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2-21-0912 (SFPUC OCEAN BEACH ARMORING)

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EXHIBITS, Part 3

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Conceptual Engineering Report

Ocean Beach Long-Term Improvements Project

PREPARED FOR:

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

SEPTEMBER 2019

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Produced by:

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McMillen Jacobs Associates

CHS Consulting Group

San Francisco Public Works

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Glossary

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

Executive Summary

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

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

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

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

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

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

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

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

The goal for the project is to:

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

The objectives for the project are:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

3. Removing the existing shoreline protection revetment and accumulated rubble.

4. Providing a wastewater infrastructure access road and public multi-use recreational trail with beach access in place of the Great Highway.

5. Establishing a program to ensure maintenance of the beach and dune system based on periodic sand nourishment.

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

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

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

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

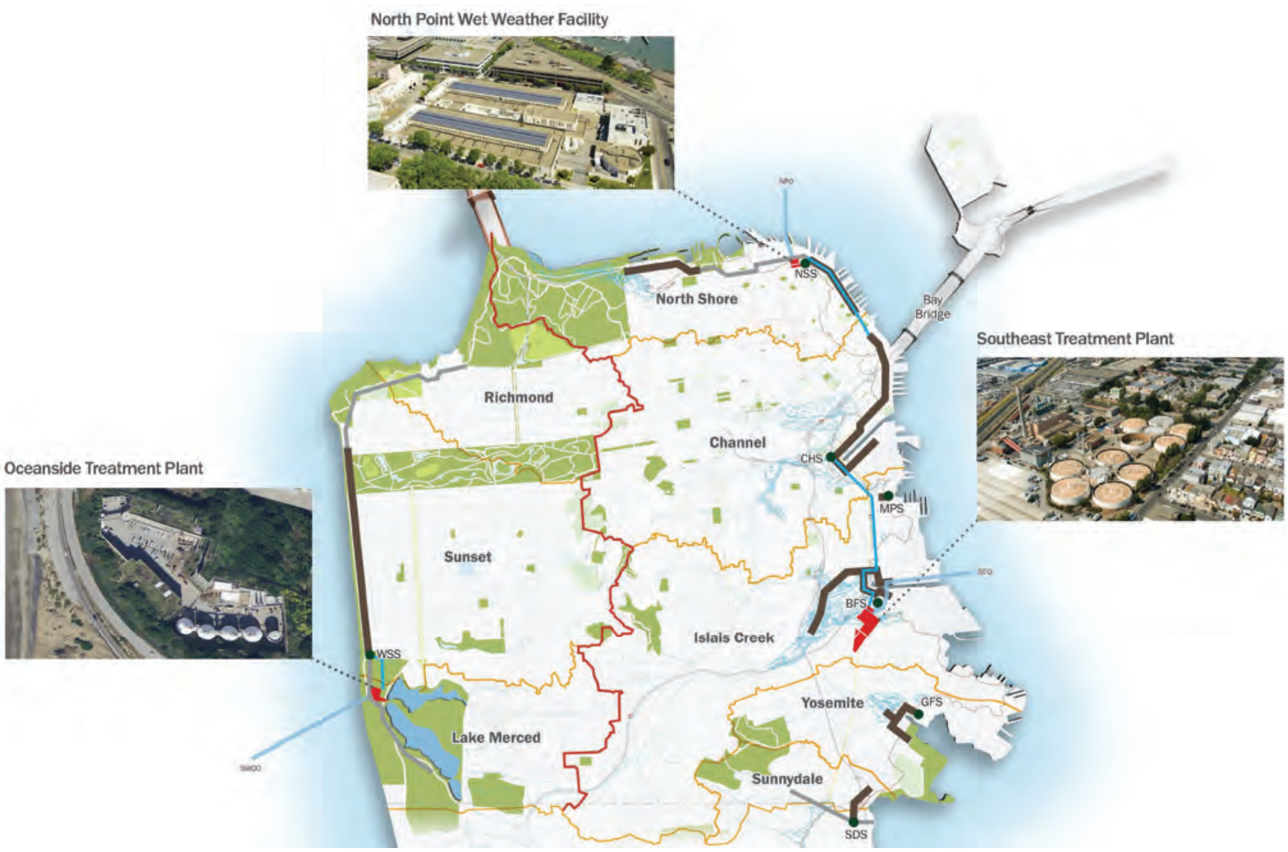

Figure ES-1: Wastewater Treatment Facilities Operated by SFPUC

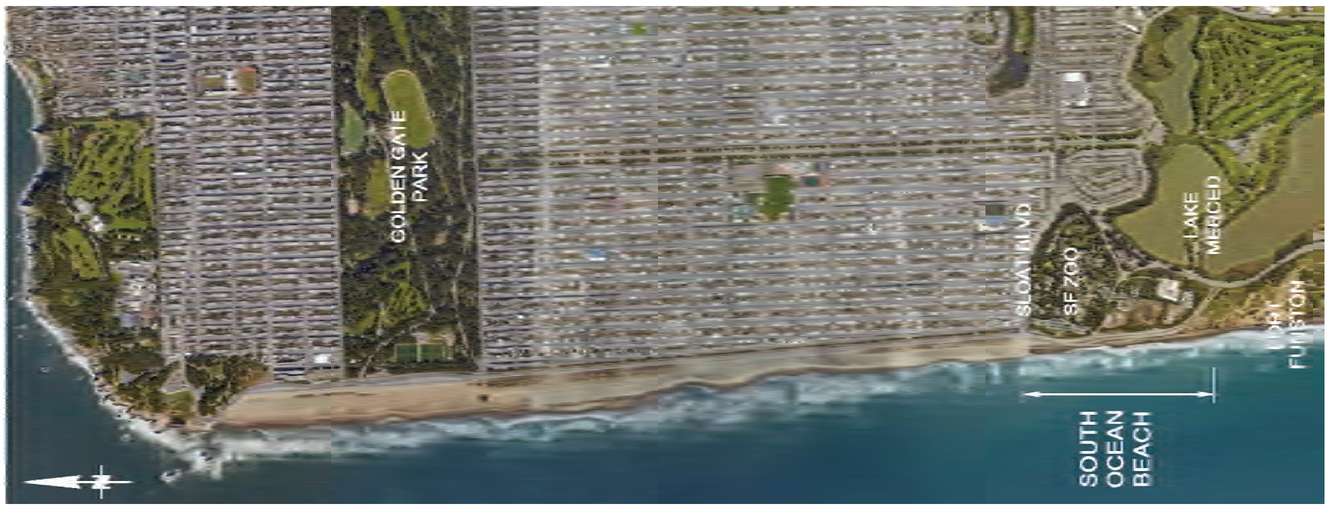

Figure ES-2: South Ocean Beach Location

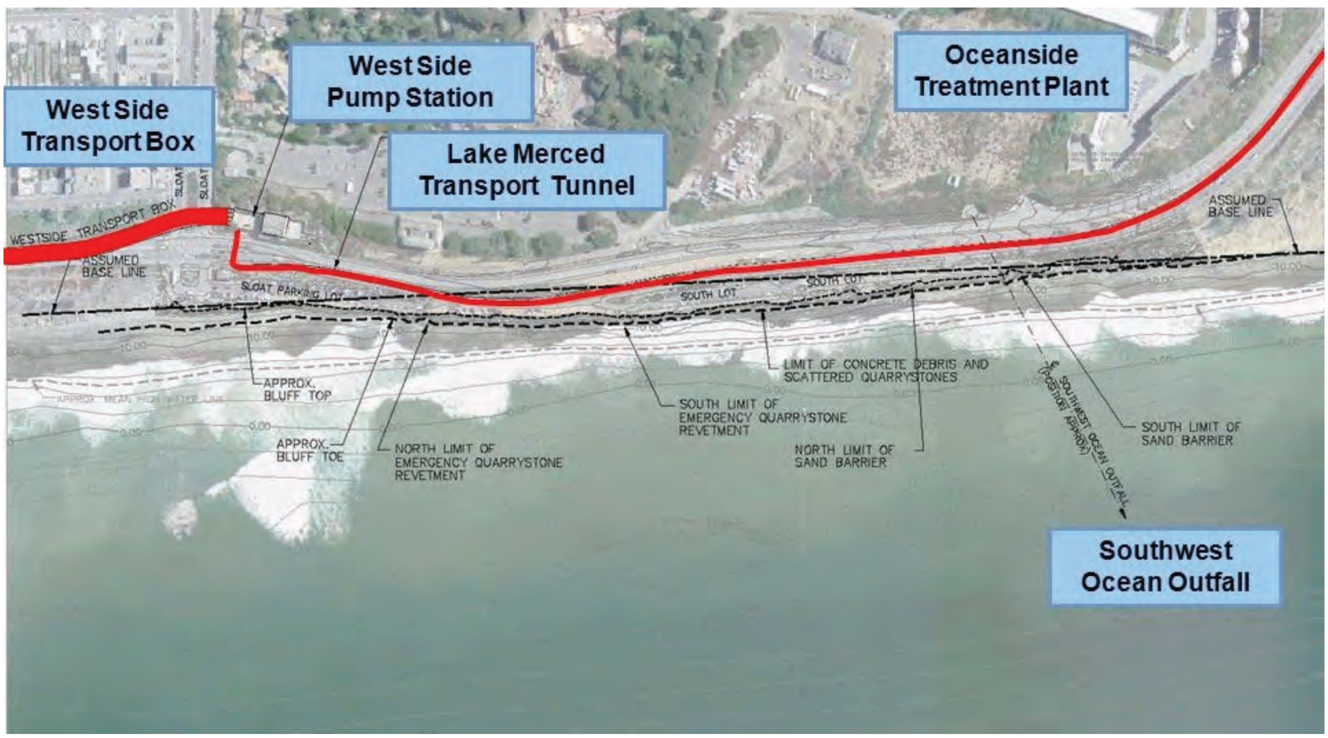

Figure ES-3: San Francisco West Wastewater Facilities

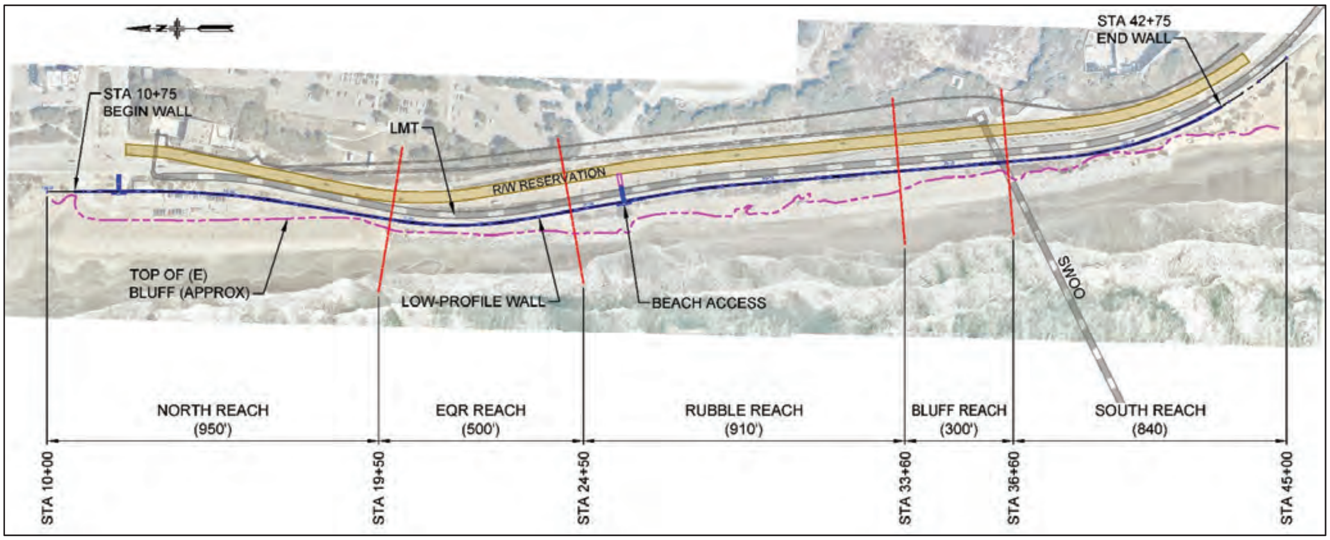

