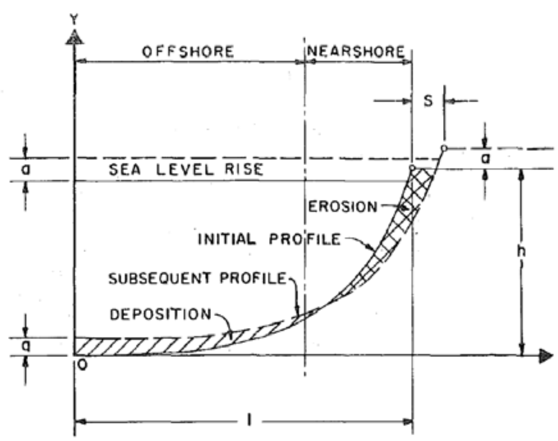



**FIG. 2.—Bruun's (1983) Concept of Shore Erosion as Sea Level Rises**

Using the assumptions that: (1) Beach and offshore profile equilibrium exists; and (2) the shore in question is in a state of quantitative materials balance, Bruun (4) determined the practical approximation of shoreline movement, $s$ (Fig. 2), to be

$$s = \frac{al}{h} \ldots\ldots (1)$$

in which $a$ = $RSL$ rise; $h$ = maximum depth of exchange of material between the nearshore and the offshore; and $l$ = length of the profile

**Figure 2:** Definition sketch and equation for Bruun Rule, from Everts (1985)

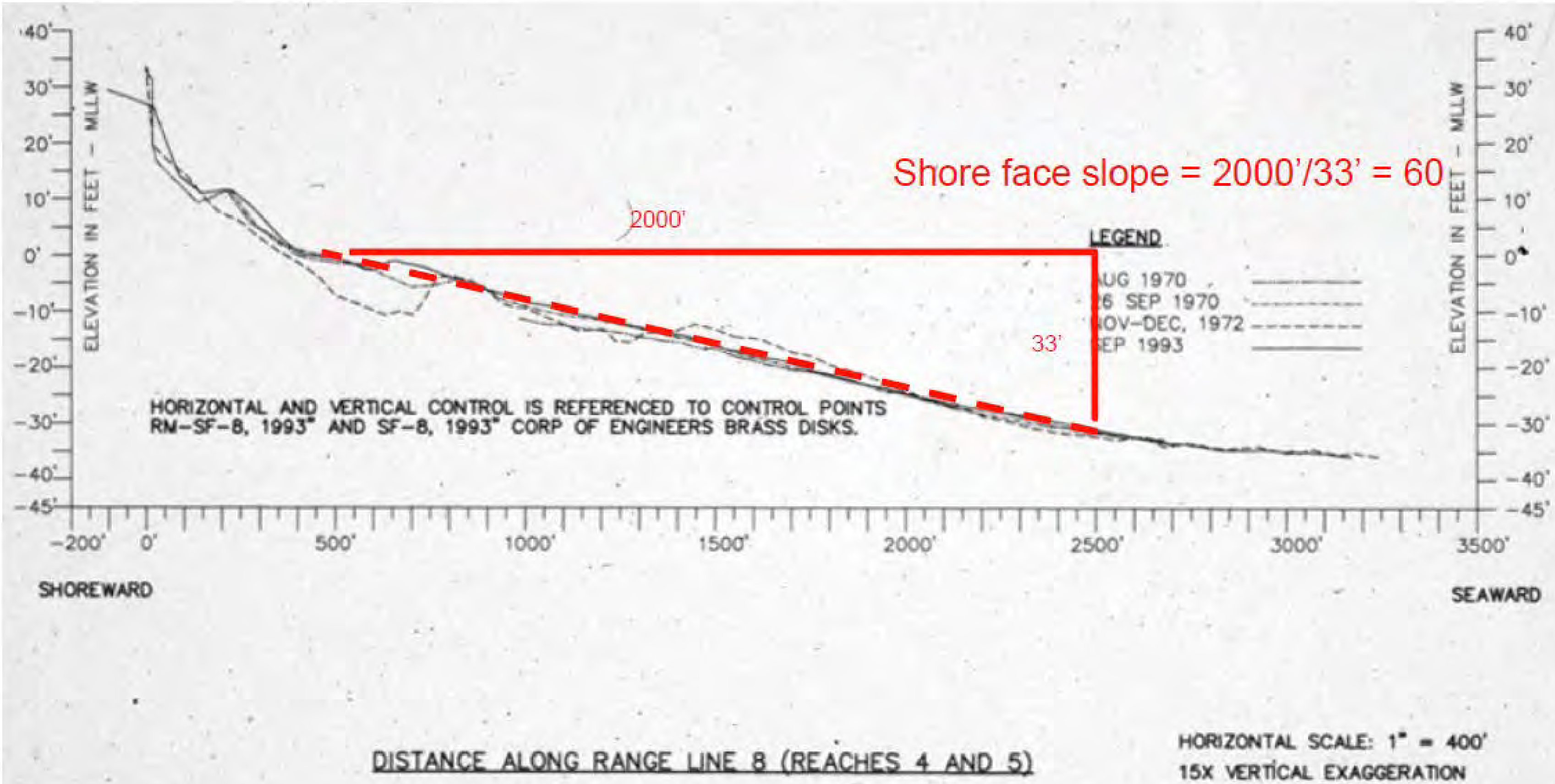



**Figure 3:** Elevation surveys though the surf zone at Ocean Beach, San Francisco (Modified from source: Moffatt & Nichol Engineers, 1995). The red lines show a slope of 1:60 (vertical:horizontal) calculated between the low tide and offshore. Note that steeper slopes can be computed by including more of the beach and dune face. For example, extending the top of slope to beach-dune face junction of about elevation 15' and extending seaward to closure of -33' yields a profile height of 48' and a width of about 2,500 feet yields a slope of about 1:50. If the profile is extended seaward to -35' the slope is 50 feet/3,000 feet, which is about 1:85.

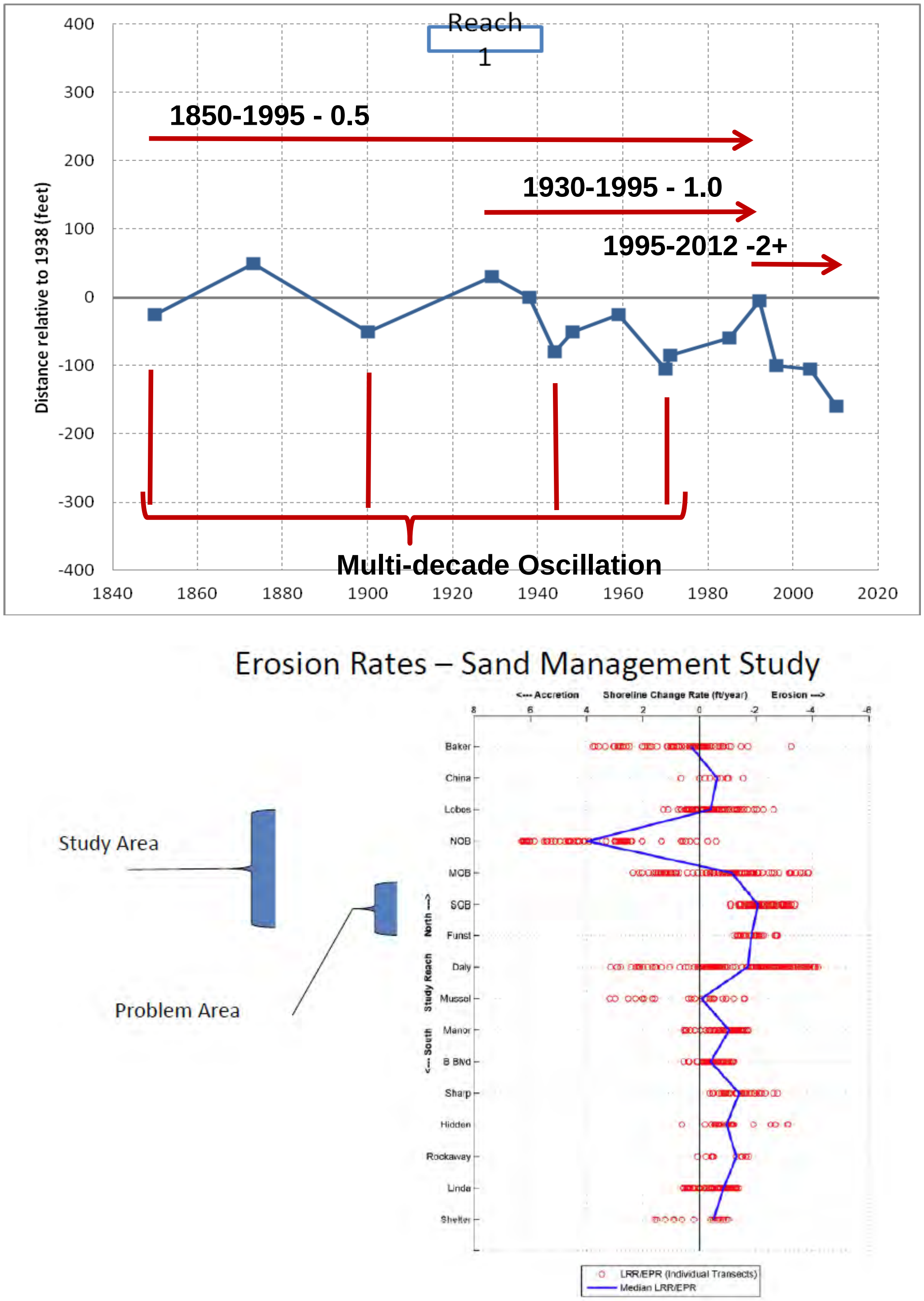



**Figure 4:** (a – Above): Historic shore position change time series for Fort Funston and southern portion of South Ocean Beach. Derived from Sources: ESA 2005, MNE, 1995; January 2010 location added from January 28, 2010 survey by the USGS. (b – Below): Shore recession rates for multiple profile locations and time frames. Source: ESA PWA in press.

**Figure 5:** Pictures of bluffs in SOB

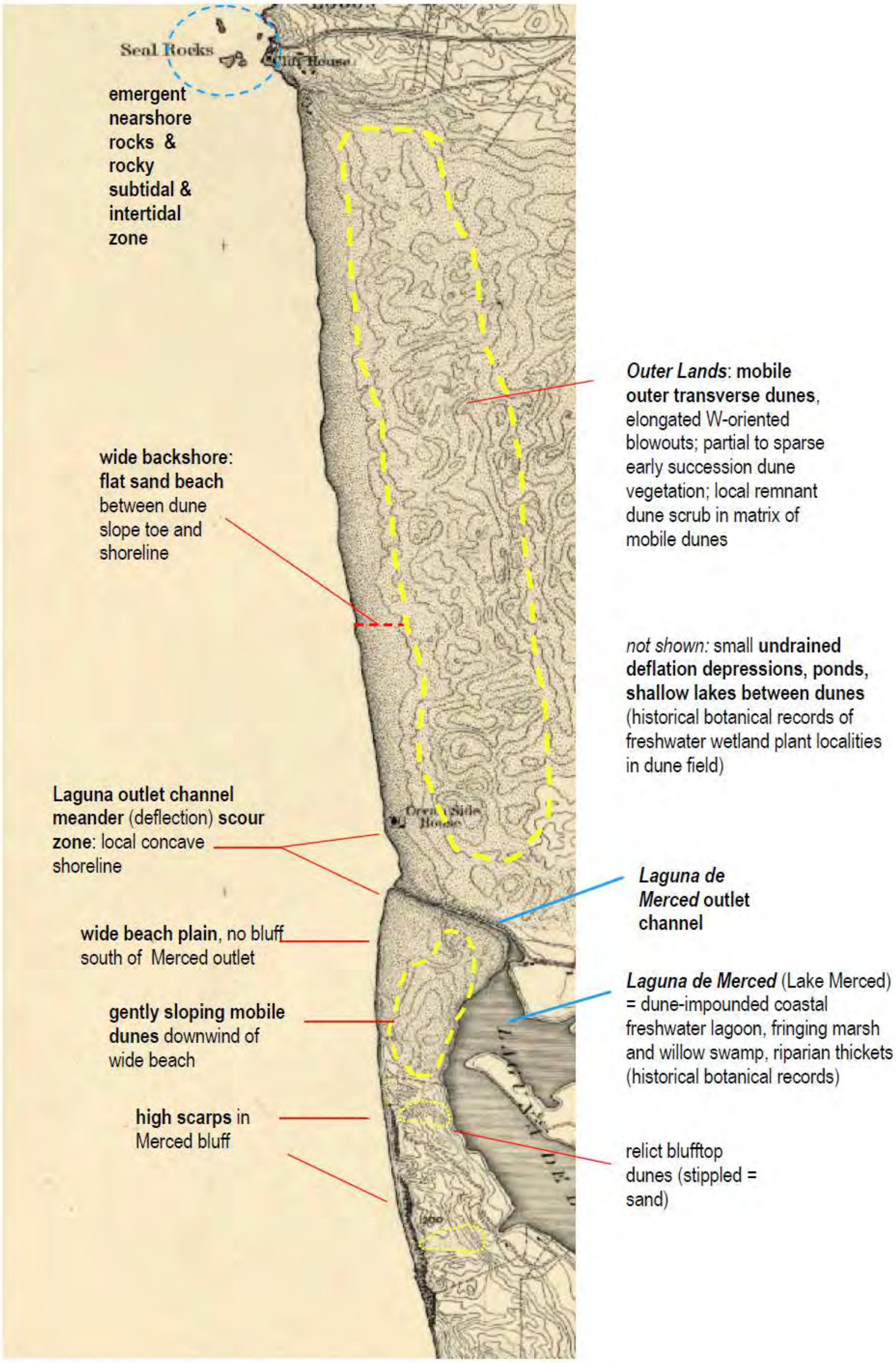



**Figure 6:** San Francisco coastline 1869 (composite U.S. Coast Survey T sheets 1850s topography): Seal Rocks, Ocean Beach, Outer Lands (western dunes: Sunset, Richmond), Lake Merced outlet, Merced bluffs and bluff-top dunes.
Source, Peter Baye, PhD.

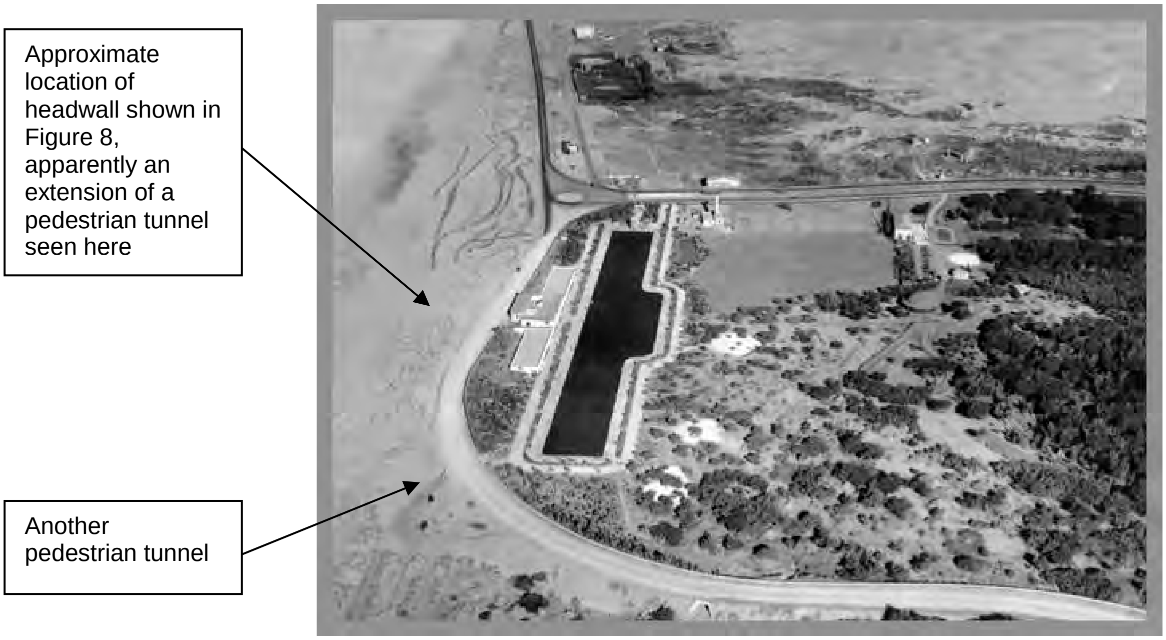



**Figure 7:** The Flieshakker Pool and building constructed in the late 1800s. Note the roadway embankment seaward of the pool, and the headwalls indicating culverts or tunnels, similar to that shown in Picture Headwall. photo date 1925 Source: James Smith, San Francisco City Guides, http://www.sfcityguides.org/public_guidelines.html?article=290&submitted=TRUE&srch_t

**Figure 8:** The inscription says “1926 200’ EXTENSION”. This structure, apparently constructed in the early 1900s has just recently emerged from the eroding fill at the south Ocean Beach “erosion hotspot.” A parking lot previously extended seaward on fill covering this structure. This headwall location corresponds to a location just north of the building in Figure 7, and about 200’ seaward, beyond the roadway and in the beach. Photo Battalio April 2012

**Figure 9:** South Ocean Beach: Top: Post construction in 1990s. Bottom: eroded in early 2000s. The wood posts (top photo) were removed by erosion, and were replaced in the parking remaining parking lot (bottom photo). Photographs, Bob Battalio.