Figure ES-4: Low Profile Wall-Plan

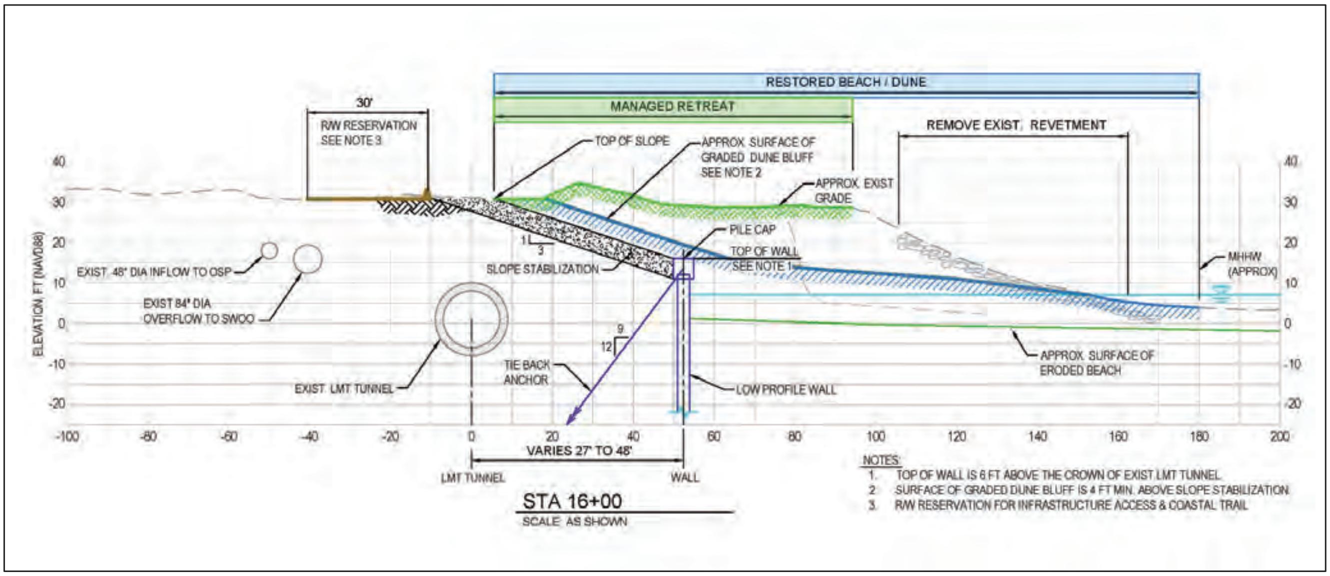

Figure ES-5: Low Profile Wall-Representative Section

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1. Introduction

1.1. Purpose and Need

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

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

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

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

1.2. Project Goals and Objectives:

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

Primary objectives that the proposed project intends to achieve are:

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

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

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

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

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

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

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

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

1.3. Scope of Proposed Project

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

The proposed scope of work presented in this report includes:

1) Installing a low-profile secant pile wall seaward of the LMT

2) Re-contouring the bluff at SOB and providing ongoing sand nourishment for the beach on an as-needed basis for increased recreational access

3) Removing the Great Highway between Sloat and Skyline Boulevards and completing intersection improvements at Sloat and the Great Highway and Skyline and the Great Highway to accommodate changed traffic flows.

4) Relocating the existing parking lot and restroom, currently located along the Great Highway, south of Sloat Boulevard.

5) Creating a multiuse recreational trail and access road for the SFPUC in place of the existing north bound lanes of the Great Highway

6) Providing access points to the beach for the public

7) Modifying the entrance to the zoo to accommodate changed traffic flows.

8) Modifying MTA bus turn-around at Sloat and Great Highway to account for changed traffic flows.

9) Providing landscaping and sand management strategies for the re-contoured bluff and the beach.

10) Removing the Existing Shoreline Revetments and Rubble

1.4. Approach to Developing Conceptual Design

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

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

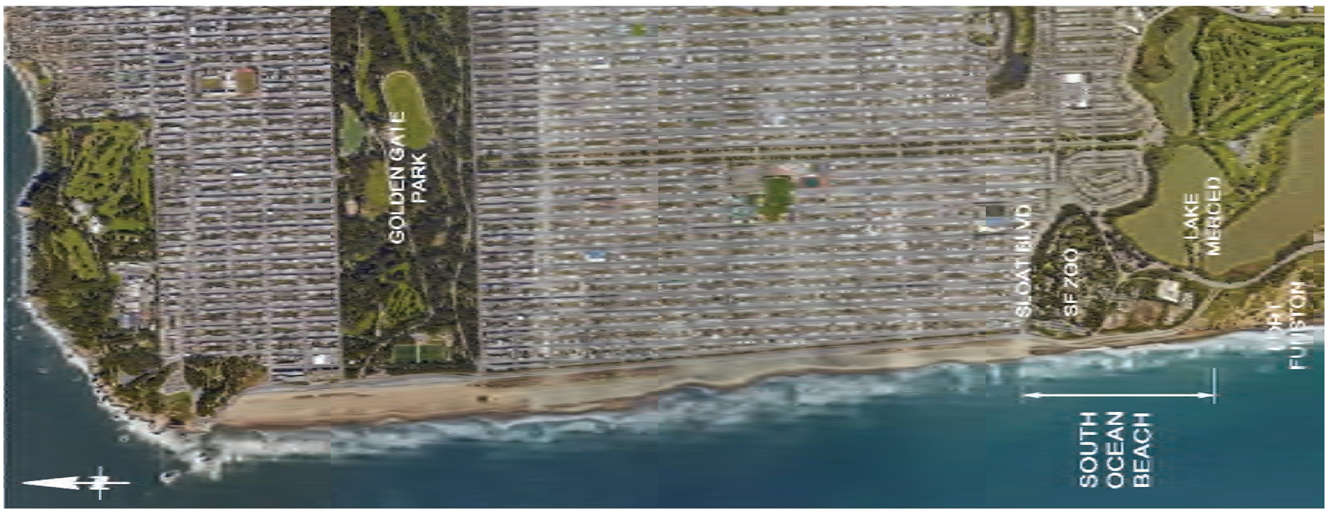

Figure 1-1: South Ocean Beach Location

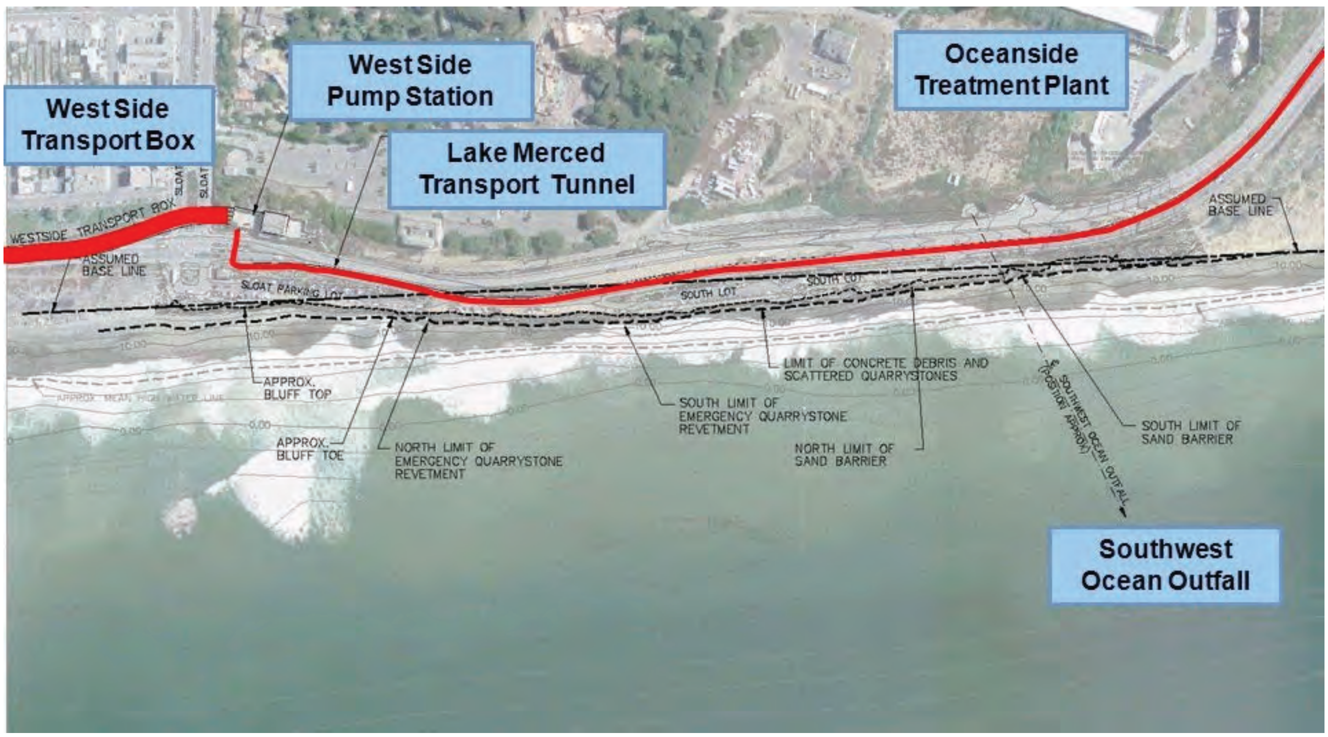

Figure 1-2: San Francisco West Wastewater Facilities

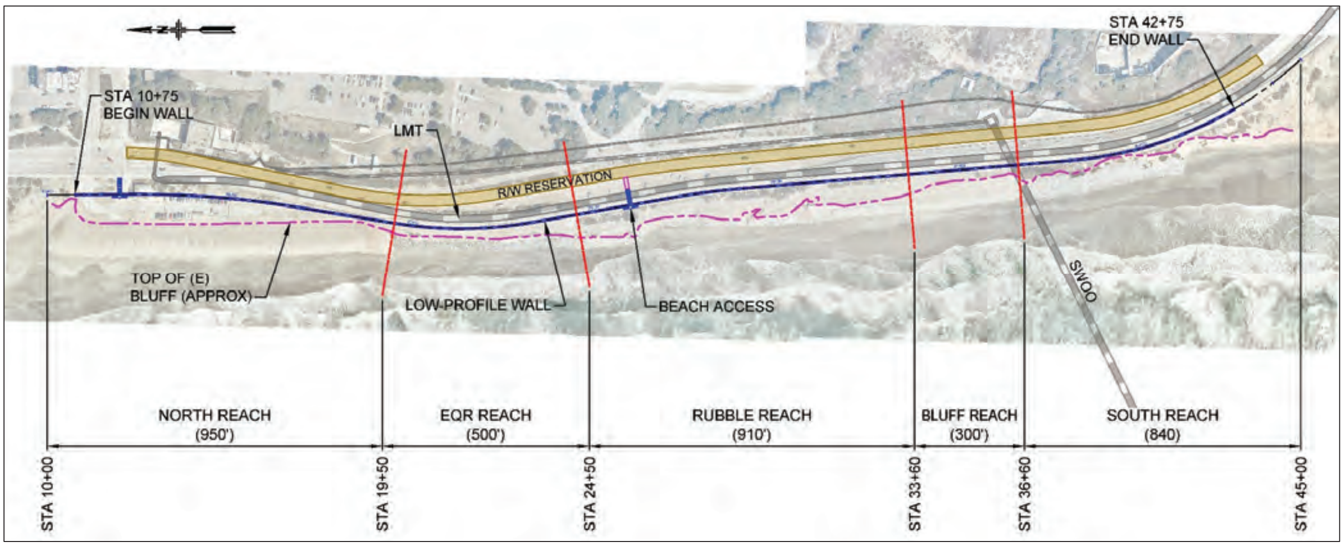

Figure 1-3: Low Profile Wall-Plan

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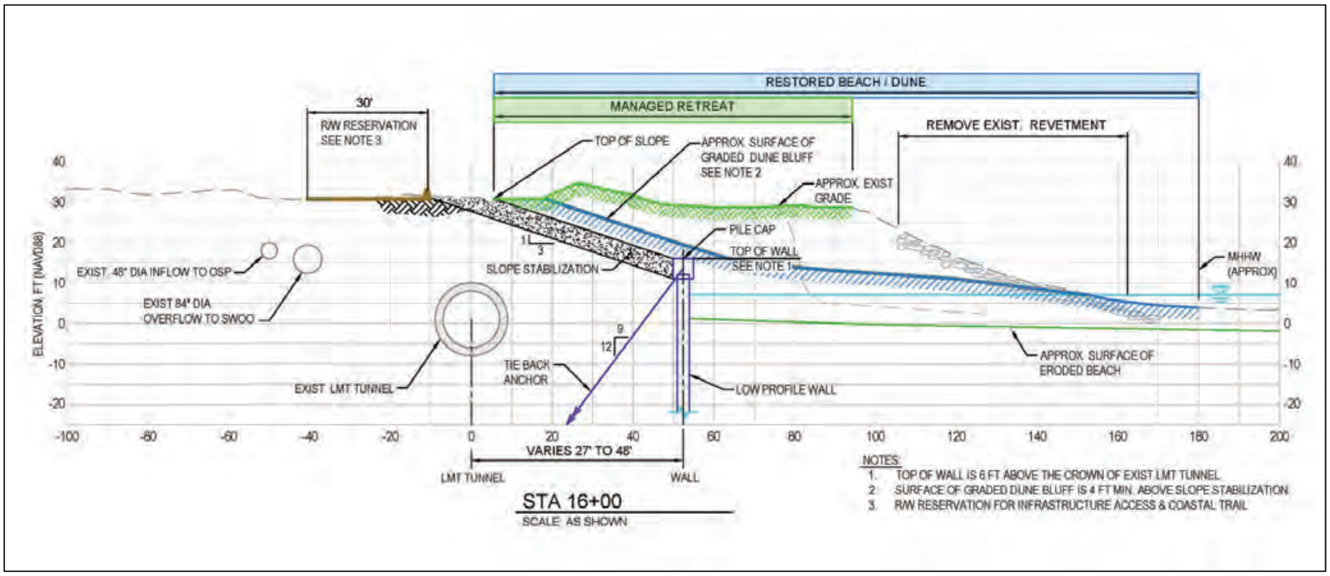

Figure 1-4: Low Profile Wall-Representative Section

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2. Background

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

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

2.1. Existing System

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

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

2.2. Current Operation

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

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

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

2.3. Summary of Needs

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

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

- beach access and recreation
- beach and dune habitat

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

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

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

2.4. Ocean Beach Master Plan

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

1. Reroute the Great Highway inland behind the San Francisco Zoo via Sloat Blvd. and Skyline Blvd., including:
   a) Closure of the Great Highway south of Sloat Blvd. and incorporation of a coastal trail,
   b) Reconfiguration of Sloat Blvd. and key intersections to create a safer, more efficient traffic flow,
   c) Consolidation of street parking, the L Taraval terminus and bicycle access along the south side of Sloat Blvd.; and
   d) Reconfiguration of the zoo's parking lot to permit access via Skyline Blvd. and Zoo Road.
2. Introduce a multipurpose coastal protection/restoration/access system, including:
   a) Managed retreat and phased removal of the Great Highway and adjoining parking lots, allowing erosion to proceed inland,
   b) Protection of the existing wastewater infrastructure including the LMT in place with a low-profile wall, a cobble berm, and beach nourishment with placed sand,
   c) Allowing storm surges to wash over the low-profile wall and dissipate toward higher ground; and
   d) Restore and revegetate the higher ground to enhance recreational and ecological functions.