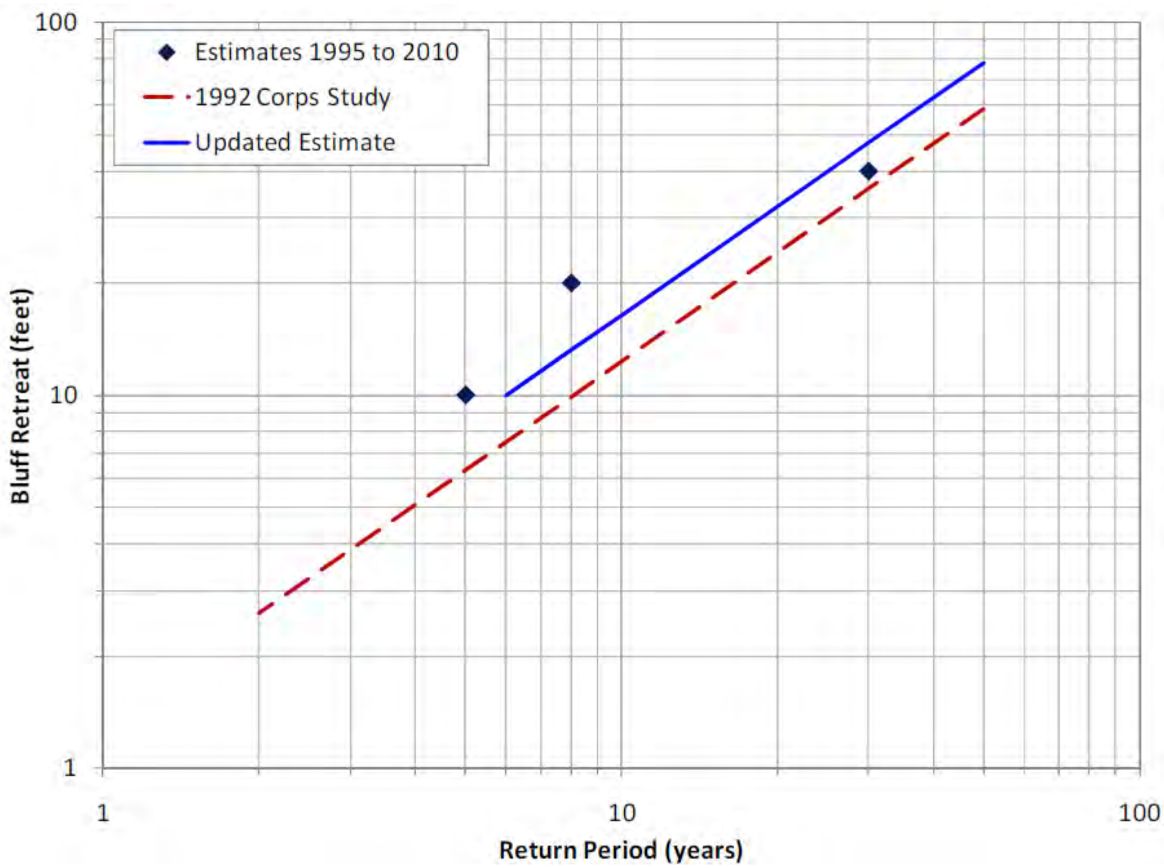



**Figure 10:** Bluff crest retreat distances vs Return Period for South Ocean Beach. Source, Moffatt & Nichol Engineers.

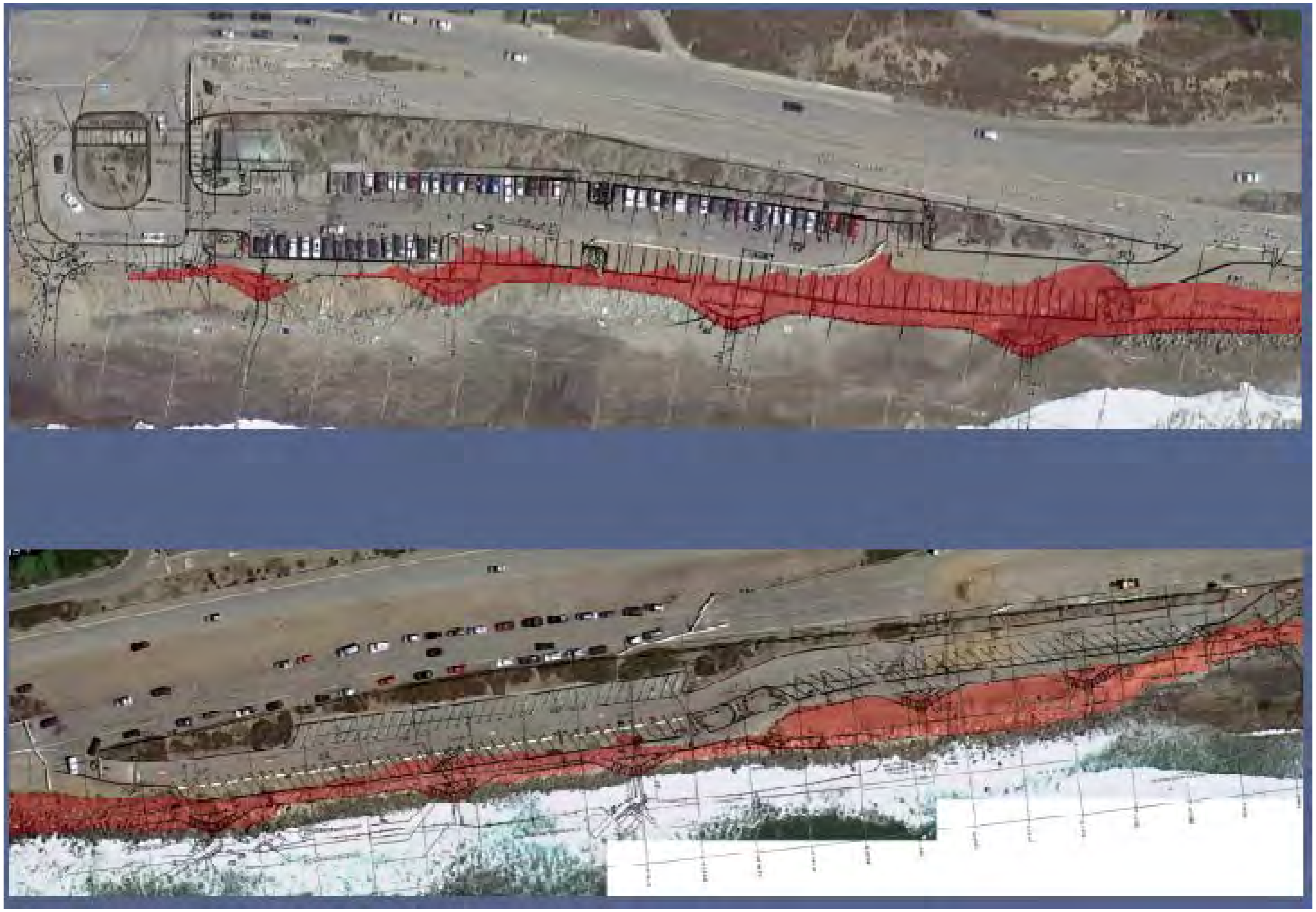


**Figure 11:** Bluff top recession between 2000 and 2010 for South Ocean Beach. Source, Moffatt and Nichol Engineers.

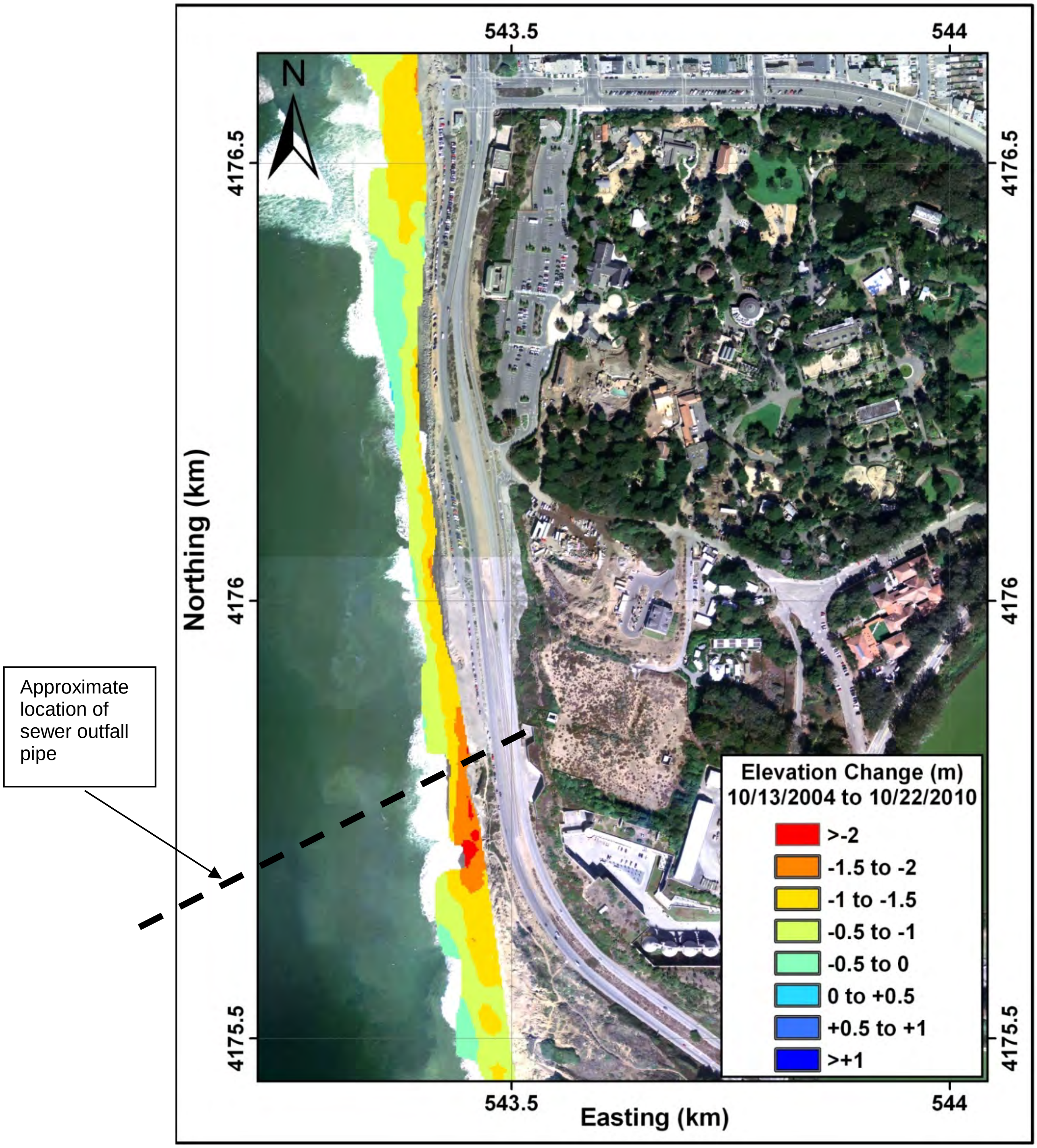



**Figure 12:** Beach elevation change between surveys in 2004 and 2010. The area of 6 meter lowering is where a rock revetment was constructed in 2010, and where the bypass roadway was constructed in 2004. Source: USGS, 2010; provided by Frank Falice, SF DPW. Modified to locate outfall.


**Figure 13:** The sewer outfall pipe crosses SOB in front of the sewer plant. The top of the outfall junction structure has been visible in the winter since the late 1990s. It is located just south of the erosion hotspot where sand was placed in early 2000s, and just north of the location of the emergency revetment constructed in 2010. Photographs, April 30, 2013 Bob Battalio.



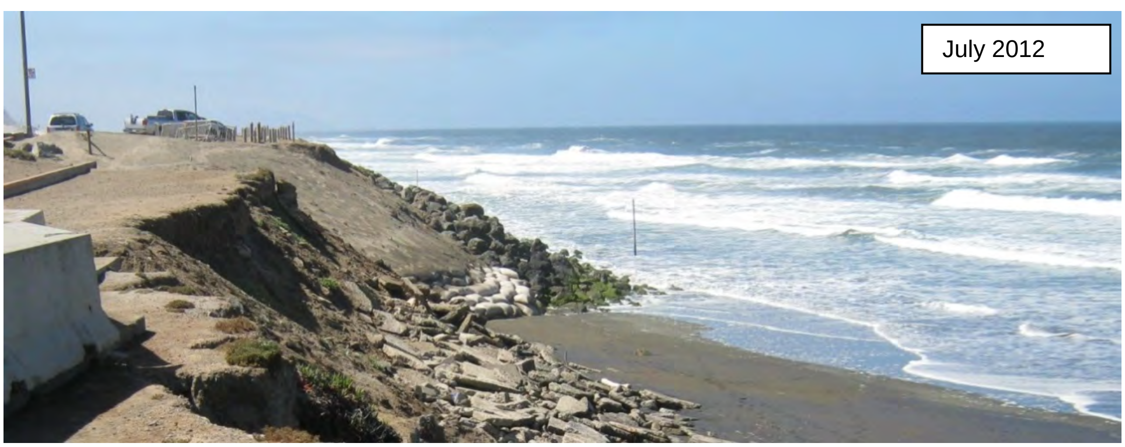



**Figure 14:** Photographs showing change in beach elevation at SOB in early 2012.
Source: Moffatt and Nichol Engineers.

**Figure 15:** Sand placement in the form of a sacrificial sand berm at SOB in 1999-2000. The location is between the SWOO and the south Parking lot, with some sand placed south of the SWOO. The SWOO is under the quarry stone seen in the middle picture. The south parking lot is adjacent to the concrete rubble in the lower picture and behind the construction equipment in the upper picture. Photographs, Bob Battalio.

**Figure 16:** Photo of sandbag structure location after sand fill placed as part of Sand Backpassing project in September 2012.
Source: Moffatt and Nichol

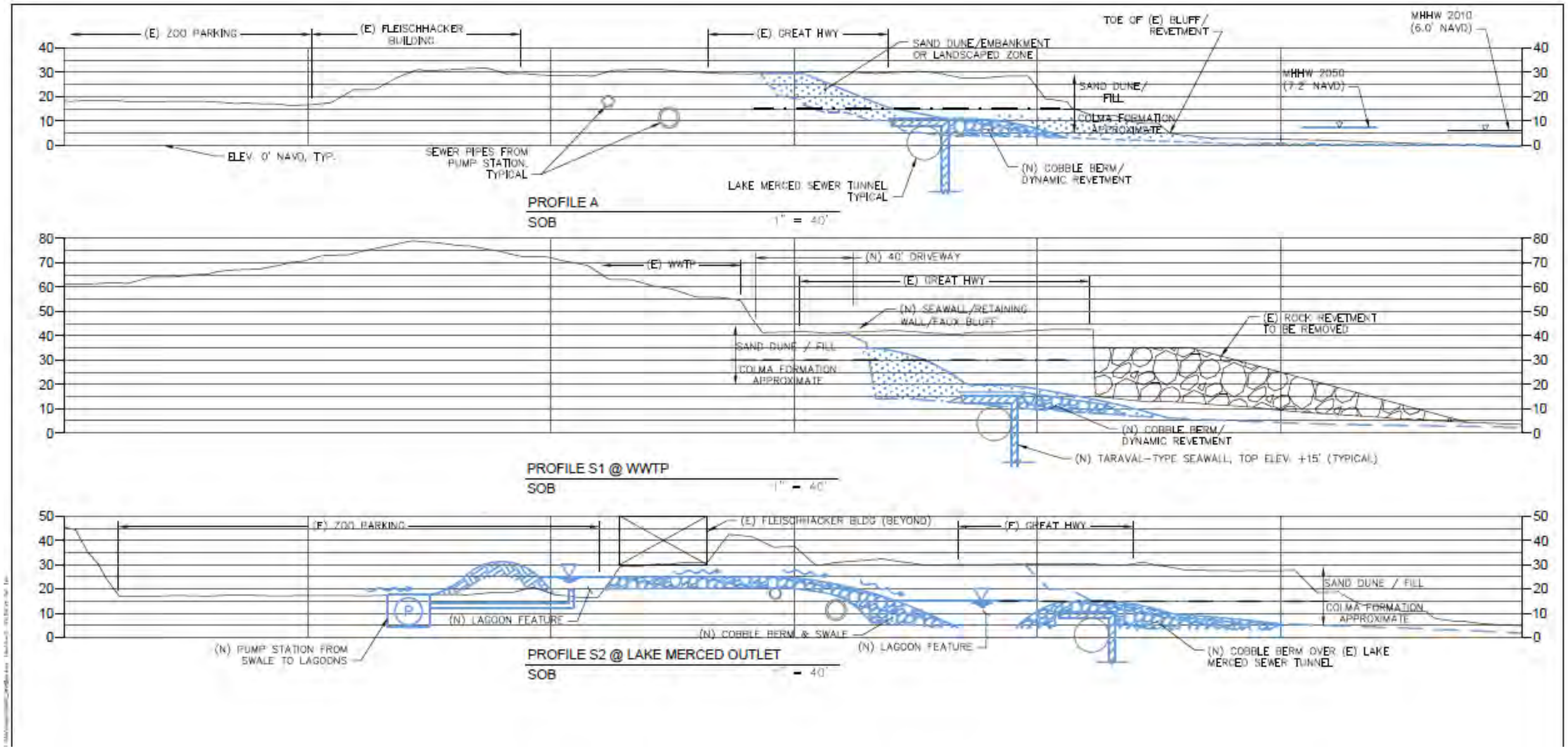
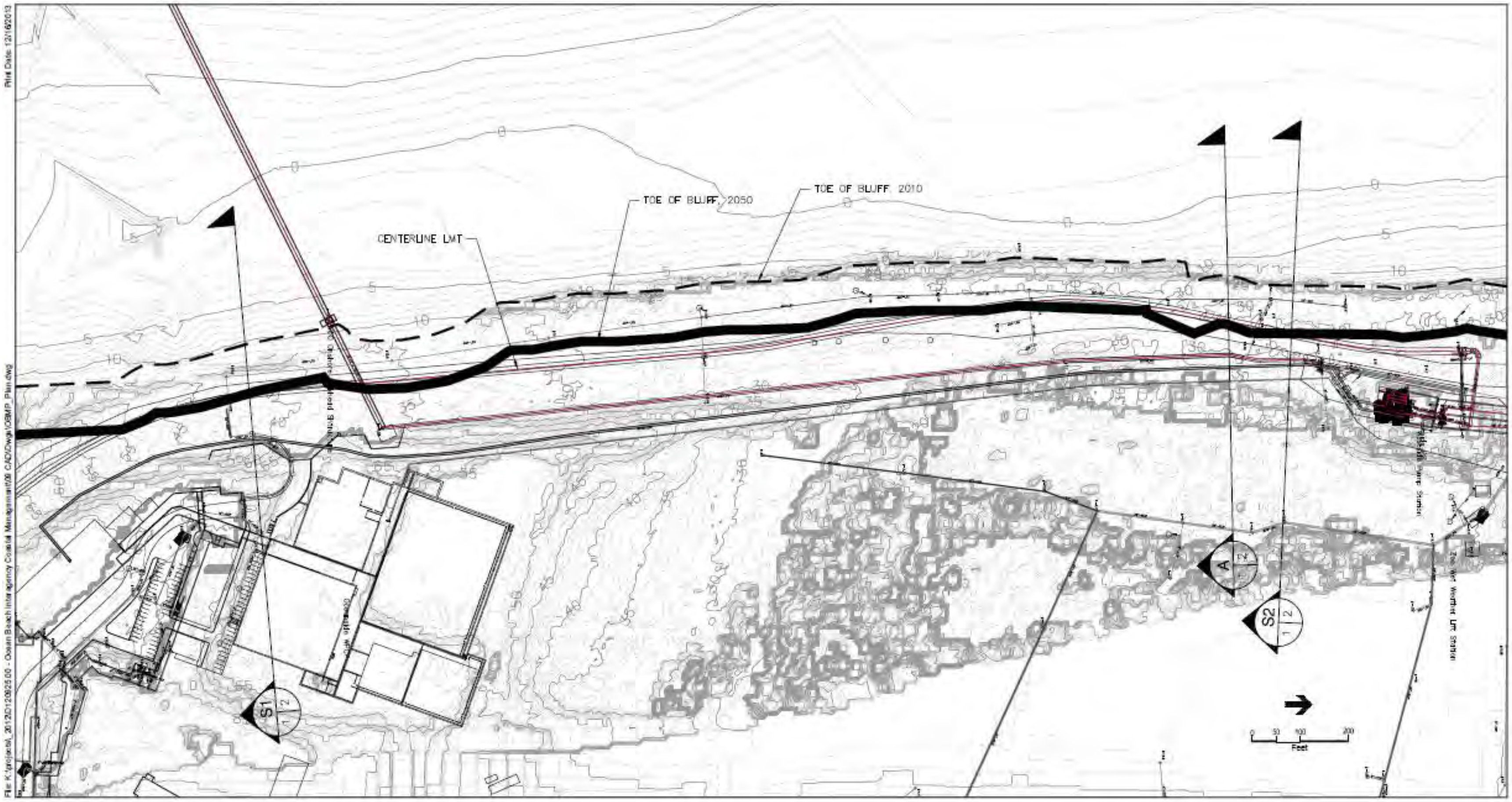



**Figure 17:** Estimated future profiles at SOB developed for the OBMP. Profiles are located in Figure 18.




Ocean Beach . D120925.00

**Figure 18:** Estimated future coastal bluff location at SOB developed for the OBMP. Profiles are shown in Figure 17.

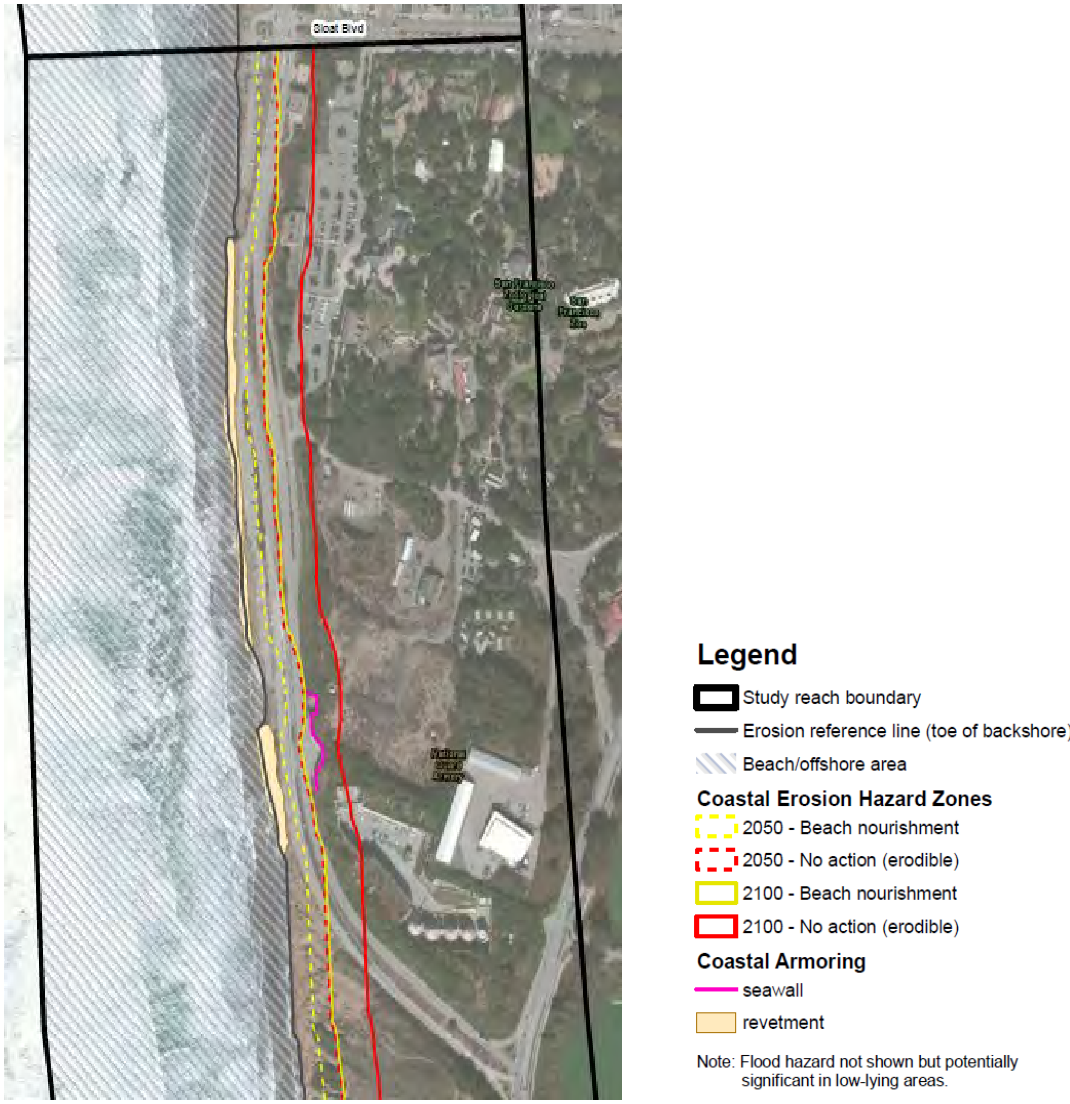



**Figure 19:** Coastal Hazard Zones from RSMP. Note that the lines do not inlcude the added setback restuling from removal of the armoring (40 feet).