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

2.5. South Ocean Beach Coastal Protection Measures & Management Strategy

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

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

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

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

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

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

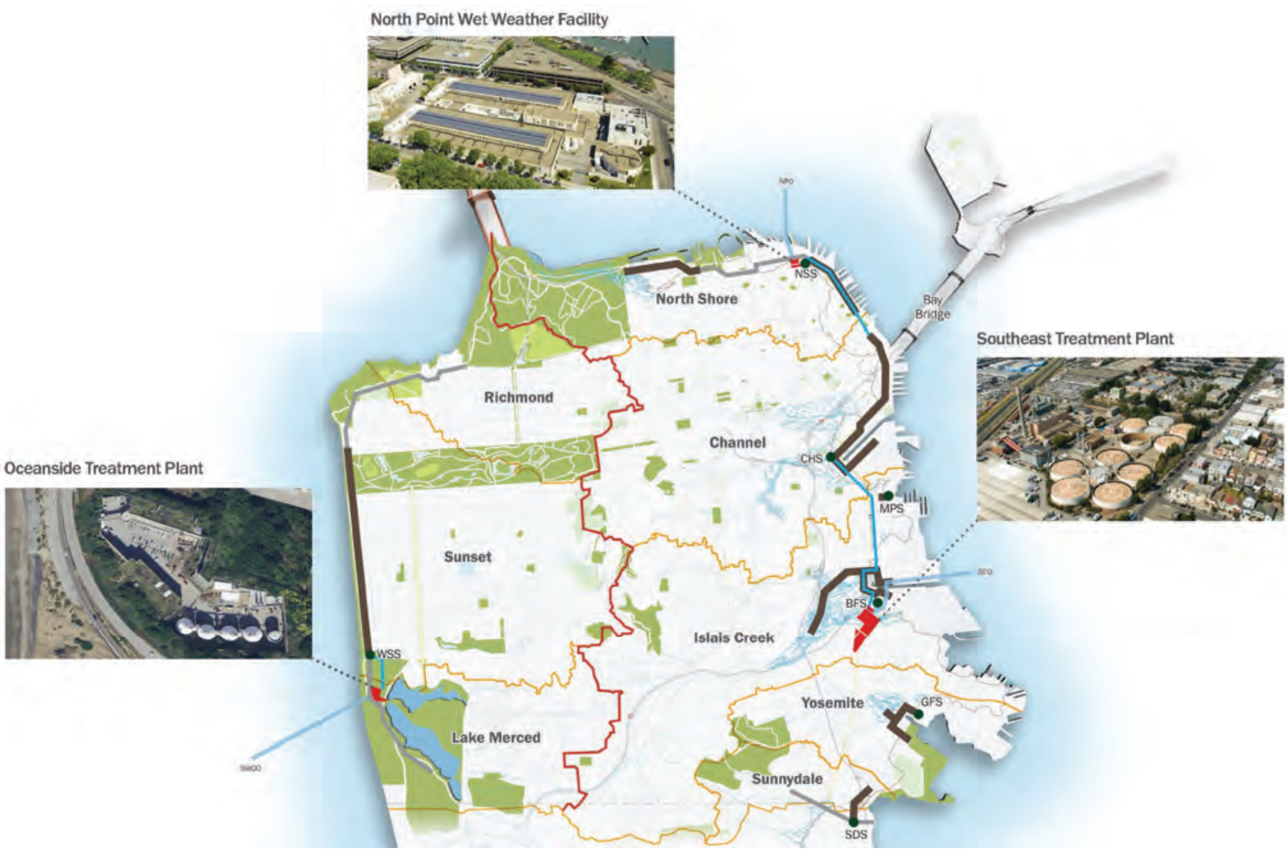

Figure 2-1: Wastewater Treatment Facilities Operated by SFPUC

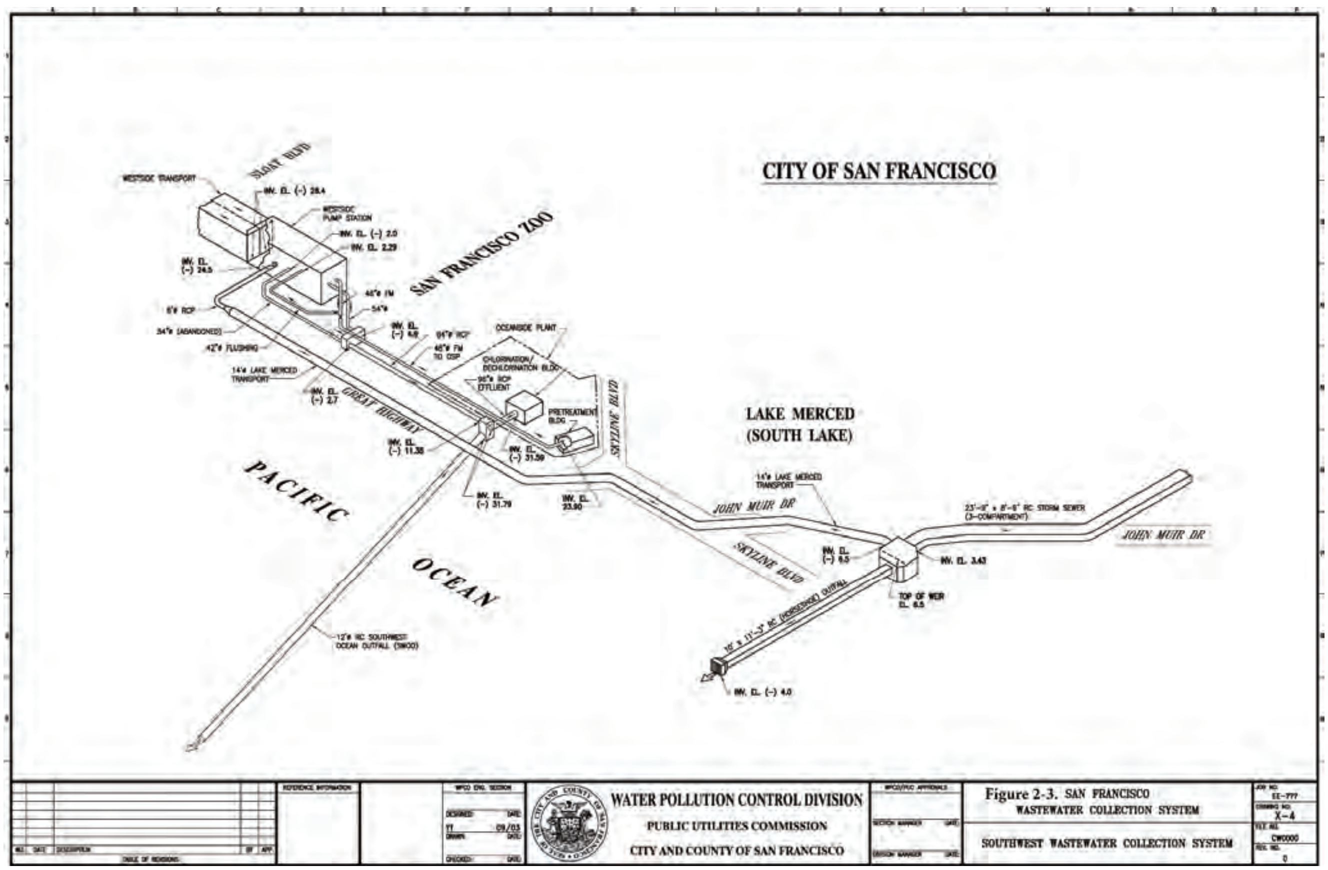

Figure 2-2: SFPUC Southwest Wastewater Collection System Schematic.

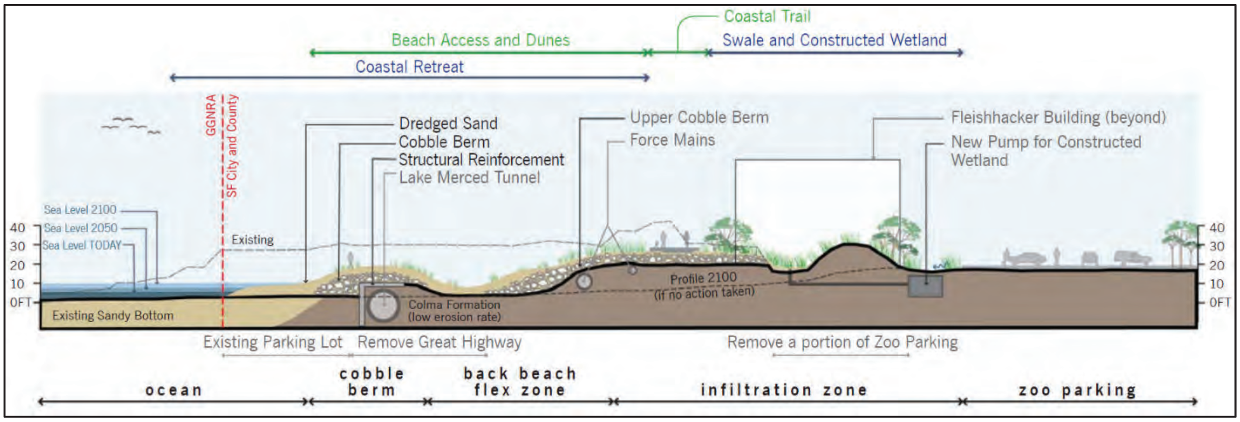

Figure 2-3: Summary of 2012 Master Plan Recommendations near Sloat Blvd/Great Highway Intersection

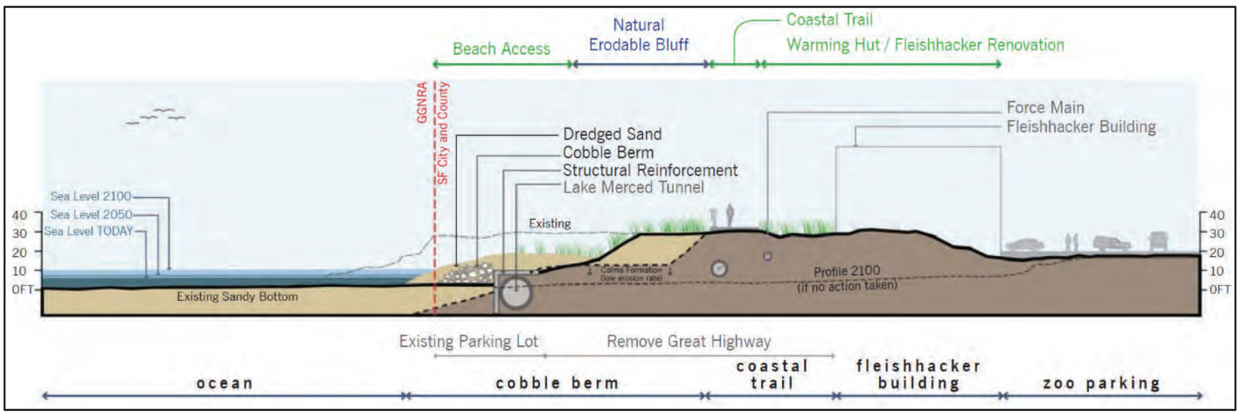

Figure 2-4: Summary of 2012 Master Plan Recommendations at Zoo Parking Lot

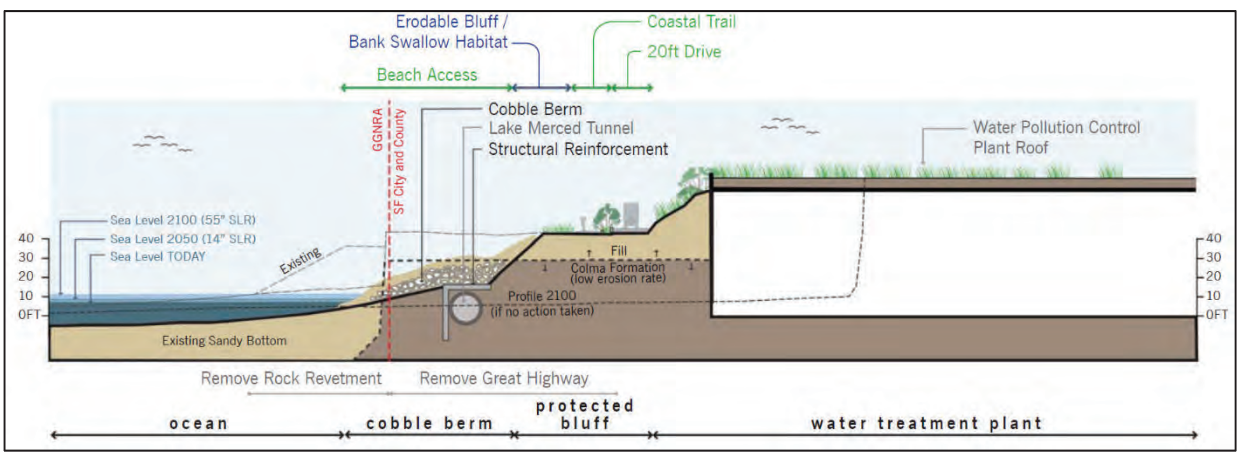

Figure 2-5: Summary of 2012 Master Plan Recommendations at Oceanside Water Pollution Control Plant.