## Estimated Coastal Erosion Hazard Zone - 2100

PWA developed the first State-wide assessment of coastal hazards zones in California in 2008 with funding from the State of California in cooperation with the Pacific Institute.

Note: First order approximate estimate for State-wide vulnerability assessment, not for general or local use. Derived from Source: Estimated Erosion with Sea Level Rise PWA, 2009 http://www.pwa-ltd.com/about/about_news.html#OPC_Report

See also:

Revell, David L., Robert Battalio, Brian Spear, Peter Ruggiero and Justin Vandever (2011) A methodology for predicting future coastal hazards due to sea-level rise on the California Coast, Climatic Change, 2011, Volume 109, Supplement 1, Pages 251-276.

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Pacific Institute

http://www.pacinst.org/reports/sea_level_rise/maps/

FEMA Discovery Map

http://www.bakeraecom.com/wp-content/upLoads/2010/02/SF_Discovery_Map_v1a1.pdf


**Figure 20:** Coastal Erosion hazard zones for Ocean Beach, CA developed for the State of California.


**Figure 21:** Excerpt from the CCAMP Discovery Map for San Francisco. The erosion hazard limits are the same as those in Figure 18, and developed for the State of California in 2008, and attained via the Pacific Institute by FEMA and their contractors – Baker-AECOM.

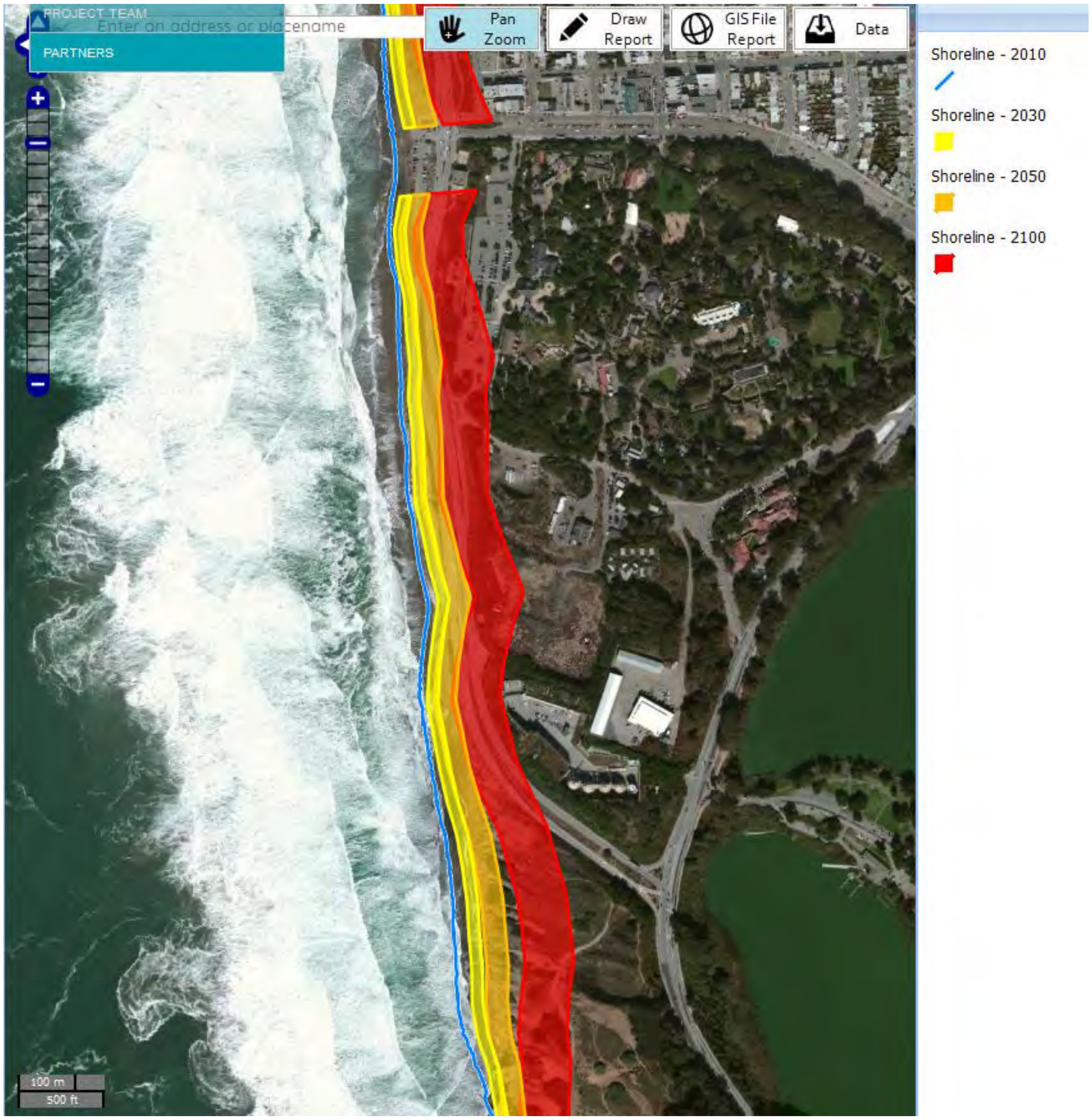


**Figure 22:** Projected future erosion for SOB by OCOF for 50cm (2050) and 150cm (2100).

**Table 3: Summary of Reference Water Levels Considered for Open Coast Inundation Mapping**

| Reference Water Level | Parameter | Typical Elevation During Storms | Period of Inundation |
|---|---|---|---|
| Stillwater Level | SWL | 5 to 8 feet NAVD | Hours |
| Dynamic Water Level | DWL | 12 to 20 feet NAVD | Minutes |
| Total Water Level | TWL | 15 to 30 feet NAVD | Seconds |

**Figure 23:** Dynamic water level extents (blue shading) for 12" of sea level rise (left panel) and 66" of sea level rise (right panel), as depicted in the SFPUC coastal hazard maps (AECOM, 2014). Wave runup and erosion are not included in these hazard maps. Table 3 from the Technical Memorandum is included, above. The Maps show DWL, and do not include wave runup or erosion.


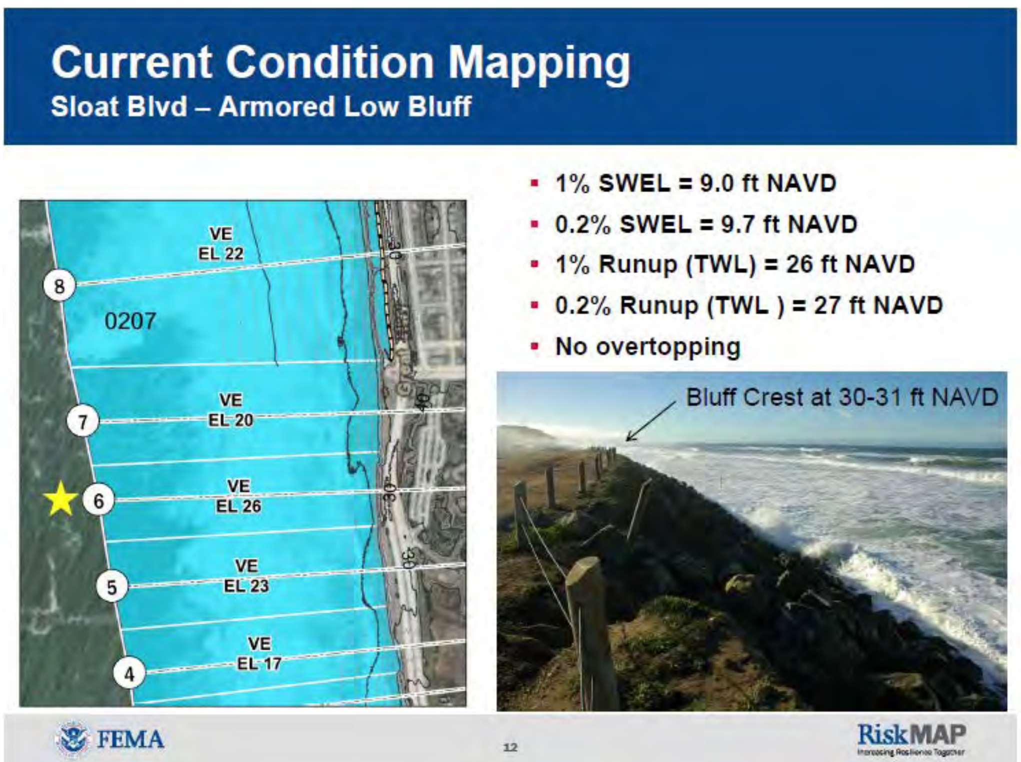


**Figure 24:** TOP: FEMA provisional map of coastal flood hazard (100-year return period) for South Ocean Beach study area. Note that the extent of wave runup reaches the southbound lanes of the roadway. Source: FEMA, 2013 and Google Earth Pro. BOTTOM: Slide from presentation (FEMA, 2014) indicating that the 100-year TWL does not extend above the bluff top. These results do not appear to include erosion during the storm but rather assume that the shore does not respond to the event. The results do not include sea level rise or future erosion. NOTE: These are provisional results and the actual maps may show different hazards when finalized.

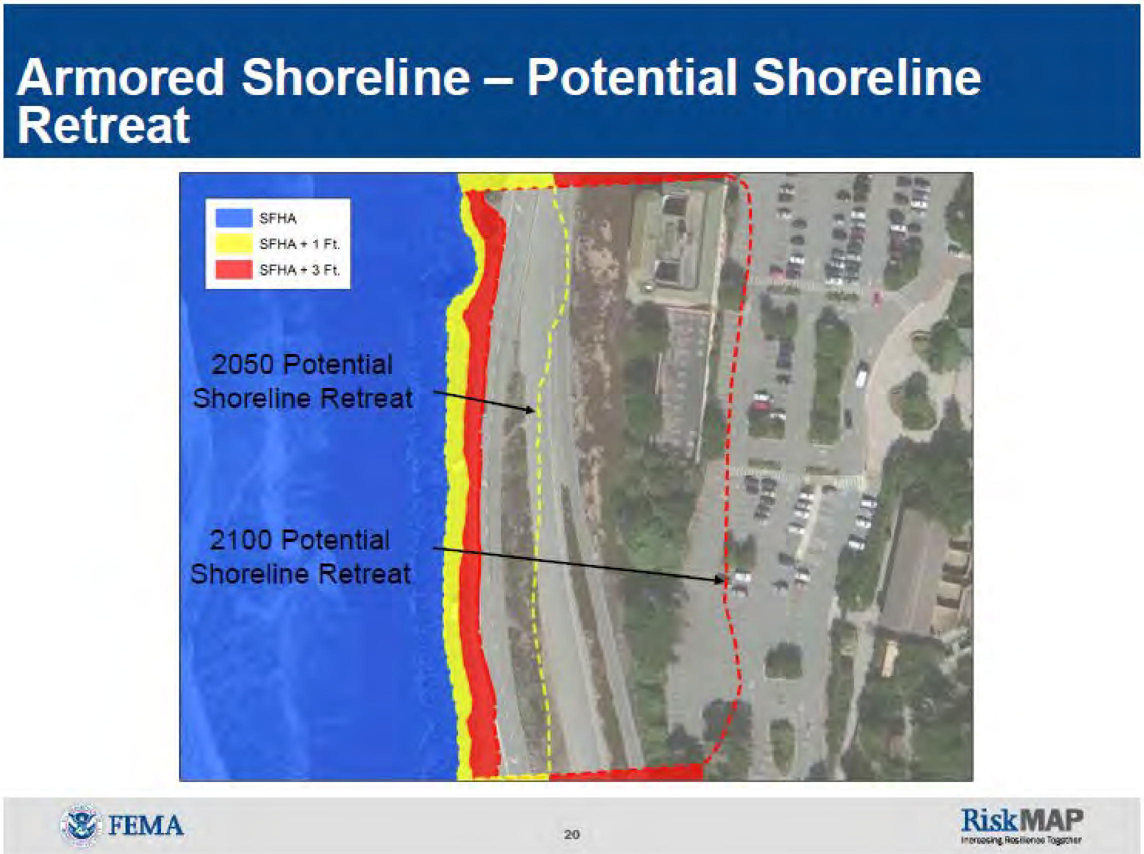


**Figure 25.** Potential shore retreat based on coastal erosion modeling for existing conditions, and sea level rise of 1 foot and 3 feet. FEMA, 2014. http://www.floods.org/Files/Conf2014_ppts/E7_Curtis.pdf

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# APPENDIX 2

## South Ocean Beach Shore Recession Estimates, Regional Sediment Management Plan

Prepared by
ESA PWA
January 2014

# Introduction

ESA PWA is working with a team of consultants and agencies to develop the San Francisco Littoral Cell, Regional Coastal Sediment Management Plan. The plan includes estimates of shore recession under a range of adaptation strategies. The purpose of this Appendix is to present the calculated erosion amounts for South Ocean Beach (SOB).

The Coastal Regional Sediment Management Plan (RSMP) is being developed for the Coastal Sediment Management Workgroup, led by the US Army Corps of Engineers and the State of California (http://dbw.ca.gov/csmw/default.aspx, last visited December, 2013). The San Francisco Littoral Cell is defined as the Pacific coast of San Francisco, Daly City and Pacifica, from the Golden Gate on the north to Point San Pedro on the south (Figure 1). The 17 mile shore was divided into 16 reaches, including North, Middle and South Ocean Beach.

Erosion was projected at South Ocean Beach (SOB) based on historic erosion rates, sea level rise, and a range of candidate adaptation actions such as beach nourishment (sand placement to widen the beach and temporarily blunt shore recession) and armoring, as well as the “no action” of allowing erosion to progress. Many of the assumptions used in the Regional Sediment Management Plan (RSMP) development were similar to those used in the Ocean Beach Master Plan (OBMP) analysis, except as follows:

- Sea Level Rise. The impact to shore recession by sea level rise is greater for the RSMP than the OBMP, owing to the use of a higher rate of rise.
  - A higher rate of rise was modeled, amounting to 1.6 feet (19 inches; based on the “high curve” from USACE 2011 guidance) for the RSMP vs. 1.2 feet (14 inches; based on the OPC 2011 interim guidance) for the OBMP.
  - A steeper shore face slope of 1:50 was used for the RSMP than the 1:60 slope used for the OBMP.
  - A “Bruun Rule” transgression would be 80 feet (RSMP) vs. 70 feet (OBMP)
- Bluff Recession: Bluff recession was modeled differently.
  - Bluff Recession Calculation: For the RSMP bluff recession due to a severe storm / winter season was added to the shore recession to get a larger bluff recession. The storm recession was based on total water level (TWL) exceedence of the back beach – toe of bluff junction elevation, as calculated in the state wide assessment of sea level rise on coastal hazards for the Ocean Protection Council (Heberger et al, 2009; Revell et al 2011). The potential storm recession was reduced for wider beaches and increased for narrower beaches, in order to assist in evaluating adaptation alternatives. In contrast, the OBMP considered the bluff to move the same as the shore line, and essentially assume a steady profile and Bruun transgression.
  - Initial Bluff Position: Both the RSMP and the OBMP assumed that the nourished bluff face would have a flatter slope than the existing face, and that the top would be about 30’ landward of the toe. However, the OBMP imposed a 40 feet recession of the bluff toe and face due to removal of armoring.

- Beach Nourishment: The RSMP adopted the beach nourishment described in the OBMP, amounting to 500,000 cubic yards placed every 20 years in the form of a 50’ wider bluff and 50’ wider beach. Both studies included two placements by 2050, however, the timing was slightly different:
  - RSMP: Sand placement at 2030, 2050, 2070, 2090;
  - OBMP: Sand placement at 2020, 2040, 2060, 2080, 2090.
- Results: The results of the two plans are generally as follows:
  - RSMP: A net recession of 54 feet of the bluff toe by 2050, consisting of about half long term recession and half storm erosion allowance. Accounting for the additional bluff recession of 40 feet expected to result after removal of existing rubble and armor increases the recession to 94 feet.
  - OBMP: A net recession of 110 feet, including the 40 adjustment for armor removal.


SOURCE: ESA PWA 2012 (Figure, Reaches)

San Francisco Littoral Cell Coastal Regional Sediment Management Plan 211658.00
**Figure ?**
Study Reaches

**Figure 1:** Study area and shore reaches, San Francisco Littoral Cell, Coastal Regional Sediment Management Plan

# Description of RSMP Methods

A quantified conceptual model was developed for assessing the ability of management actions to address coastal erosion along the San Francisco Littoral Cell coastline.

## RSMP Objectives

The purpose of this exercise was to develop an erosion projection model with the following attributes:

1. Simple, transparent methods.
2. Ability to differentiate between coastal management alternatives.
3. Automated process that can be efficiently applied to multiple reaches while still being flexible enough to address unique situations and exceptions.
4. Ability to incorporate impact of sea level rise.
5. Use historic erosion trends and shoreform characteristics specific to each study reach.
6. Output a set of useful, quantified results that can be input to an economic model.

## Quantified Conceptual Model

This model tracks the shoreline location, backshore location, and beach width over time. For beaches backed by dunes or structures the backshore location represents the toe of the dune or structure. For reaches backed by bluffs the backshore location is the toe of the bluff. For each 1-year time step the shoreline movement and backshore erosion are calculated using relationships described in the following sections.

### Beach Width

The beach width is the distance between the shoreline[1] and the backshore. A starting beach width was estimated for each reach by taking the average distance between the mean high water line[2] and the backshore location as observed in recent orthorectified aerial imagery. Subsequent beach widths are calculated based on the relative movement of the shoreline and backshore. If the shoreline erodes more quickly than the backshore then the beach narrows, and vice versa.

### Shoreline Movement

Three components contribute to shoreline movement in this quantified conceptual model: landward movement due to sea level rise (SLR), shoreline erosion caused by other coastal processes (e.g. waves, wind, changes in sediment supply), and seaward movement of the shore due to sand placement activities:

$$Shoreline\ Movement = SLR\ transgression + Erosion_{shoreline} + Beach\ Nourishment$$

[1] Assumed to be located at Mean High Water.
[2] The mean high water line was extracted from a 2010 USGS LiDAR digital elevation model.

The impact of sea level rise on shoreline movement is incorporated by assuming that the shoreline will move inland based on the shape of the beach profile and the amount of sea level rise:

$$Sea\ Level\ Rise\ Transgression = \ shoreface\ slope * increase\ in\ sea\ level$$

The shoreface slope is the slope between the backshore toe location out to the estimated depth of closure. At least one representative profile was analyzed for each reach to calculate this value. The profile data came from a digital elevation model developed by USGS using recent high resolution bathymetric and topographic surveys. The sea level rise curve used in this analysis is based on the “High” sea level rise scenario described in recent U.S. Army Corps of Engineers guidance (USACE, 2011). This curve predicts 1.6 feet of sea level rise by 2050 and 5 feet by 2100 (relative to 2000). As the rate of sea level rise increases towards the end of the century, the contribution of sea level rise to shoreline movement will likely be greater than erosion.

Shoreline erosion is specified as a function of beach width. When the beach is nourished the beach widens and the shoreline moves seaward. In this unusually wide beach configuration the shoreline erosion rate is expected to increase (Dean 2002). If the shoreline moves inland (either due to sea level rise or erosion), the beach narrows and shoreline erosion decreases. When the beach width goes to zero, inland shoreline movement stops (but bluff erosion accelerates, as described in the next section) until the beach widens again. An exponential empirical relationship was established between shoreline erosion rate and beach width for each reach that reflects this conceptual model.

$$E_{shoreline}(t) = \min(E_{shoreline,historic} * e^{a\left(\frac{BW(t)}{BW_{ambient}}-1\right)}, E_{shoreline,max})$$

*Where:*

| | |
|---|---|
| $E_{shoreline}$ (t) | = Shoreline erosion at time t |
| $E_{shoreline,\ historic}$ | = Historic shoreline erosion rate |
| $E_{shoreline,max}$ | = Maximum shoreline erosion rate |
| BW (t) | = Beach width at time t |
| $BW_{ambient}$ | = “Ambient” beach width |
| a | = decay calibration parameter that determines how responsive the erosion rate is to beach width |

Similar exponential relationships have been proposed for existing sand placement projects (Dean, 2002). One assumption is that sand placements are self-similar. Existing studies have shown that an exponential relationship may overestimate the erosion rates (Dette et al 1994). Since very little data exists related to response of shoreline erosion to sand placement, we assume a decay parameter equal to 1. When a reef is implemented, this decay parameter is decreased to 0.5, to account for the reef wave sheltering effect.

An example of this relationship is plotted in Figure 2. When the beach width is equal to the ambient beach width then the erosion rate is equal to the long term historic erosion rate. The equation is capped with a maximum erosion rate to acknowledge that there is a limit to how quickly sand can be removed from the beach. A high value of the calibration parameter (a) leads to erosion rates being more responsive to beach width. A value of 0 would result in a constant erosion rate equal to the historic erosion rate, regardless of beach width.

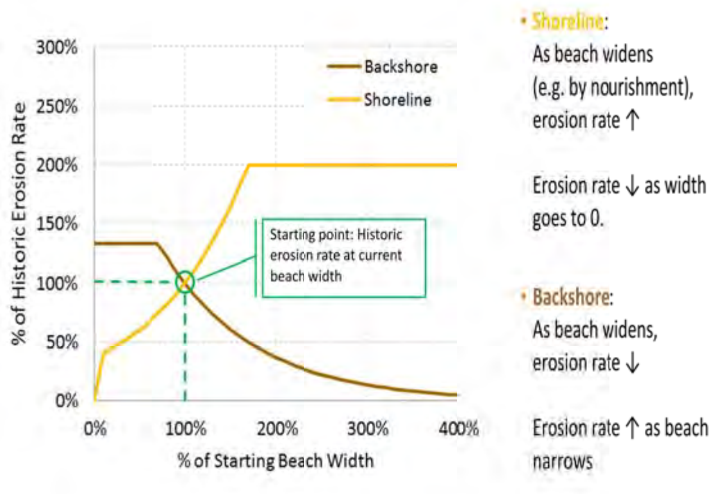


**Figure 2: Example of Empirical Relationships between Erosion Rate and Beach Width.** The graph is in terms of relative beach width and erosion rate, with 100% indicating equal to existing width and historic erosion rate.

## Backshore Erosion

The backshore location is tracked using a similar empirical relationship as the shoreline (see below). However, bluff erosion is expected to have the opposite response to beach width: when the beach width is wide the backshore is expected to erode more slowly than if the beach is narrow, due to the additional protection from waves provided by the wide beach. When the beach becomes narrow, the backshore is expected to erode more quickly due to more frequent wave contact at the backshore toe. Once again, the erosion rate is capped by the maximum backshore erosion rate to acknowledge that the backshore (bluff/cliffs in particular) should have a maximum erosion rate which is a function of geology. Soft, weak bluffs would have a much higher maximum erosion rate than hard, impervious rock.

$$E_{backshore}(t) = \min(E_{backshore,historic} * e^{-b\left(\frac{BW(t)}{BW_{ambient}}-1\right)}, E_{backshore,max})$$

*Where:*

| | |
|---|---|
| $E_{backshore}$ (t) | = Backshore erosion at time t |
| $E_{backshore,\ historic}$ | = Historic backshore erosion rate |
| $E_{backshore,max}$ | = Maximum backshore erosion rate |
| BW (t) | = Beach width at time t |
| $BW_{ambient}$ | = “Ambient” beach width |
| b | = calibration parameter that determines how responsive the erosion rate is to beach width |

Again, we assume a decay parameter (b) equal to 1. This value could be modified in more detailed studies with detailed information about how the backshore responds to narrower or wider beaches. Most reaches were relatively insensitive to this parameter.

It is important to note that this model does not address backshore erosion due to terrestrial processes.

# Model Application

## Management Actions

The quantified conceptual model described above was used to analyze five types of management actions. Up to four of these scenarios were assessed for each study area. For many of the reaches a scenario may combine multiple management actions for a “hybrid” approach. Each of the potential management actions and the associated model input parameters are described below. These descriptions focus on the physical implications of each management action rather than economic implications (which will be discussed in a later memo).

### No Action, Hold the Line

This action maintains existing coastal protection infrastructure (seawalls, revetments) where it currently exists. With continued shoreline erosion and the additional impact of sea level rise, the beach will continue to narrow. This action is implemented by setting backshore erosion rate to zero. Some hazard still remains behind the structures due to high velocity flooding and potential for failure during a major (i.e. 100-year) erosion event. For the purposes of this model, presence of a structure is assumed to reduce the erosion of a 100-year erosion event by 50%.

### No Action, Allow Erosion

The shoreline and backshore are allowed to erode at a natural rate. As sea level rises, the shoreline erosion is predicted to occur at a higher rate than backshore erosion, resulting in a beach that narrows over time, depending on the maximum permitted bluff erosion rate.

### Managed Retreat

From a physical modeling perspective, this management action is very similar to *No Action, Allow Erosion*. One additional input parameter is the “permitted erosion distance,” which caps backshore movement to a set value in situations where the inland movement is limited.

### Sand placement

Sand placement is implemented in the model by moving the shoreline seaward by the sand placement width (50 feet at SOB). Sand placements are triggered at the beginning of the model and every subsequent time the beach reaches a “minimum beach width”, or at a set frequency: For SOB, the sand placement was set to occur every 20 years. A “hold the line” option can still be specified for the backshore but was not used for SOB for the period to 2050.

### Sand placement with Offshore Reefs

The sand placement component of this management option is treated in the same manner as described in *Sand placement,* above. Offshore reefs are implemented in the model by adjusting the empirical relationship between erosion rate and beach width, historic erosion rate, and ambient beach width. Offshore reefs have successfully demonstrated the ability to widen the beach through formation of a salient (widening of the beach) along the beach behind the reef (Mead 2009, Black 2000). The beach reaches a new, wider equilibrium. This is implemented in the conceptual model by increasing the “ambient beach width” in the empirical relationships described previously. Another benefit of offshore reefs is the wave sheltering effect. Ignoring sea level rise, the future erosion rates are expected to decrease because of the added protection provided through wave dissipation at the reef. This is implemented in the model by decreasing the erosion rates in the empirical relationships described previously.

Limited data exist to quantify the extent to which offshore reefs would change shoreline movement rates, especially with the contribution of sea level rise. In general, a consistent approach was chosen for all reaches in the absence of robust data availability.

Figure 3 shows the modeling results for an example location in terms of shore and back shore locations over time.

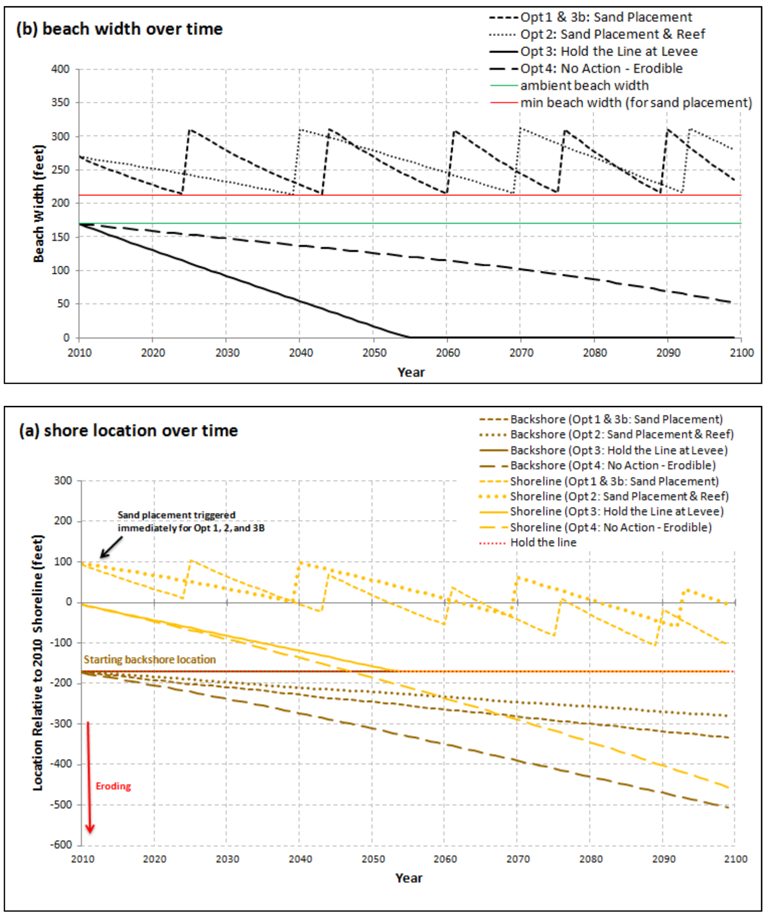


**Figure 3: Example of shoreline, backshore, and beach width over time. (a)** For the hold-the-line scenario, the shoreline erodes until the beach width reaches zero (see **b**). If sand placement is implemented at the start of the model, the shore erodes more rapidly at first due to the un-naturally wide beach. In this example, backshore erosion is allowed to occur for both sand placement scenarios. Sand placement in combination with an offshore reef results in less rapid erosion initially because the beach is more stable in its wider state. Sand placements are triggered when the minimum beach width is reached (see **b**). The sand placement is triggered later in time with an offshore reef. Additionally, backshore erosion occurs more slowly with a reef in place. **(b)** For the hold-the-line scenario, the beach narrows until there is no longer a beach. When sand placement is implemented, the beach narrows more quickly at first, due to the unnaturally wide beach. With a reef the beach narrows less quickly and reaches the trigger beach width later in time.

## Hazard Zone Mapping

The model output was mapped, with the landward limit being the top of the wave cut bluff. The erosion hazard was measured from the reference toe line (backshore toe location in year 2010) to the inland extent of the erosion hazard. This value is calculated from the backshore erosion, storm-induced erosion, and offset from the toe to the top of wave cut bluff. Figure 4 shows the hazard zones for SOB for a range of adaptation scenarios for the years 2050 and 2100.

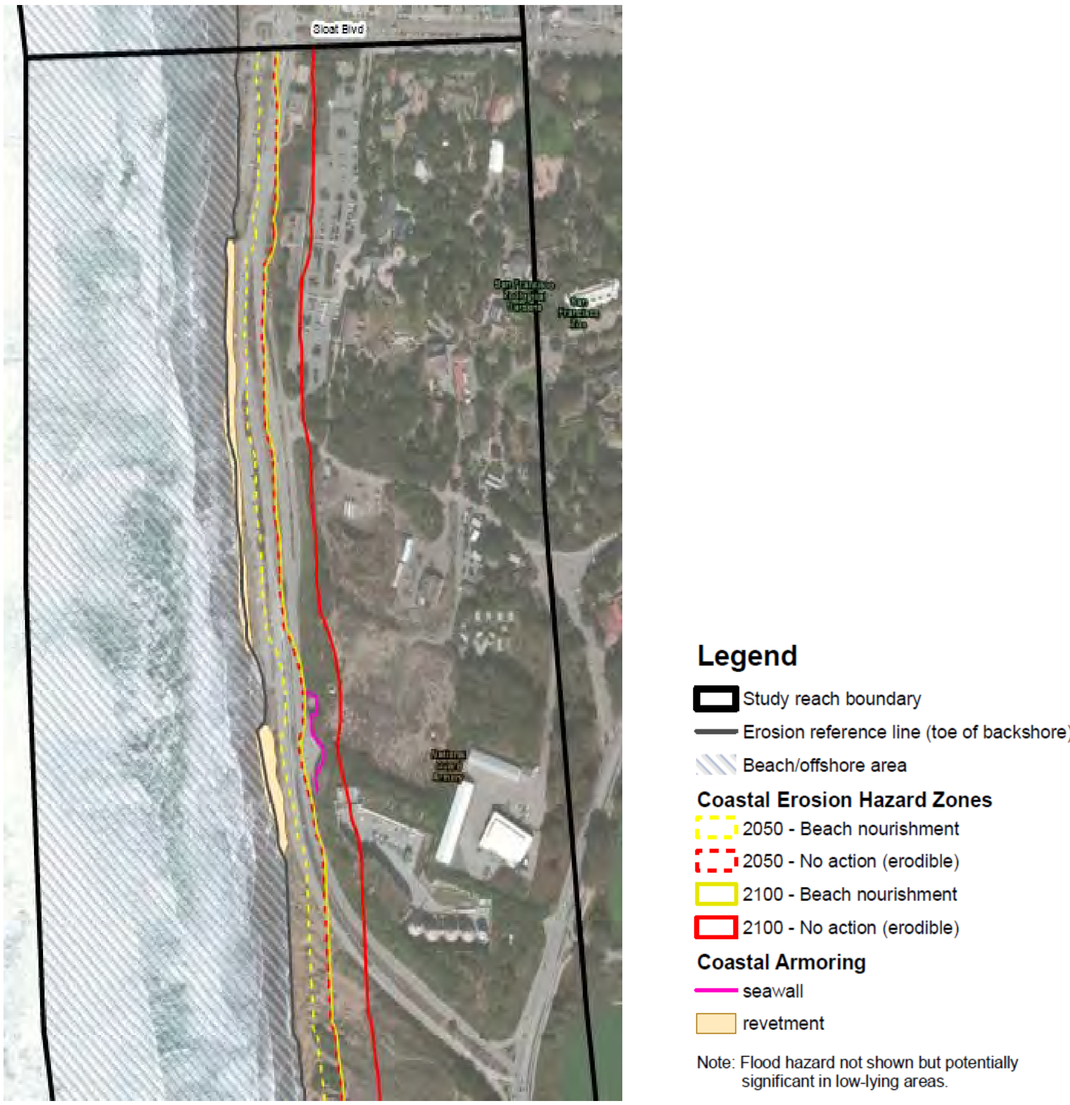


**Figure 4: Coastal Hazard Zones from RSMP.** Note that the lines do not inlcude the added setback restuling from removal of the armoring (40 feet).

# Example Application

The following figures explain the model application in general, with results of model application at south Ocean Beach. Figures 5 through 9 are for SOB.

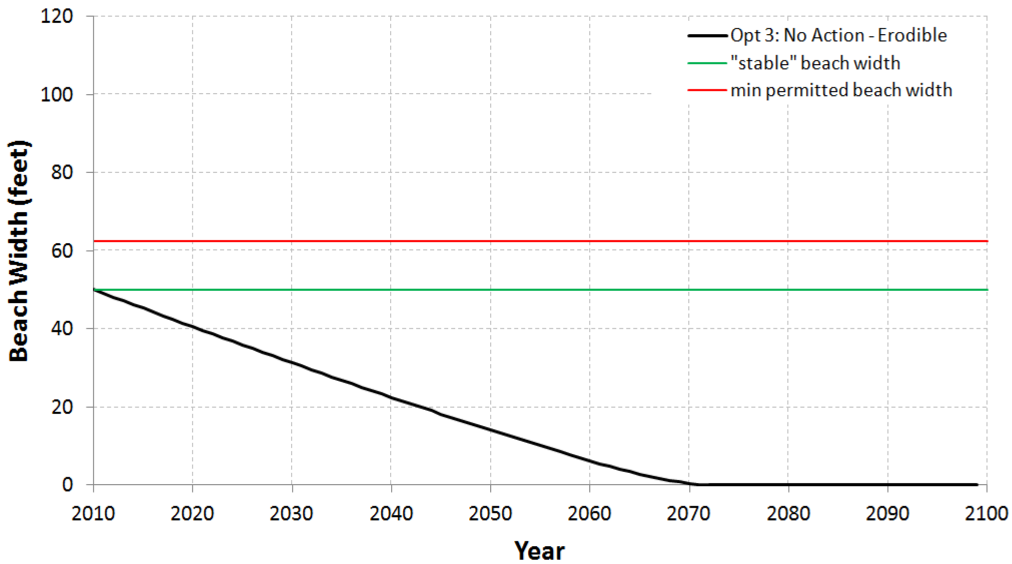


**Figure 5: Parameters selected for SOB.** Note that the black line indicates the assumed 50-foot wide beach in 2010 and the loss of the beach by 2070. This rate of loss is less than projected by the Bruun rule because of the presumed slow of shore erosion as the beach width approaches zero.

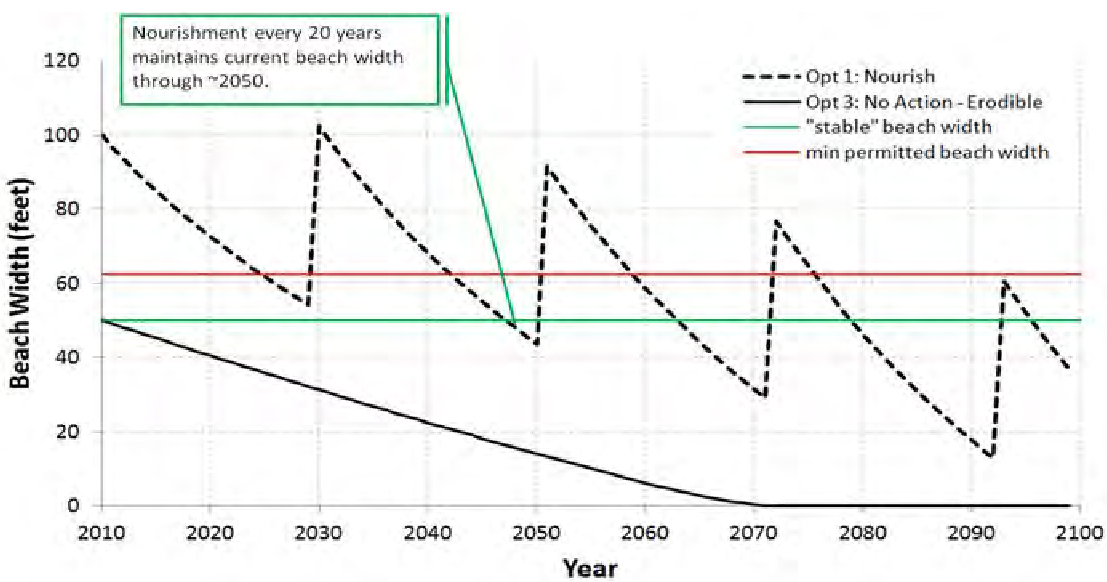


**Figure 6: Beach widths.** Option 1 is similar to the OBMP with 50' of widening every 20 years. Abrupt increases in beach width indicate sand placement. (dashed line). Note that the beach width becomes narrower over time, but maintains a widened beach through 2050.

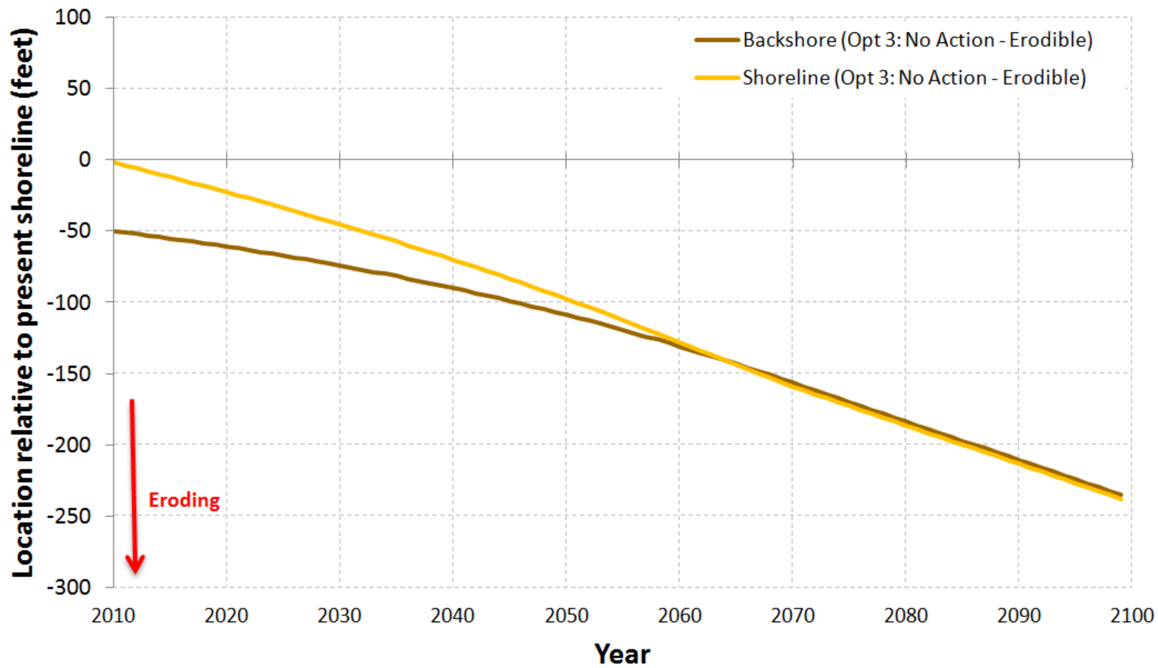


**Figure 7:** Shore location over time for no action showing that the shore and bluff retreat as the beach width decreases, for the not action case, with the assumption that the bluff and beach are erodible.