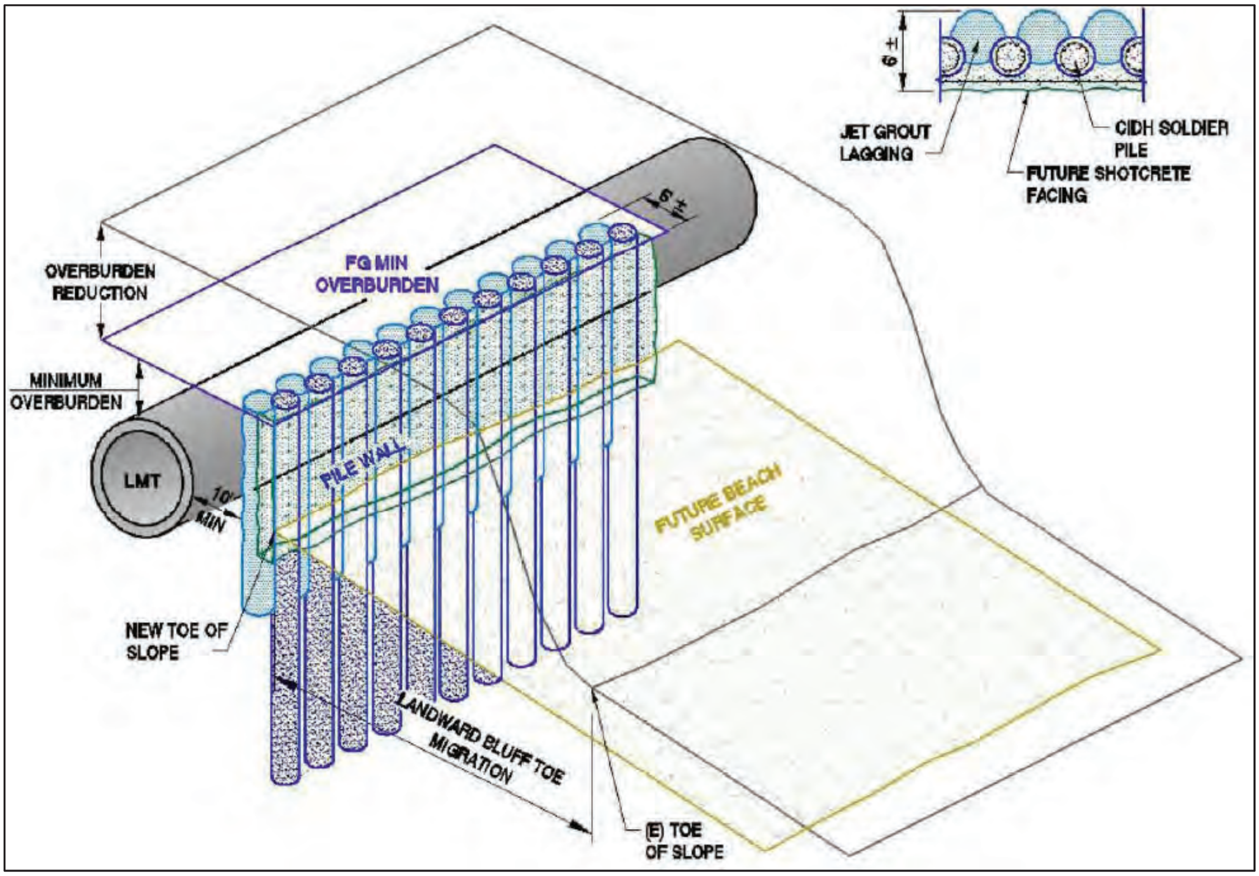

Figure 2-6: Isometric View of Reinforced Secant Pile Wall, reproduced from *SPUR (2015).*

3. Selected Alternative

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

3.1. Alternatives Analysis

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

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

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

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

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

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

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

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

Table 3-1: Summary of Evaluation Criteria Weighting

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

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

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

Table 3-2: Alternatives Scoring and Ranking

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

[1] Higher number indicates superior rank

3.2. Proposed Project

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

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

Table 3-3: Proposed Project Components

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

4. Coastal Evaluation

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

4.1. Historical Background

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

4.2. Area Geology and Morphology

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

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

4.2.1. Bluff Material

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

4.2.2. Beach Material

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

4.3. Bluff Retreat

4.3.1. Short-Term Bluff Recession Rates

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

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

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

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

4.3.2. Long-Term Bluff Recession Rates

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

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

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

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

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

4.4. Erosion Patterns

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

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

4.5. Sea-Level Rise

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

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

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

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

4.6. Sea-Level Rise Scenarios

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

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

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

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

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

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

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

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

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

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

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

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

4.6.1. Trends in Local Relative Sea Level

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

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

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

4.7. Coastal Engineering Design

4.7.1. Design High Water Level

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

Table 4-3: Design High Water Level.