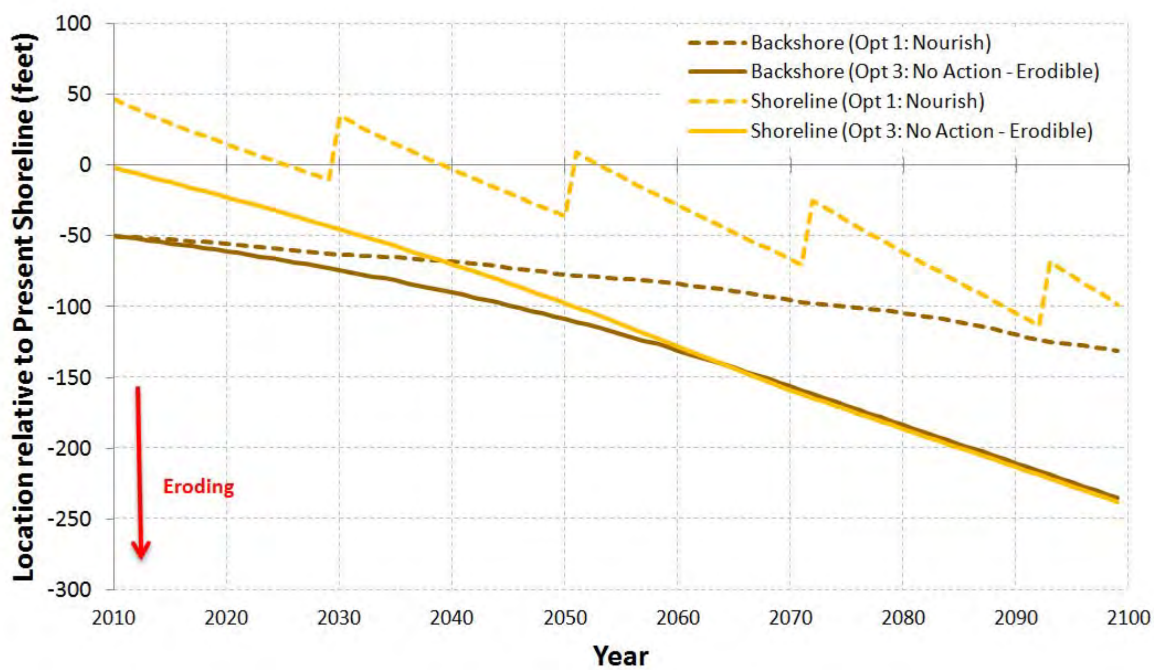


**Figure 8:** Shore and backshore locations for no action (same as Figure 7) and Option 1 nourishment (similar to OBMP). Note that the model predicts a wider beach (distance between shore and backshore lines) and less recession (line locations).

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# APPENDIX 3

## Structural Approach to Protecting the Lake Merced Transport Tunnel

# OCEAN BEACH - LAKE MERCED TRANSPORT

## Long-Term LMT Protection Feasibility Study

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February 2015

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# CONTENTS



# ACRONYMS & ABBREVIATIONS

| | |
|---|---|
| CCC | California Coastal Commission |
| cy | Cubic Yards |
| DPW | Department of Public Works |
| EQR | Emergency Quarrystone Revetment |
| LMT | Lake Merced Transport |
| NPS | National Park Service |
| SFPUC | San Francisco Public Utilities Commission |
| SWOO | Southwest Ocean Outfall |

# 1 LONG-TERM LMT PROTECTION CONCEPTS

The purpose of this report is to evaluate the feasibility of the preferred long-term (permanent) shoreline protection measures to protect the Lake Merced Transport (LMT) tunnel. This report builds upon the concept-level evaluation of the stability of the LMT previously submitted as part of the Final Interim Shore Protection and Management Strategy report which described minimum earth cover dimensions related to lateral and vertical resistance.

## 1.1 LOW-PROFILE WALL

The Ocean Beach Master Plan envisioned a concept of protecting the LMT in place with a low-profile structure providing both lateral and vertical restraint; the lateral restraint component would also act to prevent landward bluff migration. The low-profile wall providing lateral restraint to the LMT would be designed to be as low in elevation and as far landward as possible to have a minimum footprint and provide a maximum of potential beach width.

Vertical restraint is necessary due to potentially high buoyant forces acting upon the LMT. This restraint is currently provided by sand cover – the "overburden" – over the LMT. The sand cover provides overburden restraint through the weight of the sand itself and through the inherent strength of the material. The minimum overburden required to resist buoyant forces was recently reevaluated based on tunnel as-built data and updated geotechnical parameters; this study found that buoyant forces "would have to be controlled with at least 6 to 8 feet of sand cover, a hold down structure, or through a combination of cover and restraint." [1]

Currently, the LMT has approximately 30-ft of overburden in Reach 1 where Great Highway is highest, dropping to approximately 20-ft at the start of the Rubble Reach where Great Highway flattens and continuing at approximately 20-ft along the rest of the LMT going northward. The overburden can therefore be potentially reduced by 12-ft to 14-ft along the entire South Ocean Beach shoreline.

As stated above, the minimum overburden provided by 6-ft to 8-ft of sand cover must be provided over the LMT. Alternatives to sand cover – such as replacing the sand cover with concrete cover or incorporating a "hold-down" structure – would need to provide an overburden force that is equivalent to (or greater than) the 6-ft to 8-ft of sand cover. Evaluating whether alternatives to sand cover are feasible requires additional data and analysis which are not available at this time. As a result, this report assumes (and depicts) that the minimum overburden will be provided by this 6-ft to 8-ft sand cover.

Although the Low-Profile Wall can be constructed in various ways, this report will focus on two types/classes of wall that are considered feasible for the proposed project. This determination is the result of past discussions, studies, and evaluations; however it should be noted that other variations may deserve consideration as the project design progresses.

The two types of Low-Profile wall being considered in this report are the Pile Wall and the Soil-Cement Wall. These two are described in more detail below.

### 1.1.1 Pile Wall

This type of wall would be constructed below grade and would be set back a sufficient distance from the bluff face in competent material such that the construction operations will not cause the

[1] Numerical Modeling Studies, Lake Merced Tunnel, Ocean Beach Master Plan; Jacobs Associates; Sept 2014; pg 7.

existing bluff face to fail. There are many types of pile walls, including the following types that have been considered for this project:

*Tangent Pile Wall*

This type of wall does not intersect or overlap, but meets adjacent piles at a single point. Due to expected difficulties in meeting the tight tolerances required for pile alignment control, this type of wall is not considered feasible and has been removed from further consideration.

*Secant Pile Wall*

This type of wall overlaps and intersects adjacent piles. Primary, unreinforced piles are initially drilled and filled with concrete, followed by reinforced secondary piles drilled between and partially cutting into the primary piles. The secondary piles are longer (deeper) and provide the structural load-supporting element while the primary piles act as lagging to transfer loads to the secondary piles.

*Soldier Pile Wall with Lagging*

This type of wall includes initial construction of reinforced cast-in-drilled-hole (CIDH) soldier piles followed by construction of lagging between the piles using jet grouted columns. The soldier piles provide the structural load-supporting element and the lagging transfers the load to the soldier piles. An example of this type of wall is shown in Figure 4.

Construction of pile walls involves several major operations:

- First, drilling and removal of in-situ material and casing of the drilled hole.
- Second, installation of reinforcement to allow the pile to act as a cantilevered structure.
- Third, placement of concrete and extraction of the casing.

**Figure 1 – Typical Pile Wall Examples**

Since the pile wall is constructed below grade in existing subgrade material, it would initially be concealed and not supporting any loads. However, over time as beach recession and lowering occurs, the wall would be exposed and would act as a cantilevered structure supporting a load that increases as the exposed height increases. The pile wall must be designed to support loads due to the maximum exposed height even though it will not need to support this load until the bluff erodes.

With the pile wall initially below ground there is little need for aesthetic considerations. However, as the wall is exposed to view, aesthetic improvements may be desired to make the pile wall more visually appealing or more natural-looking. To accomplish this, a veneer of shotcrete could be applied to the exposed face and finished to create the desired appearance.

Proposed Design

A draft alignment for the pile wall was developed to run parallel to the LMT, offset to provide approximately 10-ft clearance from the tunnel wall as shown in Figure 2. The alignment is intended to be close to the tunnel to maximize the potential beach width while not causing damage to the tunnel during construction. The actual offset distance will ultimately depend on the specific type of pile wall constructed and the geotechnical conditions present along the tunnel.

Cross-sections were developed and are shown in Figure 3 (Soldier Pile Wall shown). The depth of the pile wall will vary slightly based on the anticipated exposed height along the shoreline. The exposed vertical face is assumed extend up to an elevation where the minimum overburden over the LMT is met; this is assumed to result in an approximately 15-ft high pile wall at the northernmost limit, increasing to 20-ft at the southern limit. The pile wall will extend to a depth of 30-ft (or more) below the exposed portion of wall.

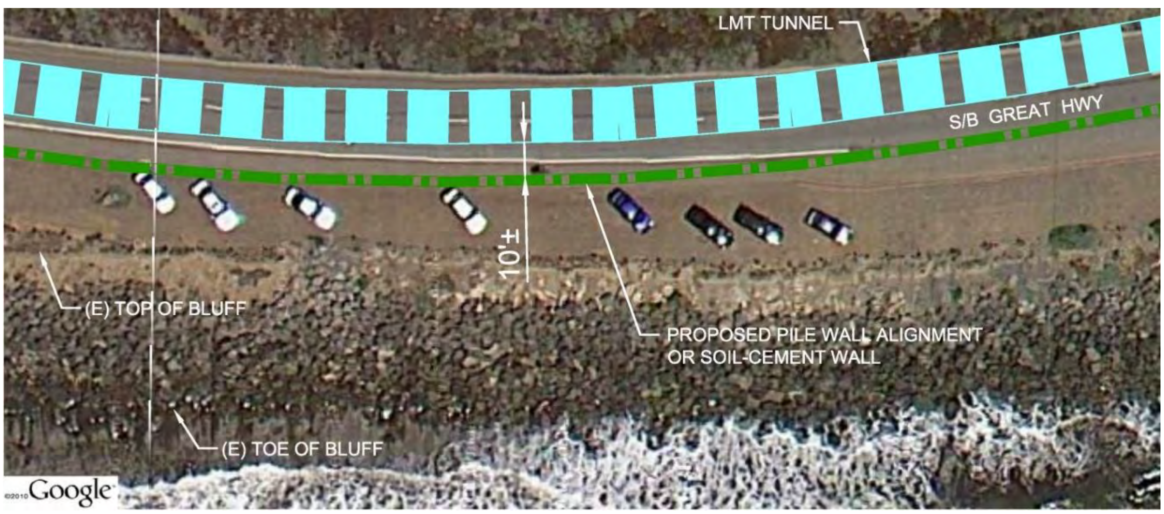


**Figure 2 – Typical Pile Wall Alignment (shown at EQR)**

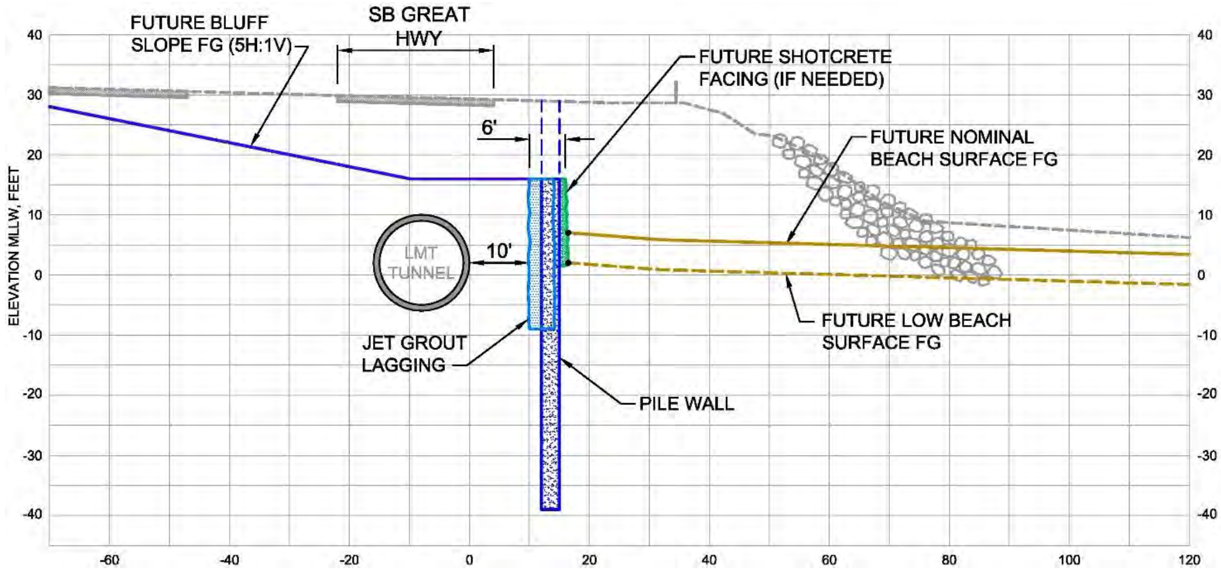


**Figure 3 – Typical Pile Wall Cross-Section (shown at EQR)**

One design option that should be considered is to include a mild batter (10v:1h or 20v:1h) to the pile wall to improve wall stability and minimize deflection. When the wall is eventually exposed, a sloping wall would appear more natural and would also ensure the wall does not attain a concave appearance due to long-term deflection.

Soldier Pile Walls (shown in Figures 3 and 4) are expected to consist of a series of 36-inch (assumed) diameter cast-in-drilled-hole piles at approximately 5-ft center-to-center spacing. The piles are drilled to the desired depth, soil is removed during drilling (disposed of off-site), reinforcing steel is placed in the hole, and the hole is filled with concrete. Once the concrete has attained the specified strength, the space between adjacent piles will be filled by unreinforced jet grout columns to act as lagging, to transfer loads to the soldier piles. The jet grout columns will extend to a depth slightly below the lowest expected beach surface elevation.

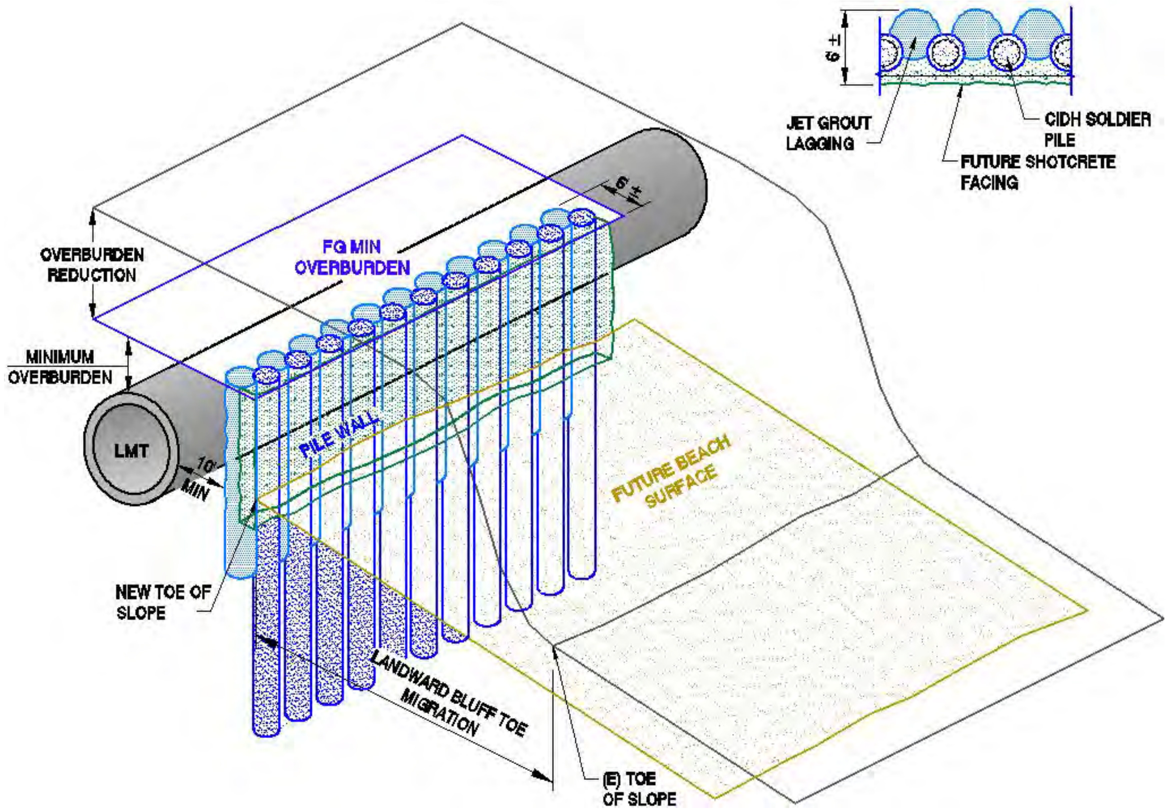


**Figure 4 – Typical Pile Wall Concept**

Secant Pile Walls consist of a series of overlapping cast-in-drilled-hole piles resulting in a continuous wall. The piles are expected to have a diameter of 30- to 36-inches. The piles are drilled to the desired depth, soil is removed during drilling (disposed of off-site), and the drilled hole is filled with concrete. The piles are drilled in two phases: the primary piles are spaced at 1.5 to 1.8 times the pile diameter and are unreinforced; the secondary piles are spaced between the primary piles, cut into (overlap) the primary piles, and are reinforced with steel. The steel-reinforced secondary piles are the primary structural component of the wall, with reinforcement consisting of either a wide-flange steel beam or a rebar cage; the former can be “stabbed” into the wet concrete while the latter must be installed prior to concrete placement. The unreinforced primary piles act as lagging between the reinforced piles; these piles can be shorter in length since they are not acting structurally. Because each pile overlaps the piles on each side, proper alignment and location relative to adjacent piles is very important. A cast-in-place concrete template is typically constructed prior to pile drilling to guide the drill rig and ensure correct pile location.

<u>Construction Considerations</u>

The construction of the pile wall will include heavy construction equipment, including a drill rig, crane(s) for casing and reinforcing steel installation, loaders/backhoes/trucks to manage spoils removed during drilling, and a concrete pump truck.

If jet grout columns are used for lagging, additional equipment and plant will be brought to the site. A significant amount of spoils is expected to be generated as excess fluid returns to the surface via the void space around the drilling rod. This fluid will need to be actively managed and off-hauled as it cannot be reused or recycled, and will require measures to contain and direct excess fluid to a predetermined area for removal.

### 1.1.2 Soil-Cement Wall

This wall is constructed by mechanically mixing the existing in-situ material while injecting a cementitious grout, resulting in a continuous wall consisting of a mixture of cement and soil. This type of wall can be built using various methods: Jet Grouting, Jet Mixing, Cutter Soil Mixing, Deep Soil Mixing and others. Jet Grouting is likely to be the most appropriate method to be used, so the Soil-Cement Wall will be assumed to be a Jet Grout Wall. If necessary, the soil-cement wall could be reinforced with steel H-piles for additional structural capacity. Examples of Soil-Cement Walls are shown in Figure 5.


**Figure 5 – Typical Soil-Cement Wall Examples – Jet Grout shown**

Similar to the pile wall, the soil-cement wall is constructed below grade in existing subgrade material and would initially be concealed. As erosion causes the soil-cement wall to be revealed, the wall acts increasingly as a retaining structure. The soil-cement wall will be designed to support the loads due to the full exposed height even though it will not need to support this loading until the bluff erodes.