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

4.7.2. Wave Action

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

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

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

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

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

4.7.3. Scour Elevations

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

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

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

4.7.4. Wave Runup

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

4.7.5. Wave Loads

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

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

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

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

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

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

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

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

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

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

4.7.6. Scour at Wall Crest

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

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

4.7.7. Beach Nourishment

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

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

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

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

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

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

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

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

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

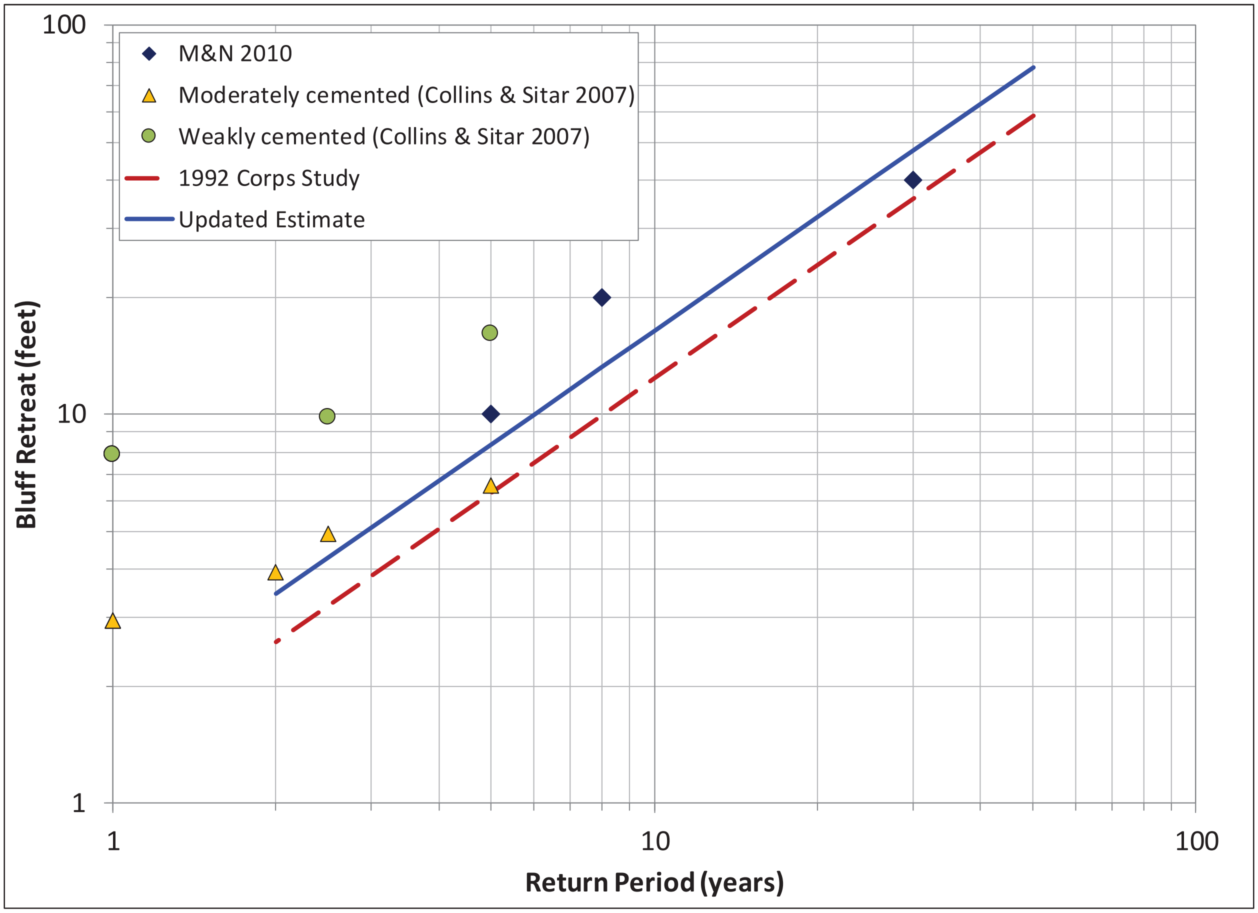

Figure 4-1: Estimated Return Period Bluff Retreat Values

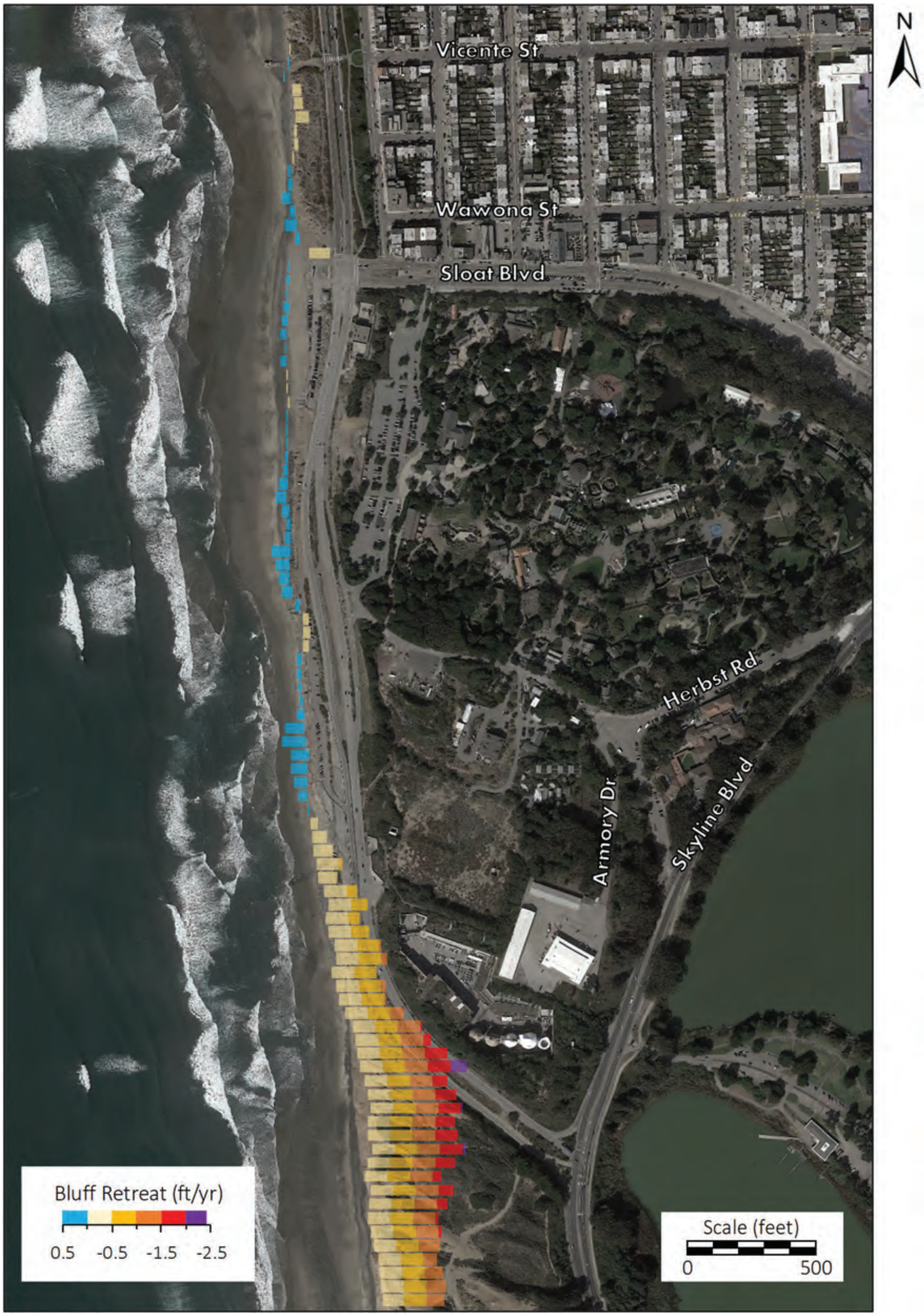

Figure 4-2: South Ocean Beach Bluff Retreat Rates.

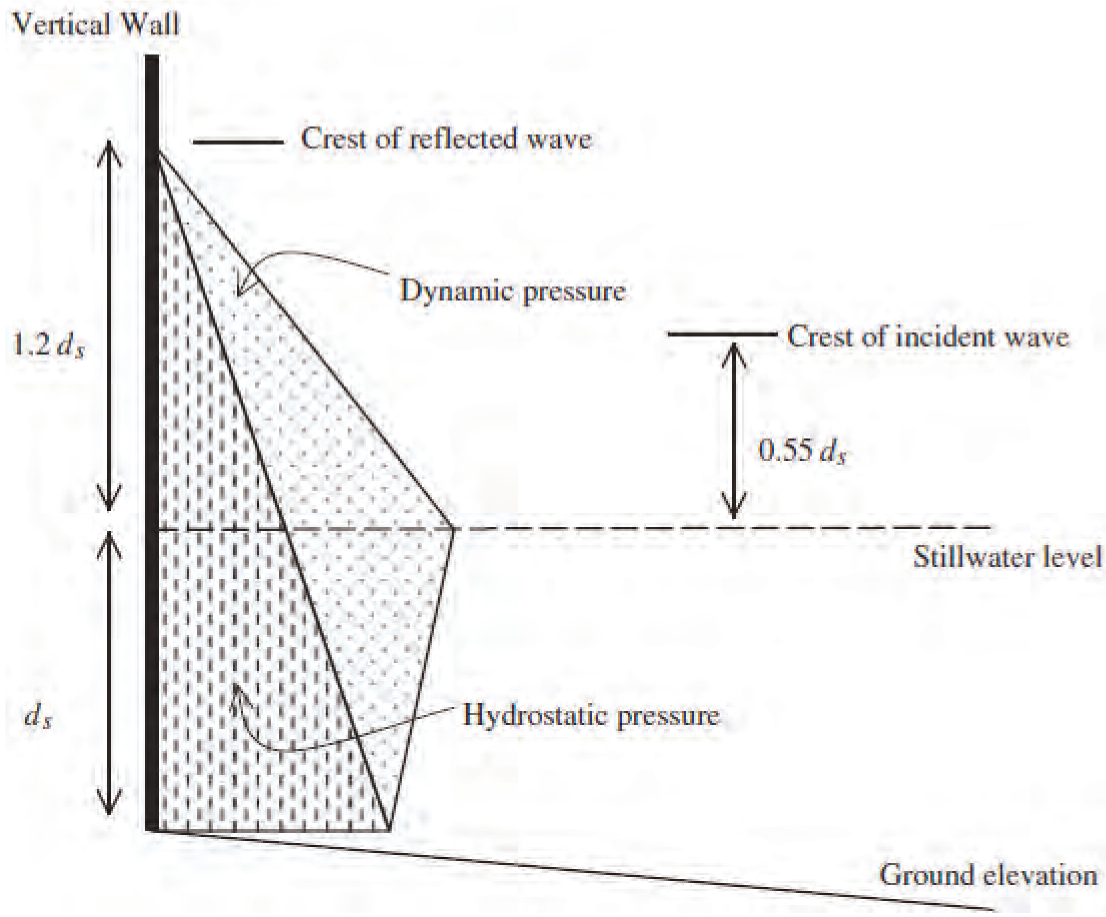

Figure 4-3: Normally Incident Wave Breaking Pressures, *ASCE (2016).*

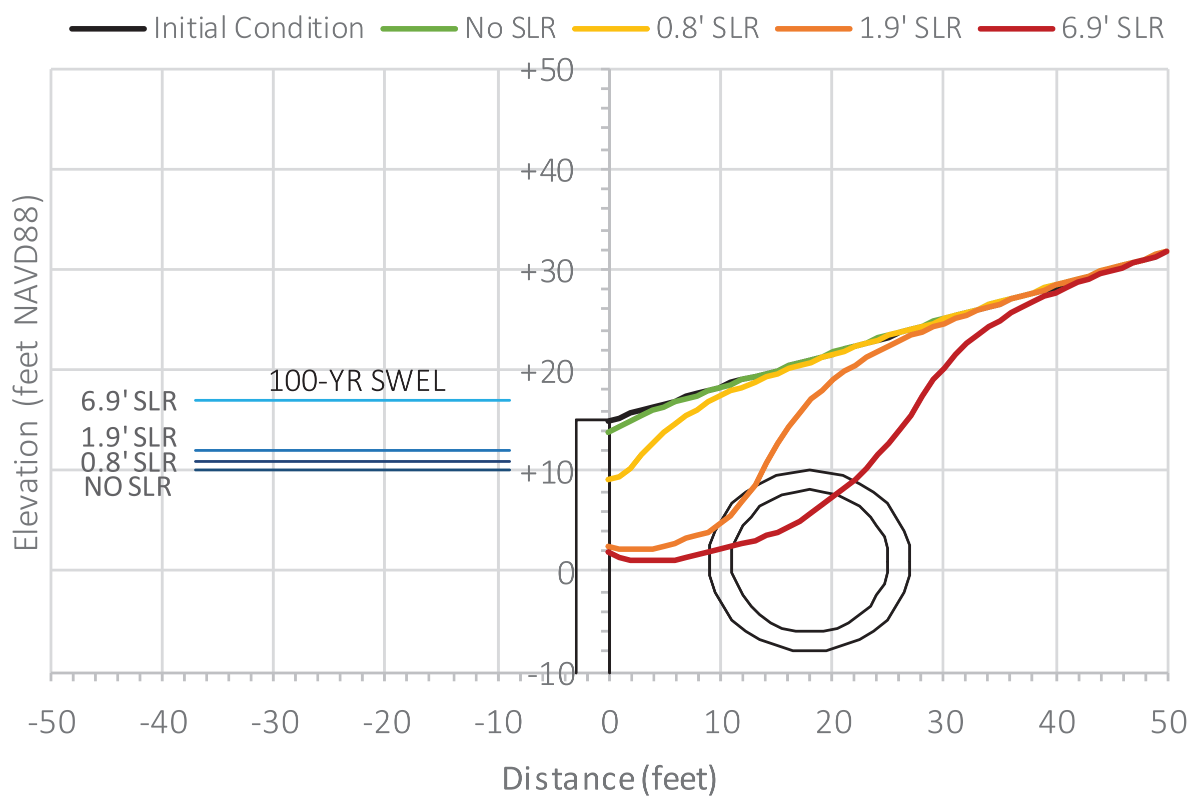

Figure 4-4: Assessment of Potential Scour of Slope above Wall Crest (No Slope Protection).

5. Geotechnical Evaluation

The Geotechnical Assessment Report is provided in Appendix B.

5.1. Introduction

5.1.1. General

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

5.1.2. Project Elements

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

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

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

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

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

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

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

Table 5-1: Reach Descriptions

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

5.1.3. Existing Data Review

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

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

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

5.1.4. Field Exploration Program

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

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

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

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

5.1.5. Geotechnical Laboratory Testing Program

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

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

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

5.1.6. Environmental Laboratory Testing Program

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

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

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

5.1.7. Codes and Standards

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

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

5.2. Initial Geotechnical Findings

5.2.1. Site Geology

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

Artificial Fill (Qaf)

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

Dune Sand (Qd)

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

Beach Sand (Qb)

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

Colma Formation (Qc)

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

Merced Formation (Qm)