Because the soil-cement wall utilizes the existing materials within the wall, the wall's appearance will generally reflect the existing conditions and appear more natural than a wall constructed entirely of concrete. The exposed face will be undulating and irregular, which may also be considered more natural than a rounded face created by a drilled hole filled with concrete.

Proposed Design

The soil-cement wall will have the same alignment, height, and depth as the pile wall; however the soil-cement wall will have a wider cross-section because of two primary reasons: first, because it incorporates in situ material it will have lower strength than pure concrete; second, it is a gravity wall instead of a cantilevered structure. Cantilevered structure must resist high moment loads therefore they require reinforcing steel. In contrast, gravity walls are subjected to high shear loads and minimal moment loads and thus require no reinforcing steel; the increased width of a gravity wall will resist the high shear loads. Figure 6 shows a conceptual sketch of the proposed soil-cement wall.

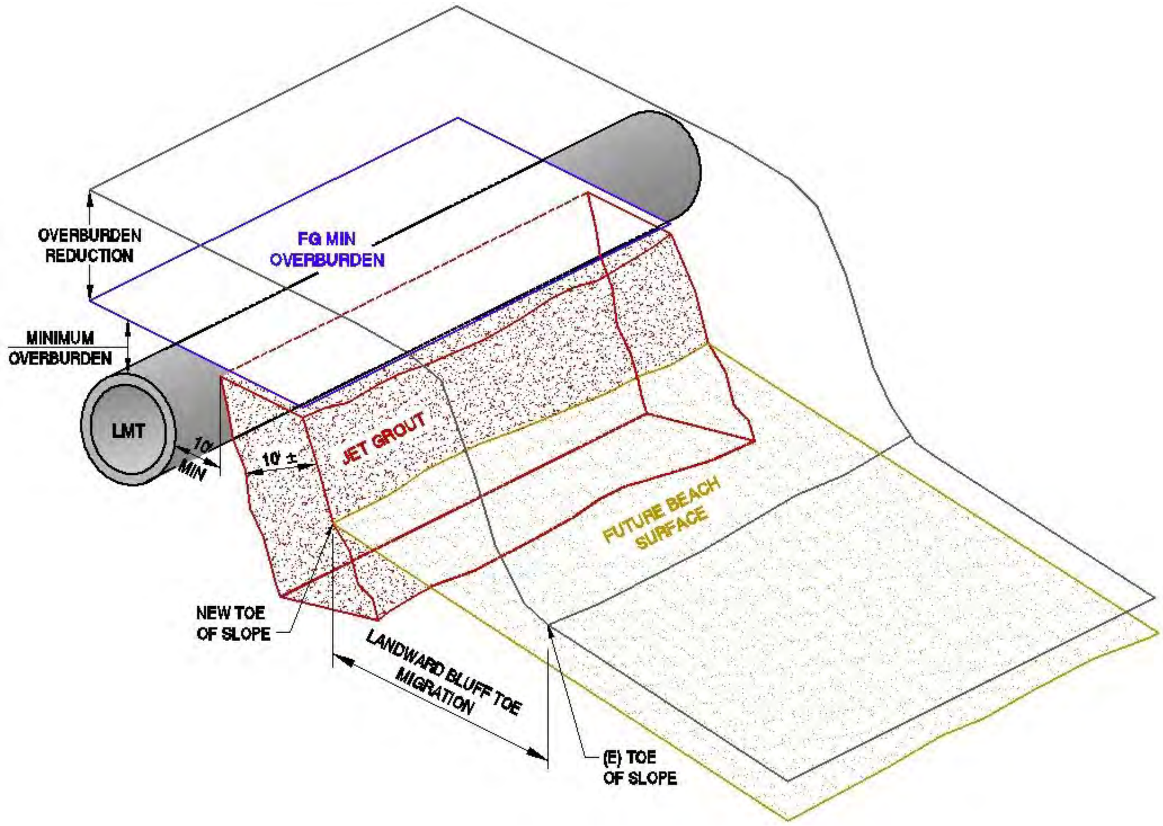


**Figure 6 – Typical Soil-Cement Wall Concept**

Construction Considerations

The construction of the soil-cement wall will include heavy construction equipment and an on-site cement grout batch plant. Equipment will likely include a drill rig and loaders, excavators, and dump trucks to manage spoils removed during drilling.

As described in the Pile Wall concept above, a significant amount of spoils is expected to be generated as excess fluid returns to the surface via the void space around the drilling rod. This fluid will need to be actively managed and off-hauled as it cannot be reused or recycled, and will require measures to contain and direct excess fluid to a predetermined area for removal.

## 1.2 TOE WALL ALTERNATIVE

As an alternative to the long-term (permanent) Low-Profile Wall, a wall could be installed along the bluff toe that would anchor the toe of bluff to prevent overall bluff retreat while allowing the upper bluff to erode; ultimately the upper bluff would attain a stable angle of repose assumed to be between 2h:1v and 5h:1v. This wall would not be permanent but would have a service life of 10 to15 years, providing immediate protection to the LMT while long-term solutions are selected and implemented.

The Toe Wall would consist of driven steel H-piles and precast concrete lagging panels; see Figure 7 for an example of this type of wall. The wall would be located at the nominal bluff toe assumed to be at an approximate elevation of +12’ MLLW. Steel Z-shaped sheet piles may need to be driven behind the precast concrete lagging panels to facilitate lagging panel installation; the sheet piles will either remain in place for additional strength or be removed for use elsewhere during construction.

Driving of steel piles would be done from the bluff top wherever possible. The construction process would likely consist of the following operations:

- Removal of existing concrete rubble along the toe wall alignment.
- Installation of a temporary driving template.
- Driving of H-piles at specified spacing.
- Driving of sheet piles (for temporary shoring, may be left in place).
- Excavation on front (ocean) side of wall to anticipated scour depth.
- Installation of precast concrete lagging panels to anticipated scour depth.
- Installation of scour protection measures (armor rock or repurposed rubble)
- Construct precast or cast-in-place concrete cap.
- Backfill in front of wall.
- Install scour protection behind wall (terrace) consisting of repurposed rubble or armor rock.

The completed wall will be partially exposed with the majority of the wall buried. Once completed, the entire wall could be covered with sand as part of beach nourishment.

Proposed Design

A proposed alignment and cross-section were developed and are shown in Figures 8 and 9. The height of the wall will vary based on the beach surface elevation, with an assumed maximum exposed face height of 15-ft that could occur if severe beach lowering occurs. Soldier piles will extend approximately 30-ft below grade, and will therefore have an overall length of approximately 45-ft.

**Figure 7 – Typical Toe Wall Examples**

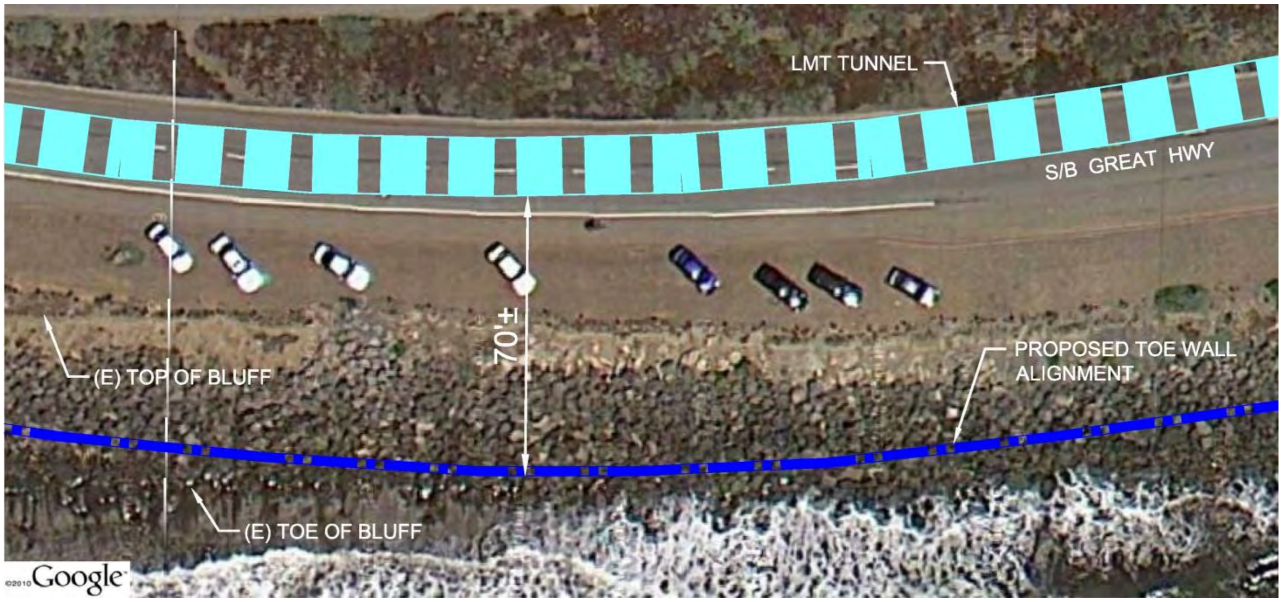


**Figure 8 – Typical Toe Wall Alignment (shown at EQR)**

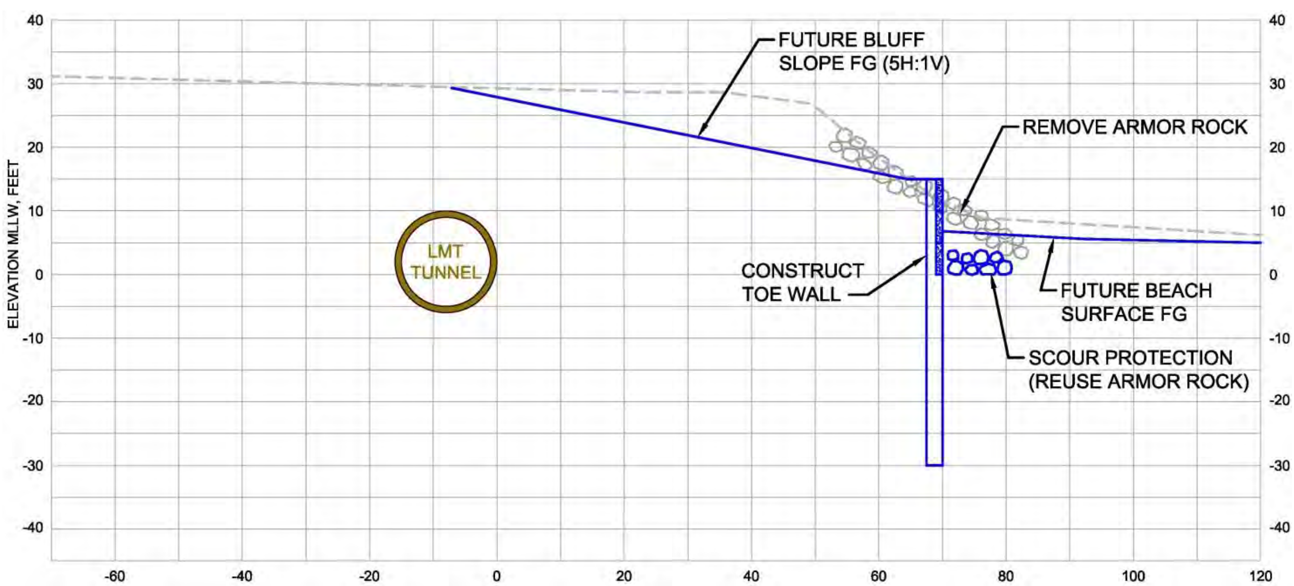


**Figure 9 – Typical Toe Wall Cross-Section (shown at EQR)**

## 1.3 NON-STRUCTURAL BLUFF ENHANCEMENTS

Other elements of Long-Term bluff protection have been proposed; these enhancements consist of non-structural approaches, including beach nourishment and dynamic cobble revetments. These will be described in a separate report.

# 2 POTENTIAL IMPLEMENTATION BY REACH

The following Table shows a potential scenario for phased implementation of the preferred project (Low-Profile Wall) for each reach. This is a general approach and the details of implementation and phasing will be determined during a subsequent stage of the project in collaboration with the responsible agencies – CCC, SFPUC, and NPS.

For conceptual purposes, the scenario presented below is shown being applied to entire reaches, however it is probable that the limits of the proposed work will change during design. The overall implementation effort is shown in Figure 10.

**Table 1 – Potential Implementation Schedule**

| Reach | Year 0 | Year 3 – 5 | Year 5-15 | Year 15+ |
|---|---|---|---|---|
| **Reach 3 (North Lot)** | Place Sand & Cobble | Place Sand & Cobble | Place Sand & Cobble | **Low-Profile Wall (with Public Access)** |
| **Reach 3 (Sandbag)** | **Low-Profile Wall** | Remove Sandbags & Rubble | Reduce Overburden over LMT | Place Sand & Cobble |
| **EQR** | **Low-Profile Wall** | Remove Rock | Reduce Overburden over LMT | Place Sand & Cobble |
| **Rubble Reach** | No Action | **Low-Profile Wall**<br>Remove Rubble | Reduce Overburden over LMT | Place Sand & Cobble |
| **Reach 2** | **Low-Profile Wall** | Remove Rubble | Reduce Overburden over LMT | Place Sand & Cobble |
| **Reach 1** | Remove Rock from Upper Slope | No Action | **Low-Profile Wall** | Place Sand & Cobble |

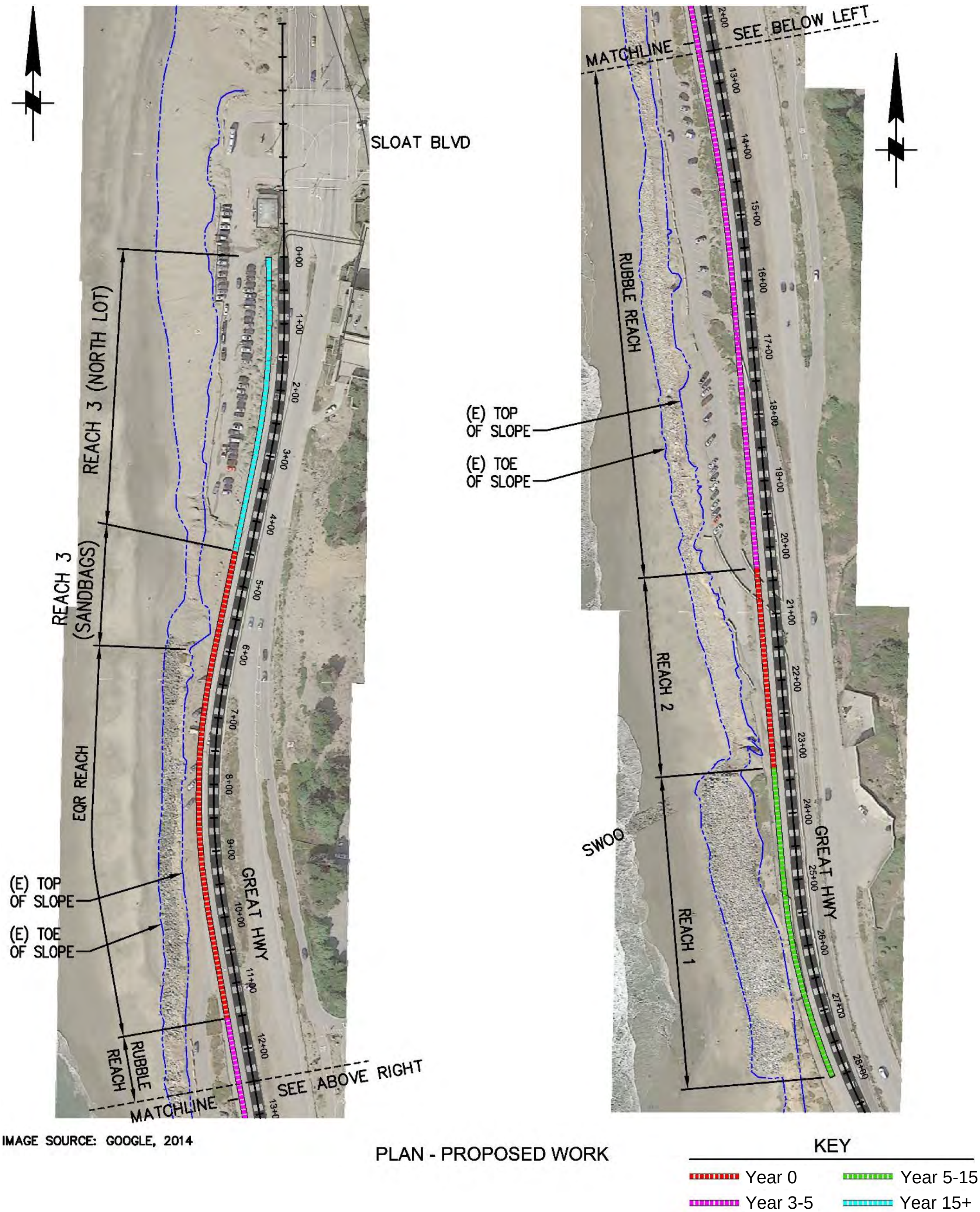


**Figure 10 – Overall Low-Profile Wall Implementation Schedule**

# APPENDIX 4

## Structural Analysis of Lake Merced Tunnel

# NUMERICAL MODELING STUDIES
# LAKE MERCED TUNNEL
# OCEAN BEACH MASTER PLAN

Prepared for:

## ESA | Environmental Hydrology

In support of:

## Coastal Protection Implementation and Interim Management Plan for South Ocean Beach


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## I. Introduction

The following summarizes the modeling studies that were completed to assess the vulnerability of the Lake Merced Transport tunnel to bluff retreat and loss of existing overburden. Preliminary studies were previously performed and more recent studies were carried out to refine the analyses. This summary report includes results from the most recent analyses. Those distances were corrected in the more recent analyses to reflect the outside face of the tunnel.

The tunnel was constructed with a segmental liner that are 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 Abramson (1993). A 12-inch thick reinforced tunnel liner was modeled in the studies.

Input for the geo-structural parameters was extrapolated from existing geotechnical reports cited in the References, input from AGS, the team's geotechnical consultant. Conditions 1, 2 and 3 identified below were evaluated using the geo-structural model presented in Figure 1 below with the groundwater level located at 4 feet above the tunnel invert and the tunnel empty of effluent. Because Condition 3 represents a plausible final configuration, this condition was further analyzed assuming the tunnel was submerged below groundwater. The geo-structural modeling parameters along with the geologic profile that was modeled in the studies is summarized as follows:

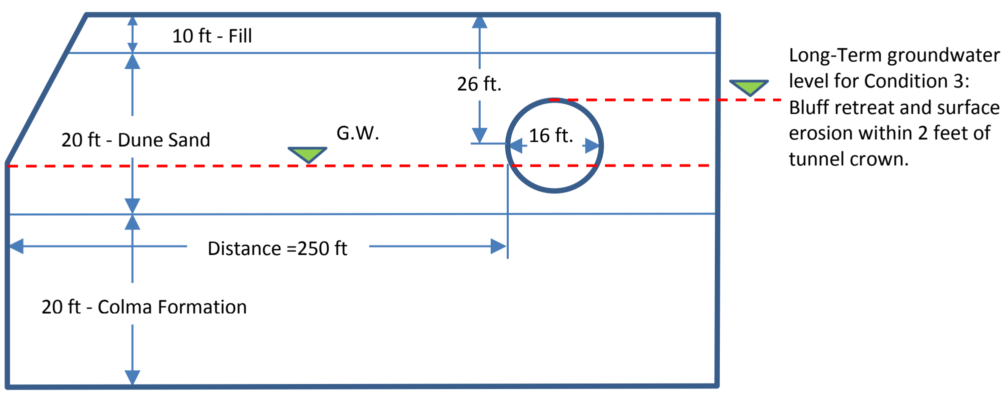



Figure 1 – Geo-Structural Model

| Engineering Parameter | | Dune Sand | Colma Formation |
|---|---|---|---|
| Shear Modulus at low strain (psf) | | 2,150,000 | 5,800,000 |
| Poisson's Ratio | Above Groundwater and below groundwater (drained condition) | 0.33 | 0.35 |
| | Below Groundwater (undrained condition) | 0.48 | 0.48 |
| Cohesion[a] (psf) | | 0 | 100 |
| Friction Angle (deg) | | 33 | 37 |
| Ko | Above Groundwater and below groundwater (drained condition) | 0.49 | 0.54 |
| | Below Groundwater (undrained condition) | 0.9 | 0.9 |

[a] Refer to discussion that follows

Table 1 - Geo-Structural Modeling Parameters Used for Analyses

| Thickness (in.) | Compressive Strength (psi) | Elastic Modulus (psf) | Poisson's Ratio |
|---|---|---|---|
| 12 | 5000 | 5.80E+08 | 0.2 |

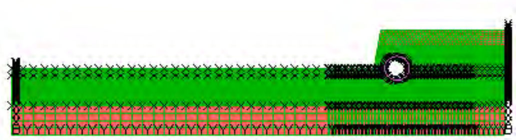
Table 2 – Tunnel Structural Liner Properties

Initial studies found that the bluff outer face was unstable in its configured profile using the modeling parameters above. A parametric study was initiated to determine the bluff internal strength required to have a slope at the configured profile. That study determined that the slope was stable with an apparent cohesion of 2,500 psf. The calculated apparent cohesion value was carried forward in the modeling analyses to represent a slope that "stands on its own" to be consistent with conditions that currently exist. The apparent cohesion used in our analyses should be confirmed by site specific geotechnical studies completed during the implementation phase.

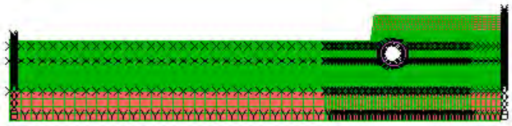
## II. Conditions Analyzed

Three conditions were analyzed as depicted by the cartoon models below using the geo-structural model in Figure 1.

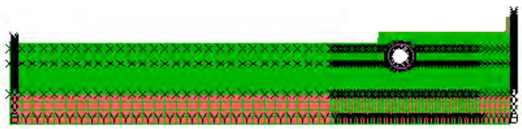
Condition 1: The existing slope retreats towards the tunnel, the slope toe erodes down to the tunnel springline, and no vertical erosion above the tunnel.



Condition 2: The existing slope retreats towards the tunnel, the slope toe erodes down to the crown of the tunnel, and no vertical erosion above the tunnel



Condition 3: The existing slope retreats towards the tunnel, the slope toe erodes down to the crown of the tunnel, and vertical erosion extends down to 2 feet above the tunnel



An additional analysis was completed for Condition 3 to evaluate the implications of a long-term rise in the groundwater level resulting in a submerged tunnel. The groundwater level was placed at the tunnel crown as depicted in Figure 1 above to represent a fully buoyant condition with the tunnel empty of effluent.

The following summarizes the findings of the modeling studies due to bluff retreat and loss of overburden confinement.

## II. Summary of Analyses

When the vertical and horizontal confinement regime on the tunnel is changed, the corresponding impact is tunnel distortion. The calculated distortion of the tunnel for the three conditions was compared against published recommended distortion criteria. The following criterion was utilized to assess the tunnel distortions. Although these are recommended distortions, there is no indication of the safety factor corresponding to the recommendations.

Typical lining distortions due to ground loading in circular tunnels are shown in Table 1 for different ground conditions. The distortion is defined as the change in radius, ΔR, divided by the tunnel radius, R.

| Soil Type | ΔR/R |
|---|---|
| Stiff to hard clays, overconsolidation ratio>2.5–3.0 | 0.15-0.40% |
| Soft clays or silts, overconsolidation ratio<2.5–3.0 | 0.25-0.75% |
| Dense or cohesive sands, most residual soils | 0.05-0.25% |
| Loose sands | 0.10-0.35% |

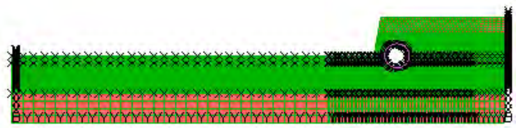
Table 3- Recommended Distortion Ratios for Circular Tunnels[1]

[1] FHWA, Table 10-2

The bluff soils are closely aligned with dense or cohesive sands. Therefore, the recommended range for the tunnel distortion ratio (ΔR/R) against which our analysis was compared to is 0.05%-0.25%.

A. Analyses of Condition 1

The numerical analysis that represents Condition 1 is summarized in Table 4 below.



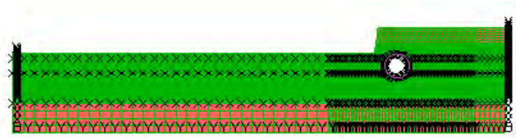
| Horizontal Erosion Only, Full Overburden (Distance from Tunnel Edge to Bluff Toe) | Average Distortion (%) |
|---|---|
| Baseline Condition, No Erosion | 0.04 |
| 30ft Horizontal Erosion (215 ft to Bluff) | 0.08 |
| 60ft Horizontal Erosion (185 ft to Bluff) | 0.11 |
| 90ft Horizontal Erosion (155 ft to Bluff) | 0.13 |
| 150ft Horizontal Erosion (95 ft to Bluff) | 0.19 |
| 200ft Horizontal Erosion (45 ft to Bluff) | 0.24 |
| 225ft Horizontal Erosion (20 ft to Bluff) | 0.26 |
| 235ft Horizontal Erosion (15 ft to Bluff) | 0.26 |
| 240ft Horizontal Erosion (10 ft to Bluff) | 0.26 |
| 245ft Horizontal Erosion (5 ft to Bluff) | 0.25 |

Table 4 – Average Tunnel Distortion Condition 1

Table 4 indicates that the average tunnel distortion is marginally exceeded when the bluff toe erodes down to the tunnel spring line within a distance of 20 feet from the outside face the tunnel. Our analysis indicates the tunnel distortion is not adversely impacted if the bluff toe gets closer than 20 feet.

B. Analyses of Condition 2

The numerical analysis that represents Condition 2 is summarized in Table 5 below.



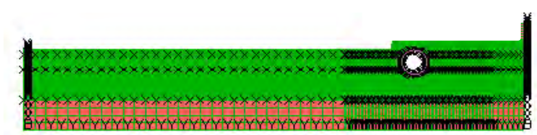
| Horizontal Erosion Only, Full Overburden (Distance from Tunnel Edge to Bluff Toe) | Average Distortion (%) |
|---|---|
| Baseline Condition, No Erosion | 0.03 |
| 30ft Horizontal Erosion (215 ft to Bluff) | 0.07 |
| 60ft Horizontal Erosion (185 ft to Bluff) | 0.08 |
| 90ft Horizontal Erosion (155 ft to Bluff) | 0.10 |
| 150ft Horizontal Erosion (95 ft to Bluff) | 0.14 |
| 200ft Horizontal Erosion (45 ft to Bluff) | 0.17 |
| 225ft Horizontal Erosion (20 ft to Bluff) | 0.18 |
| 235ft Horizontal Erosion (15 ft to Bluff) | 0.19 |
| 240ft Horizontal Erosion (10 ft to Bluff) | 0.19 |
| 245ft Horizontal Erosion (5 ft to Bluff) | 0.18 |

Table 5 - Average Tunnel Distortion Condition 2

Table 5 indicates that the average tunnel distortion is not exceeded even when the bluff toe erodes to within 5 feet of the outside face of the tunnel.

C. <u>Analyses of Condition 3</u>

The numerical analysis that represents Condition 3 with a groundwater level 4 feet above the tunnel invert is summarized in Tables 6 and 7 below. Vertical erosion was analyzed with the bluff toe at 20 from the tunnel (Table 6) and with the bluff toe at 10 feet from the tunnel (Table 7).



| Vertical Erosion after Horizontal Erosion (Bluff Toe 20 ft from Tunnel) | Average Distortion (%) |
|---|---|
| Baseline Condition, No Erosion | 0.03 |
| 225ft Horizontal Erosion (20 ft to Bluff) | 0.19 |
| 3ft Vertical Erosion (14 ft to Surface) | 0.12 |
| 6ft Vertical Erosion (11 ft to Surface) | 0.09 |
| 10ft Vertical Erosion (7 ft to Surface) | 0.07 |
| 11ft Vertical Erosion (6 ft to Surface) | 0.07 |
| 12ft Vertical Erosion (5 ft to Surface) | 0.07 |
| 13ft Vertical Erosion (4 ft to Surface) | 0.07 |
| 14ft Vertical Erosion (3 ft to Surface) | 0.08 |
| 15ft Vertical Erosion (2 ft to Surface) | 0.08 |

Table 6 - Average Tunnel Distortion Condition 3, Toe 20 Feet from Tunnel

| Vertical Erosion after Horizontal Erosion (Bluff Toe 10 ft from Tunnel) | Average Distortion (%) |
|---|---|
| Baseline Condition, No Erosion | 0.03 |
| 240ft Horizontal Erosion (10 ft to Bluff) | 0.18 |
| 3ft Vertical Erosion (14 ft to Surface) | 0.14 |
| 6ft Vertical Erosion (11 ft to Surface) | 0.10 |
| 10ft Vertical Erosion (7 ft to Surface) | 0.08 |
| 11ft Vertical Erosion (6 ft to Surface) | 0.08 |
| 12ft Vertical Erosion (5 ft to Surface) | 0.08 |
| 13ft Vertical Erosion (4 ft to Surface) | 0.09 |
| 14ft Vertical Erosion (3 ft to Surface) | 0.09 |
| 15ft Vertical Erosion (2 ft to Surface) | 0.09 |

Table 7 - Average Tunnel Distortion Condition 3, Toe 10 Feet from Tunnel

Our interpretation of the above conditions and analyses is that removal of subjacent lateral support causes the tunnel to 'egg out' along the horizontal axis. When vertical erosion of the bluff occurs after the bluff has retreated, the vertical forces that cause the 'egging' are reduced and the tunnel regains some of its roundness.

## III. Summary

**<u>Condition 1</u>**

Condition 1 represents what is thought to be a 'short term' event associated with a rather extreme storm event condition that would rapidly erode the toe below the tunnel crown down to the level of the springline. What the analysis indicates is that the tunnel distortion criterion is marginally exceeded when the bluff erodes to about 20 feet from the outside face of the tunnel. In the absence of slope protection measures built into the slope prior to the extreme storm event, the analyses indicates the point at which aggressive pro-active measures would be required to ensure tunnel distortion does not degrade to an unacceptable level. This analysis suggests that the tunnel is more prone to distortion when subjacent lateral support is removed.

**<u>Condition 2</u>**

Condition 2 represents what is thought to be a more realistic long-term erosion of the bluff toe. So long as subjacent lateral support is maintained, the bluff toe can erode up to within 5 feet of the outside face of the tunnel without exceeding tunnel distortion criterion.

**<u>Condition 3</u>**

Condition 3 represents what could eventually be a designed beach configuration consistent with long-term planning for Ocean Beach. Where the bluff toe erodes down to the crown of the tunnel and to within 5 feet of its outer face and the bluff top erodes or is sculpted to have at least two feet of cover over the tunnel, our analyses indicates the tunnel distortion criteria would not be exceeded under these conditions.

However, to evaluate a long-term condition for the case where the groundwater level rises resulting in a submerged tunnel, Condition 3 was analyzed with a groundwater level at the crown of the tunnel to represent a fully buoyant tunnel empty of effluent. Table 8 below summarizes the results of that analysis.

| Vertical Erosion after Horizontal Erosion (10 ft from the Bluff), Groundwater at Tunnel Crown | Average Distortion (%) |
|---|---|
| Baseline Condition, No Erosion | 0.03 |
| 240ft Horizontal Erosion (10 ft to Bluff) | 0.18 |
| 3ft Vertical Erosion (14 ft to Surface) | 0.13 |
| 6ft Vertical Erosion (11 ft to Surface) | 0.09 |
| 10ft Vertical Erosion (7 ft to Surface) | 0.08 |
| 11ft Vertical Erosion (6 ft to Surface) | 0.08 |
| 12ft Vertical Erosion (5 ft to Surface) | 0.08 |
| 13ft Vertical Erosion (4 ft to Surface) | 0.09 |
| 14ft Vertical Erosion (3 ft to Surface) | 0.10 |
| 15ft Vertical Erosion (2 ft to Surface) | 0.11 |

Table 8 - Average Tunnel Distortion Condition 3, Toe 10 Feet from Tunnel, Groundwater at the Tunnel Crown

With respect to tunnel deformation from the confining pressures, the tunnel structure itself does not exceed the distortion criteria. However, under the following conditions – an empty tunnel, groundwater at the tunnel crown, two feet of cover over the tunnel – our buoyancy calculations reveal the tunnel is buoyant and would pop out of the ground. It would require at least 6 feet of cover on top of the tunnel to counterbalance the buoyant forces exclusive of any safety factor. This simplified approach ignores the “bending resistance” the structural lining would provide in a longitudinal direction. Even so, the structural liner was not designed for this condition and it would not be prudent to rely on the structural liner to resist buoyant forces in a longitudinal direction. Buoyancy would have to be controlled with at least 6 to 8 feet of sand cover, a hold down structure, or through a combination of cover and restraint.

The numerical analyses completed for this phase of the study considered static loading on the tunnel only. Our analyses evaluated the implications to the structural liner when subjacent lateral support is removed, when vertical overburden is removed, or a when a combination of the two occurs. Additional engineering analyses would be needed to evaluate the implications of seismic forces on the tunnel for the different conditions analyzed above. We recommend additional engineering analyses be completed after site-specific geotechnical investigations are completed to evaluate the impacts of seismic loads on the tunnel and to incorporate any revisions to the geotechnical parameters.

## IV. REFERENCES

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# APPENDIX 5

## Geotechnical Summary of Subsurface Conditions at South Ocean Beach

# Appendix 5: Summary of Geotechnical Considerations at South Ocean Beach

## Prepared by AGS, Inc.

## January 7, 2014

The project along the Ocean Beach is located along the Pacific Coastline, South of Sloat Boulevard(see Plate 1). The west side of the San Francisco Peninsula was drowned by rising seas during Pleistocene time and includes a northwest-trending structural depression, known as the bay block. The bay block is within the Coast Ranges geomorphic province, a region characterized by generally northwest-trending mountains, 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 Francisco Peninsula east of the San Andreas fault is underlain by Jurassic to Cretaceous age bedrock of the Franciscan Complex, which in the project vicinity is overlain by a thick sequence of Quaternary sedimentary units. In the project vicinity the Franciscan bedrock surface is estimated to occur deeper than 300 feet (Schlocker, J., and Bonilla, M.G., 1972).

The Quaternary deposits along the coastal bluffs in the project vicinity consist of four geologic units. These units are artificial fill (Qaf), Holocene dune sand (Qd), Pleistocene Colma Formation (Qco), and ePlio-Pleistocene Merced Formation (QTm) (Clifton and Hunter, 1987; Caskey, et al., n.d., CDMG, 2005).

***Artificial Fill -*** The artificial fill seems to consist mainly of reworked dune sand, with occasional gravel and construction debris, and is commonly underlain by dune sand. Fill occurs as pavement subgrade, infill along some portions of the bluffs, and as backfill over utilities. Since the 14-foot diameter Merced Transport Tunnel was excavated with a tunnel boring machine the backfill area next to the tunnel is limited to pothole areas, where the tunnel was apparently exposed for maintenance, and to the junction (undercrossing) of the Southwest Ocean Outfall (SWOO) pipe, where the tunnel was exposed and backfilled. In general, a thick layer of asphaltic concrete pavement covers most of the fill, with a measured thickness exceeding 12-inches near some of the cliff edges. Inspection of the native stratigraphy in the exposed bluffs indicates there is generally less than 5 feet of fill along the coastal bluffs; however, thicker fill (up to 30 feet) was encountered by AGS during previous investigation for Emergency bluff stabilization project near the buried SWOO. Geotechnical properties of the existing fill vary significantly along the project alignment. Loose to very dense sand with varying amount of silt and gravel and rubbles were encountered during previous investigations. The friction angle of the fill material may range between 28 to 34 degrees with or without cohesion.

***Holocene Dune Sand -*** Beneath the artificial fill are recent Holocene dune sand deposits, and where the recent dune sands have been eroded away older dune sands and the Colma Formation occur at relatively shallow depths. At Point Lobos and the Presidio the dune sand overlies the Franciscan Complex. The thickness of the dune sand ranges from zero to nearly 100 feet; its relative density grades from loose near the surface to dense and very dense with depth. From a geotechnical perspective, dune sand is fine- to medium-grained and well sorted, with trace to few amounts of fines by weight. The friction angle of the Holocene dune sand may range between 30 to 32 degrees without cohesion.

***Pleistocene Colma Formation*** **-** The Colma Formation lies stratigraphically below the latest Pleistocene and Holocene deposits, and underlies the dune sands in the project area. The Colma Formation includes Pleistocene coastal and estuarine sediments typically consisting of dense to very dense silty sand and poorly graded sand with silt or clay, occasionally with more clay and gravel (Blake, et al., 2006). The Ocean Beach exposure of the Colma Formation was most likely deposited in a foreshore environment, although it could also have been deposited in the backshore. From a geotechnical perspective, the friction angle of the Colma Formation may range between 33 and 38 degrees with or without cohesion. Presence of vertical bluffs along the ocean coastline in Ocean Bean area where Colma Formation is exposed indicates that the Colma Formation poses apparent cohesion. The apparent cohesion characteristic of the Colma Formation comes from either cementation or presence of fine material. Both cementation and fine materials are subject to erosion; therefore, apparent cohesion reduces over time.

***Plio-Pleistocene Merced Formation*** **-** Tilted marine strata of the Plio-Pleistocene Merced Formation (QTm) (Clifton and Hunter, 1987) lie unconformably below Pleistocene Colma Formation (Qco) and late Pleistocene and Holocene dune sand (Qs). The late Pliocene to Pleistocene Merced Formation (QTm), located primarily in the southwestern portion of San Francisco, consists of sand, silt, and clay basin deposits that originated in a shallow marine and coastal non-marine setting. The basal contact between Colma and the Merced Formation is visible on the cliff face from the beach at Ocean Beach area. From a geotechnical perspective, although landslide deposits were found throughout the city, they were found in greater frequency associated with the oversteepened slopes along the northern and western shoreline as well as in some inland upland areas. Of all of the geologic units, the Merced Formation (QTm) appears to have a greater abundance of landslides associated with it than other units. The friction angle of the Merced Formation may range between 26 and 34 degrees with or without cohesion. Similar to Colma Formation, near vertical bluffs are observed within the area where Merced Formation exposed. Hence, the Merced Formation also poses apparent cohesion.

At Ocean Beach, an approximately 60-foot-thick section of the Merced was mapped by Li (2005) in the sea cliffs, at the southern portion of the proposed project. Li (2005) noticed that Near Sloat Boulevard, only the upper unit of Merced Formation is exposed in the cliffs. At the Ocean Beach locality, the uppermost geological units of the Merced Formation are exposed at beach level in a northeast-dipping succession that shallows to very gently east-dipping at its northernmost exposure. In general, the Merced Formation rises in elevation from north to south, with the base of the formation reaching 60 feet above sea level at its southernmost exposure (USGS, 2005).

AGS has reviewed all the available geotechnical reports prepared previously by AGS (2012), Parsons-Brinckerhoff (PB, 1990), Harding-Lawson Associates (HLA, 1976, 1977 and 1981), and Woodward-Clyde (WWC, 1976, 1977, and 1978). Total of twenty one (21) soil borings were reviewed from the above-mentioned reports along approximately 3,100 lineal feet of existing Lake Merced Transport (LMT) Tunnel. Only borings within 200 feet from the alignment of the tunnel were considered. The soil borings extended to maximum depths ranging from 26.5 feet to 502 feet. Twelve (12) out of the 19 reviewed soil borings were located near the Ocean Side Treatment Plant. Approximate locations of each boring with their respective maximum depth are shown on Plates 2 through 7.

Based on a review of the available geotechnical information, AGS identified data gap along the Great Highway between Sloat Boulevard and south of Ocean Beach Pump Station. Plate 8 shows the data gap based on a review of the available geotechnical information. AGS used the following criteria to identify the data gap:

- Relative Location of Borings with respect to the Lake Merced Transport (LMT) Tunnel;
- Geological variability along the LMT tunnel;
- Lateral distance between the adjacent borings; and
- Availability of geotechnical parameters.

All borings except three borings presented in HLA and WWC reports were performed east of the LMT Tunnel. Furthermore, majority of the borings reports did not distinguish between Merced and Colma formation. In Borings B-4 and B-5 performed by WWC at the beach level, sandy material with high blow count (greater than 60 per foot) and friction angle of 36 degrees were encountered. WWC did not assign this layer as part of Colma or Merced Formation. However, at almost same location, USGS reported outcrop of Merced formation which typically characterized by lower blow count and friction angle.

Since the Merced Formation was encountered at much higher elevation to the south of the project, we expect to encounter the interface between Colma and Merced Formations at shallow depth from Great Highway elevation. According to USGS report, the interface dips northeast; therefore, it is expected to be deeper than the LMT Tunnel invert east of the LMT Tunnel.