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

5.2.2. Faulting and Seismicity

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

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

5.2.3. Groundwater

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

5.3. Preliminary Geotechnical Recommendations

5.3.1. Seismic Design Criteria

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

Table 5-2: Recommended Acceleration Response Spectrum

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

5.3.2. Liquefaction

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

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

- Magnitude 8.05 earthquake;
- PGA_M 1.02g; and

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

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

5.3.3. Tsunami

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

5.3.4. Secant Pile Wall

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

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

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

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

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

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

5.3.5. Lateral Earth Pressures

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

Table 5-3: Soil Properties for Lateral Earth Pressures

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

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

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

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

5.3.6. Tiebacks

Design Criteria

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

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

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

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

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

Testing and Acceptance Criteria

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

Tieback-induced Downdrag Force

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

5.3.7. Controlled Low Strength Material (CLSM)

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

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

5.3.8. Earthwork

Site Preparation

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

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

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

Excavation Characteristics

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

Unshored Excavations

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

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

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

<u>Fills and Backfills</u>

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

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

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

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

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

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

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

5.3.9. Dewatering and Groundwater Considerations During Construction

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

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

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

5.3.10. Flexible Pavement

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

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

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

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

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

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

5.3.11. Corrosion Potential

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

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

6. Civil

6.1. Background

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

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

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

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

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

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

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

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

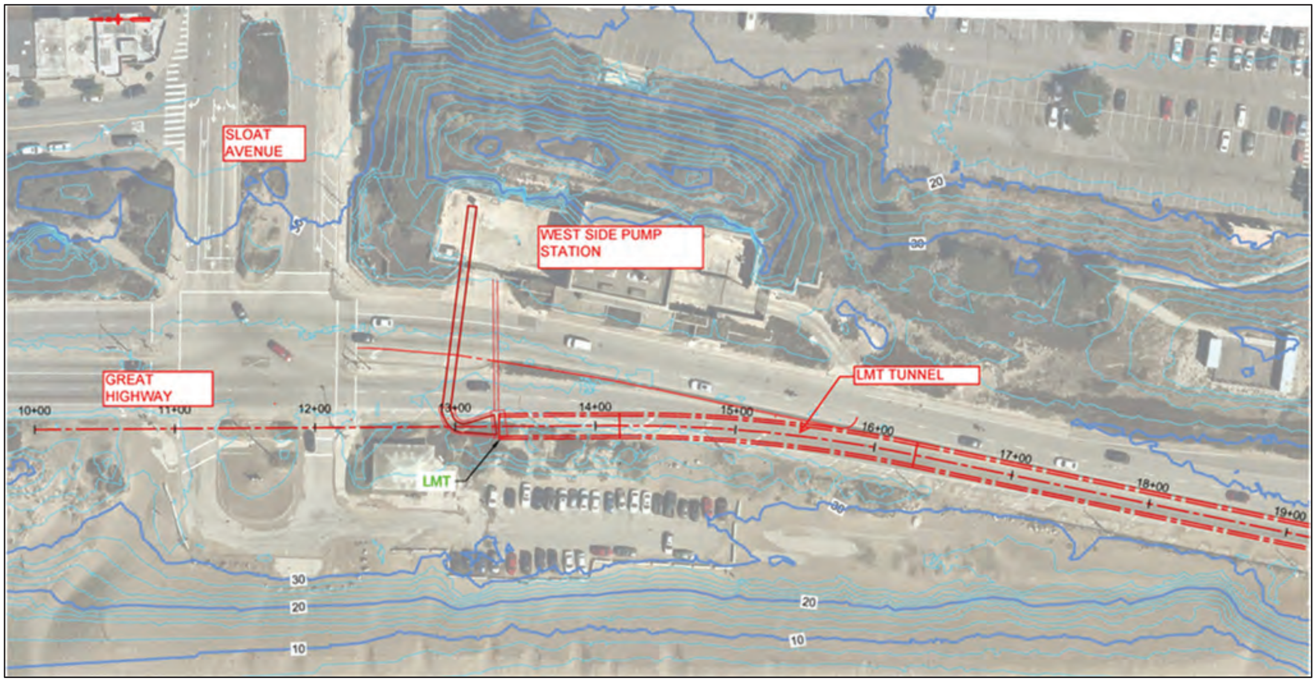

Figure 6-1: Project Site Plan, 1 of 4.

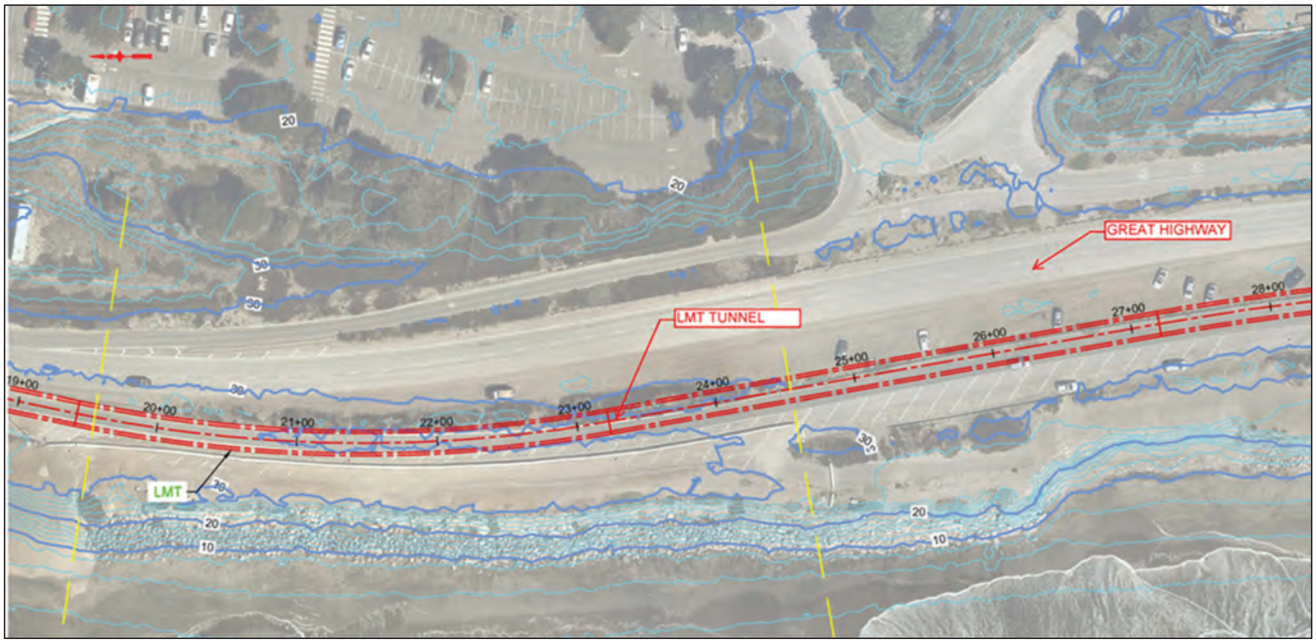

Figure 6-2: Project Site Plan, 2 of 4.

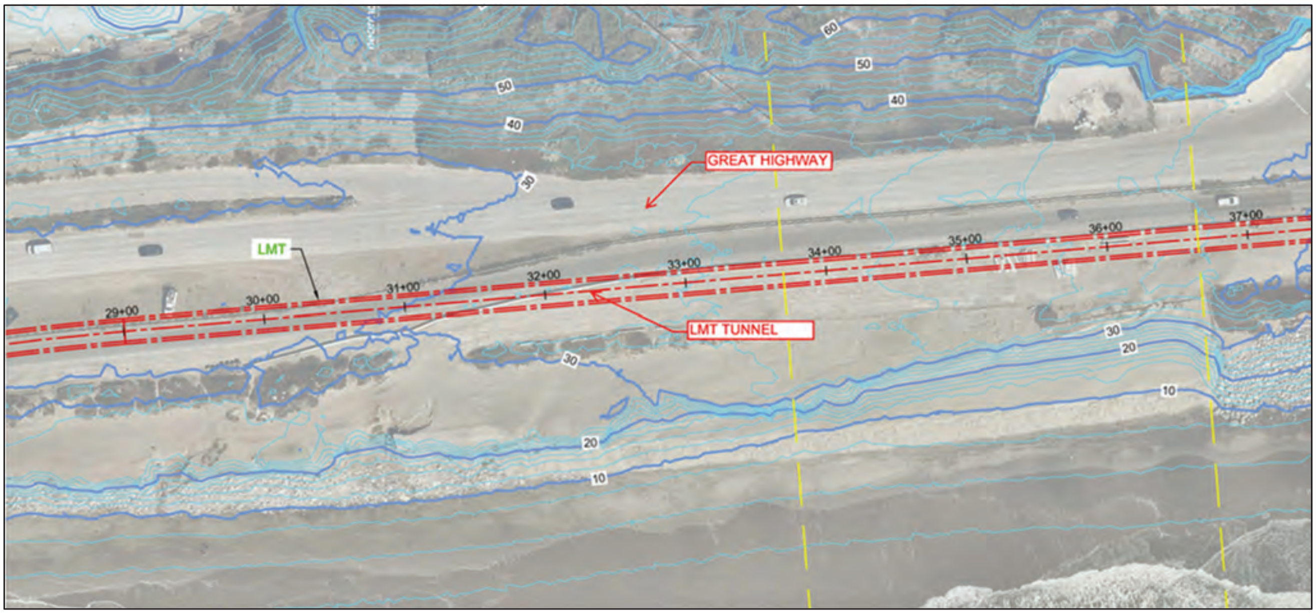

Figure 6-3: Project Site Plan, 3 of 4.

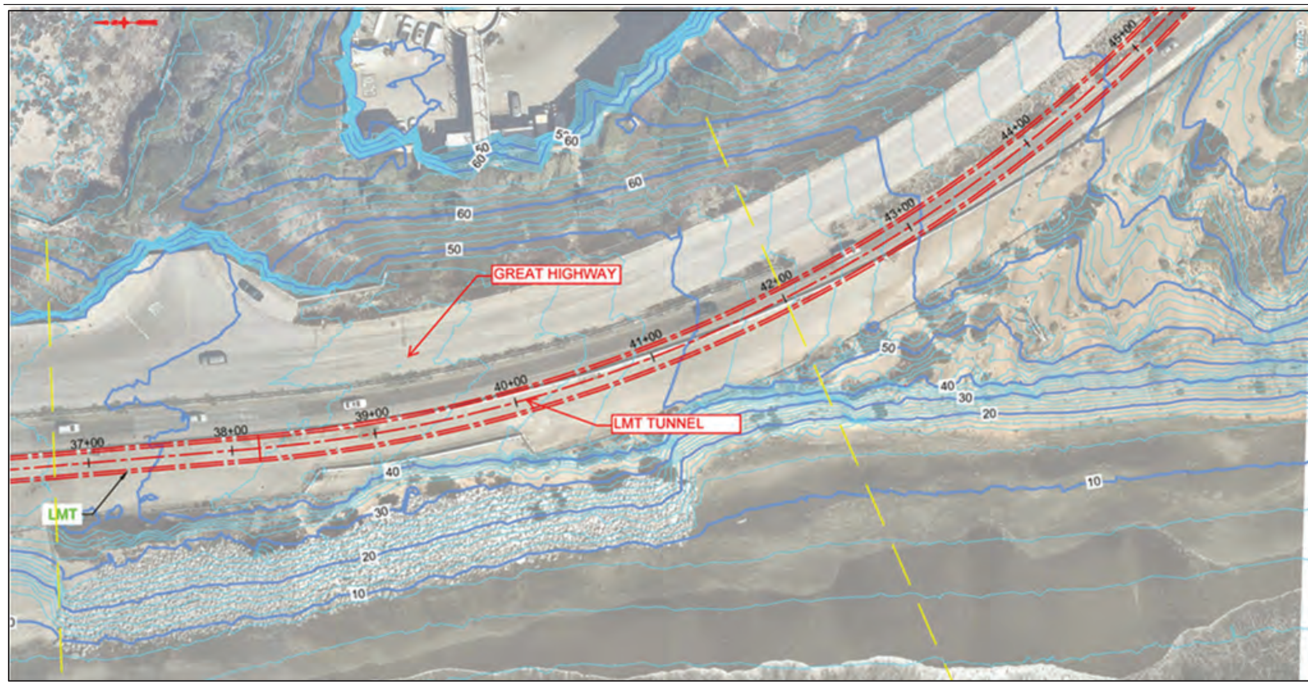

Figure 6-4: Project site Plan, 4 of 4.