Merced Formation is relatively weaker than Colma Formation and many slides have been reported within this formation near the Lake Merced area. Based on 62 shear strength test, CDMG (2000) recommended using the average friction angle of 27 and 34 degrees for Merced Formation and Colma Formation, respectively. Slope stability and erodability of the materials in front of the LMT Tunnel highly depends on the spatial location of the transition between the Merced and the Colma Formations. Previous investigations did not focus on finding this transition and may have been missed during the investigation program. The Plate 9 shows the sensitivity of the factor of safety against global slope failure with respect to variation with friction angle assuming the thickness of the materials remain constant. From the assumed subsurface profile, it is obvious that the slope stability of the slope highly depends on shear strength of Merced Formation. Based on assumed subsurface profile and shear strength, the slope is marginally statically stable. However, we expect local failure if the shear strength decreases, groundwater level increases, or thickness of the weak materials increases. Therefore, it is essential to obtain more realistic information with regard to the above-mentioned parameters.

AGS recommends supplemental investigation to obtain relatively undisturbed soil sample to perform shear strength test and slope stability analyses. The investigation program should consists of drilled soil borings both in the beach-side and land-side of the LMT Tunnel and extends minimum 70 feet. The borings will provide sufficient information with regard to the material properties and geometries of the material boundary.

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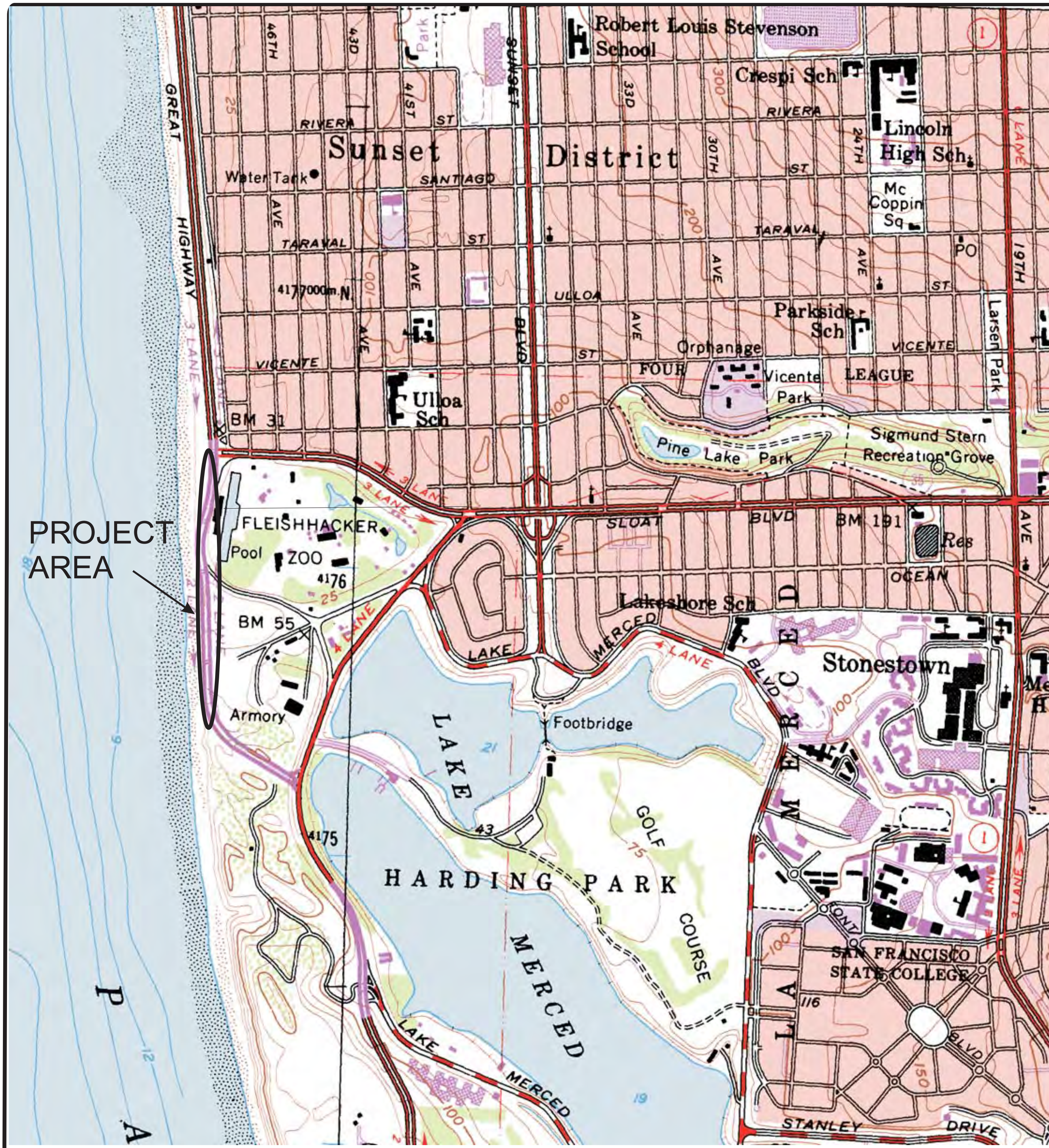



Source: US Geological Survey, 1996, San Francisco North 7.5 Minute Quadrangle.

N

0 800 1600 ft

**SITE LOCATION MAP**

OCEAN BEACH PROJECT
SAN FRANCISCO, CA

**AGS, Inc.**
CONSULTING ENGINEERS

JOB NO. AGS-12-058 | DATE: JAN. 2014 | PLATE 1


SCALE

0 200 feet

N

| EXISTING BORING LOCATIONS<br><br>OCEAN BEACH PROJECT<br>SAN FRANCISCO, CA | | AGS, Inc.<br>CONSULTING ENGINEERS |
|---|---|---|
| JOB NO. AGS-12-058 | DATE: 11/13 | PLATE 2 |


SCALE

0 200 feet

N

| EXISTING BORING LOCATIONS<br><br>OCEAN BEACH PROJECT<br>SAN FRANCISCO, CA | | AGS, Inc.<br>CONSULTING ENGINEERS |
|---|---|---|
| JOB NO. AGS-12-058 | DATE: 11/13 | PLATE 3 |


SCALE

0 200 feet

N

| EXISTING BORING LOCATIONS<br><br>OCEAN BEACH PROJECT<br>SAN FRANCISCO, CA | | AGS, Inc.<br>CONSULTING ENGINEERS |
|---|---|---|
| JOB NO. AGS-12-058 | DATE: 11/13 | PLATE 4 |


SCALE

0 200 feet

N

EXISTING BORING LOCATIONS

OCEAN BEACH PROJECT
SAN FRANCISCO, CA

**AGS, Inc.**
CONSULTING ENGINEERS

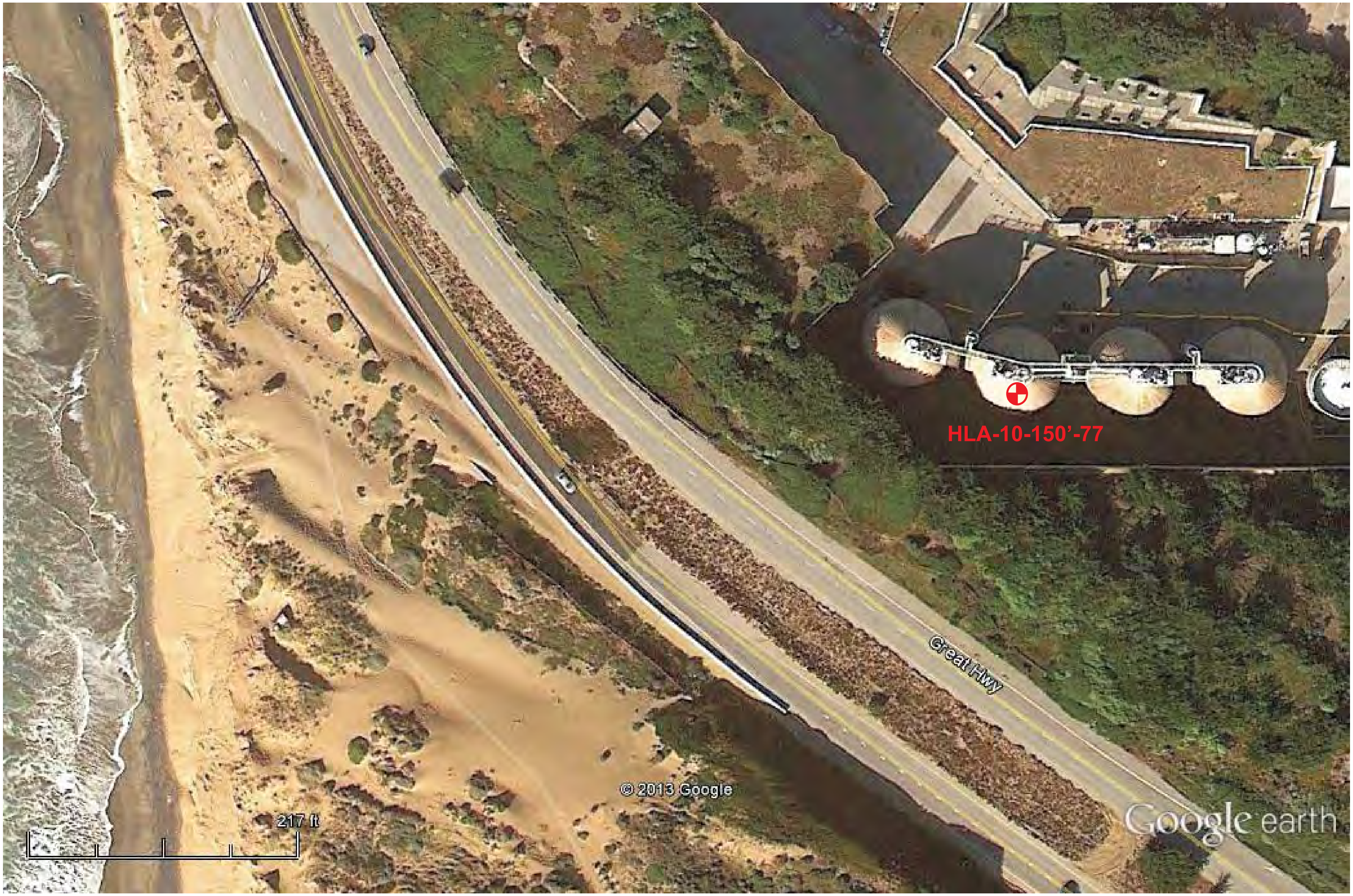
| | | |
|---|---|---|
| JOB NO. AGS-12-058 | DATE: 11/13 | PLATE 5 |


SCALE

0 200 feet

N

| EXISTING BORING LOCATIONS<br><br>OCEAN BEACH PROJECT<br>SAN FRANCISCO, CA | | **AGS, Inc.**<br>CONSULTING ENGINEERS |
|---|---|---|
| JOB NO. AGS-12-058 | DATE: 11/13 | PLATE 6 |




SCALE

0 200 feet

N

| EXISTING BORING LOCATIONS<br><br>OCEAN BEACH PROJECT<br>SAN FRANCISCO, CA | | AGS, Inc.<br>CONSULTING ENGINEERS |
|---|---|---|
| JOB NO. AGS-12-058 | DATE: 11/13 | PLATE 7 |


SCALE

0 500 feet

N

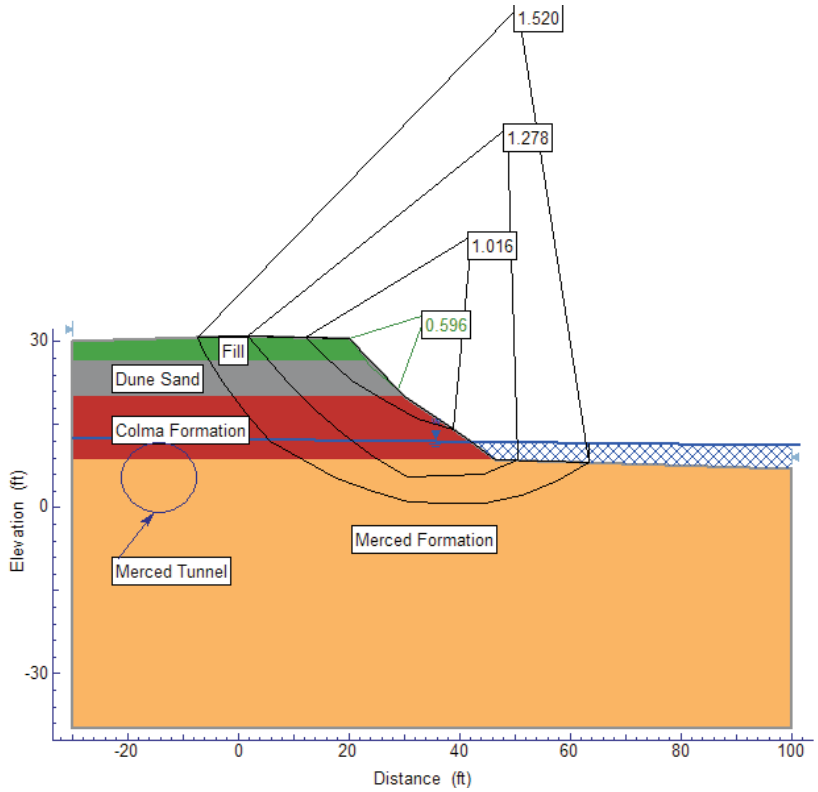
Area with Available Data
Area with Data Gap

GEOTECHNICAL DATA GAP

OCEAN BEACH PROJECT
SAN FRANCISCO, CA

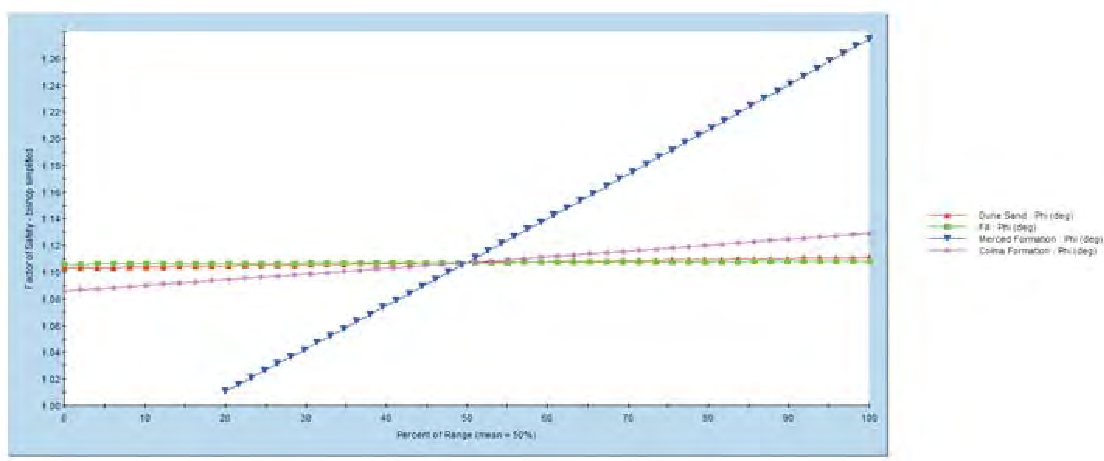
**AGS, Inc.**
CONSULTING ENGINEERS

| | | |
|---|---|---|
| JOB NO. AGS-12-058 | DATE: 11/13 | PLATE 1 |




>80% Probability of Failure
Under Static Condition in Next 10 years
For Slip Plane with $FS_{min}$=1.0

>50% Probability of Failure
Under Static Condition in Next 10 years
For Slip Plane with $FS_{min}$=1.3



**SENSITIVITY ANALYSIS FOR**
**STATIC SLOPE STABILITY RESULTS USING**

OCEAN BEACH PROJECT
SAN FRANCISCO, CA

**AGS, Inc.**
CONSULTING ENGINEERS

| | | |
|---|---|---|
| JOB NO. AGS-12-058 | DATE: 11/13 | PLATE 9 |

# APPENDIX 6

Coastal Management Framework
Scope of Work

**Ocean Beach Master Plan**
**Implementation Studies**
**Coastal Management Framework**

**Scope of Work**

TASK 1: Interim Coastal Protection Approach
Task 1 will result in a technical memo laying out the near-term approach to coastal management.

1.a Definition of Interim Protection Best Practices, Precedents, Case Studies
1.b Monitoring and Adaptive Management
1.c Definition of Triggers and Phasing
1.d Clean-up / Excavation approach
1.e Coordination with Roadway work
1.f Interim Access Planning/Surface Restoration
1.g Coastal Commission communication, coordination
1.h Coordination and Peer Review of SFPUC's Infrastructure Condition Risk Assessment study

Building on meetings conducted through the Master Plan process, the project team will work with SFPUC, Army Corps, and GGNRA to develop an interim strategy for coastal protection over a 1-10 year time period, emphasizing options that are reversible, minimally impactful, and compatible with Master Plan recommendations. SPUR will support communication and permitting efforts around the interim approach with the California Coastal Commission and partner agencies. The strategy will include:

- Development of phasing and sequencing approach for coastal protection measures, addressing interim surface conditions, access and ecological functions, the logistics of incremental implementation, and defining appropriate physical triggers for implementing key actions.
- Review and communication about the lifespan of affected SFPUC infrastructure and the process for prioritizing reconfiguration or relocation relative to other assets over time. This will ensure that the hazards facing the Clean Water Program are thoroughly evaluated and build public support for interim protection; and
- Recommendations for surface restoration and re-vegetation consistent with projected hazards, defined objectives, and the likely progression of management interventions.

TASK 2: Coastal Engineering Feasibility Studies
Task 2 will result in the production of two studies: 1) Coastal Vulnerability Analysis Study; and 2) Coastal Engineering Feasibility Study.

2.a LMT Coastal Vulnerability Analysis
    2.a.i Identify/Assess Hazard Mode, Severity, Risk
    2.a.ii Assess Future Vulnerability under sea level rise
2.b Analysis of Subsurface conditions
2.c Development and Multi-Objective Evaluation (risk, cost, access, ecology, reversibility, compatibility w/ OBMP) of LMT Protection options:
    2.c.i Toe Wall + stable slope

2.c.ii Tangent Pile @ LMT
2.c.iii Internal ballast or other reinforcement

With the interim strategy (Task 1) in place, the project team will initiate technical studies to develop and test the concepts outlined in Ocean Beach Master Plan. The first is an expanded analysis of existing and projected coastal hazards through erosion and flooding, taking account of dynamic processes such as wave setup, coastal recession, impacts of existing structures, sedimentation patterns, and climate change.

Second is a feasibility study and alternatives analysis of approaches to protecting the Lake Merced Tunnel in place. This will develop and evaluate alternative approaches with a combination of low-profile hard structures, dynamic cobble revetments, and sand placed through beach nourishment.

<u>TASK 3: Design and Manage Coastal Protection *in-situ*- Pilot Studies</u>
Task 3 will result in the production of a report reviewing key opportunities and recommendations for coastal protection pilots, design, and recommendations

- 3.a Dynamic cobble revetment pilot:
  - 3.a.i Planning
  - 3.a.ii Coordination with ACOE nourishment actions
  - 3.a.iii Management
  - 3.a.iv Data gathering

The Master Plan coastal protection concept makes use of innovative combinations of elements including hard structures, dynamic cobble revetments, and placed sand. During the interim (1-10 year) period, the opportunity exists to place some of these elements on site and study their viability in the specific high-energy conditions at Ocean Beach. The project team will work with the SFPUC and partner agencies to develop an *in-situ* pilot study of possible protection measures, including a dynamic cobble revetment. The logistics and physical installation would be conducted by the SFPUC or its contractors.

<u>TASK 4: Interagency Coastal Management Agreement</u>
Task 4 will result in a Coastal Management Framework Report and a Draft Memorandum of Understanding between the SFPUC, Army Corps, and GGNRA.

- 4.a Coastal Protection Strategy (1-40 years)[GG7]
  - 4.a.i Definition of Phasing, with Climate and Erosion Triggers
  - 4.a.ii Engineering Development
  - 4.a.iii Capital Planning
- 4.b Interagency coordination of Coastal Protection Framework (item 1.d above)
- 4.c Regulatory Strategy: CEQA/NEPA/CCC
- 4.d MOU execution and Coastal Development Permit

Based on the analysis conducted in tasks 2 and 3, the project team will work closely with the SFPUC, Army Corps and GGNRA to develop and execute an Interagency Coastal Management Agreement, which would define schematic engineering, phasing and sequencing of coastal management actions and interventions, according to defined triggers. These actions, including retreat, beach nourishment, and armoring, would be elaborated in a framework plan defining each agency's responsibilities and be agreed to in a Memorandum of Understanding, Joint Management Agreement, or similar structure. The framework plan would also outline sources of capital funds, regulatory approval pathways, and processes requiring interagency coordination.