6.2. Vertical Datum Reference

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

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

Table 6-1: Relationship between Vertical Datums.

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

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

6.3. LMT Tunnel Alignment

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

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

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

6.4. Proposed Wall Alignment

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

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

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

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

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

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

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

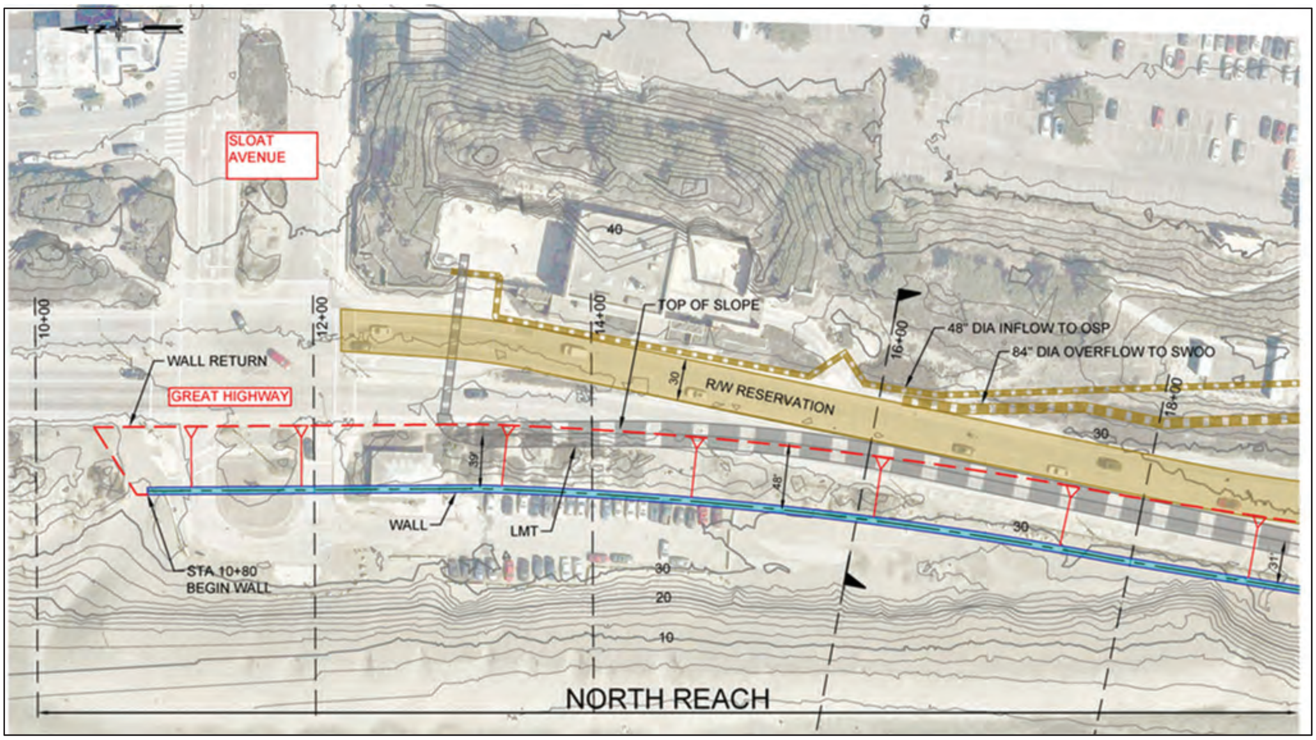

Figure 6-5: Wall Alignment Plan, 1 of 4.

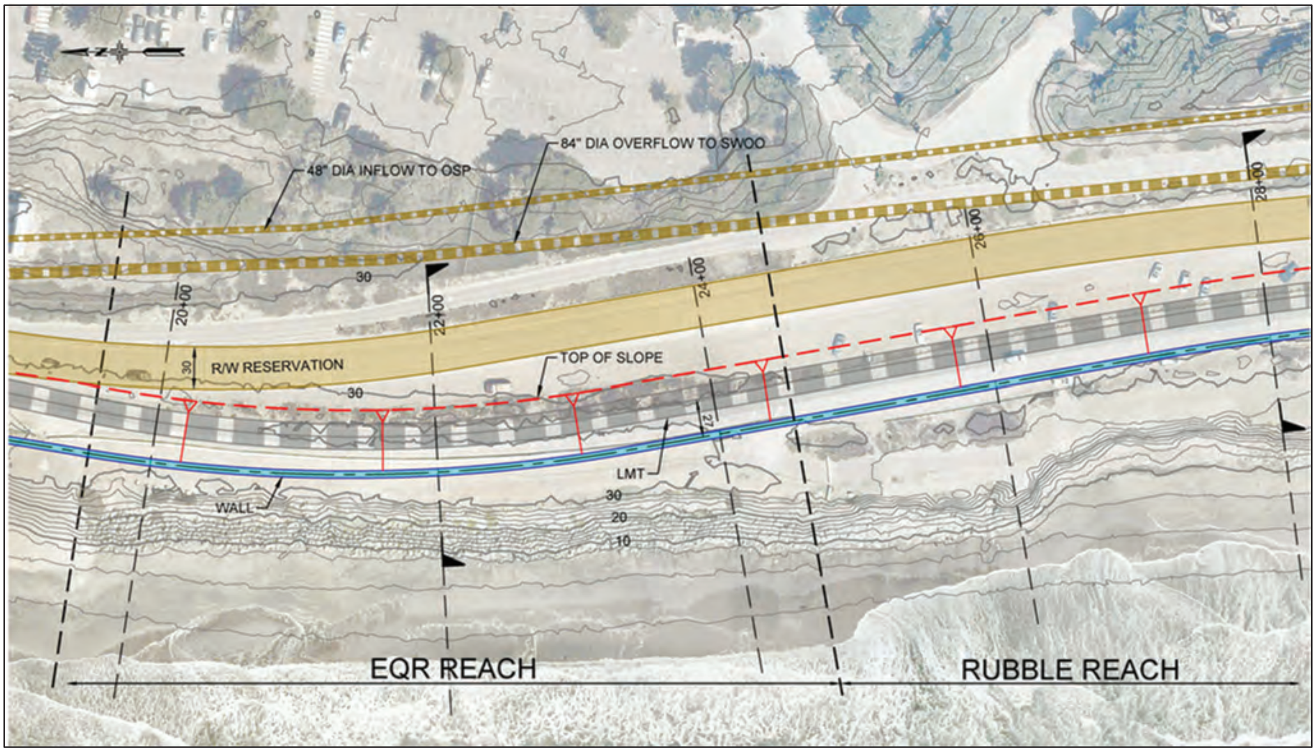

Figure 6-6: Wall Alignment Plan, 2 of 4.

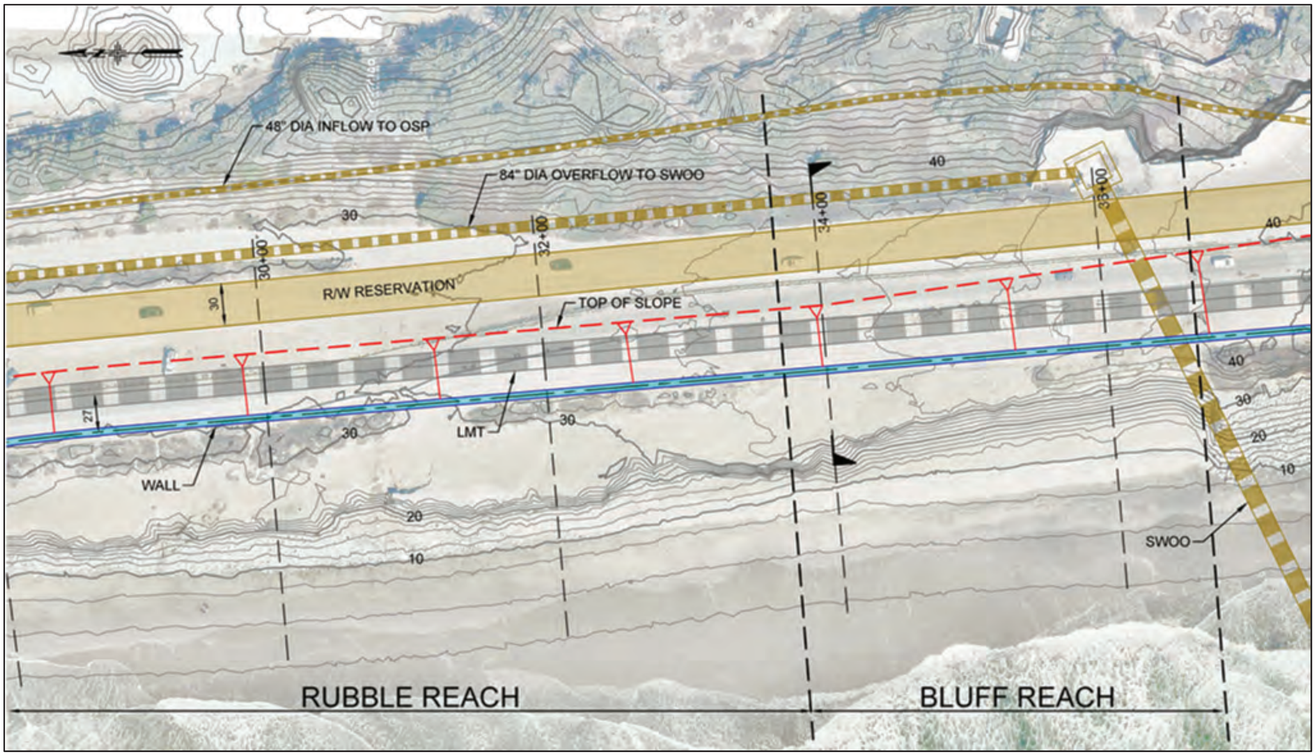

Figure 6-7: Wall Alignment Plan, 3 of 4.

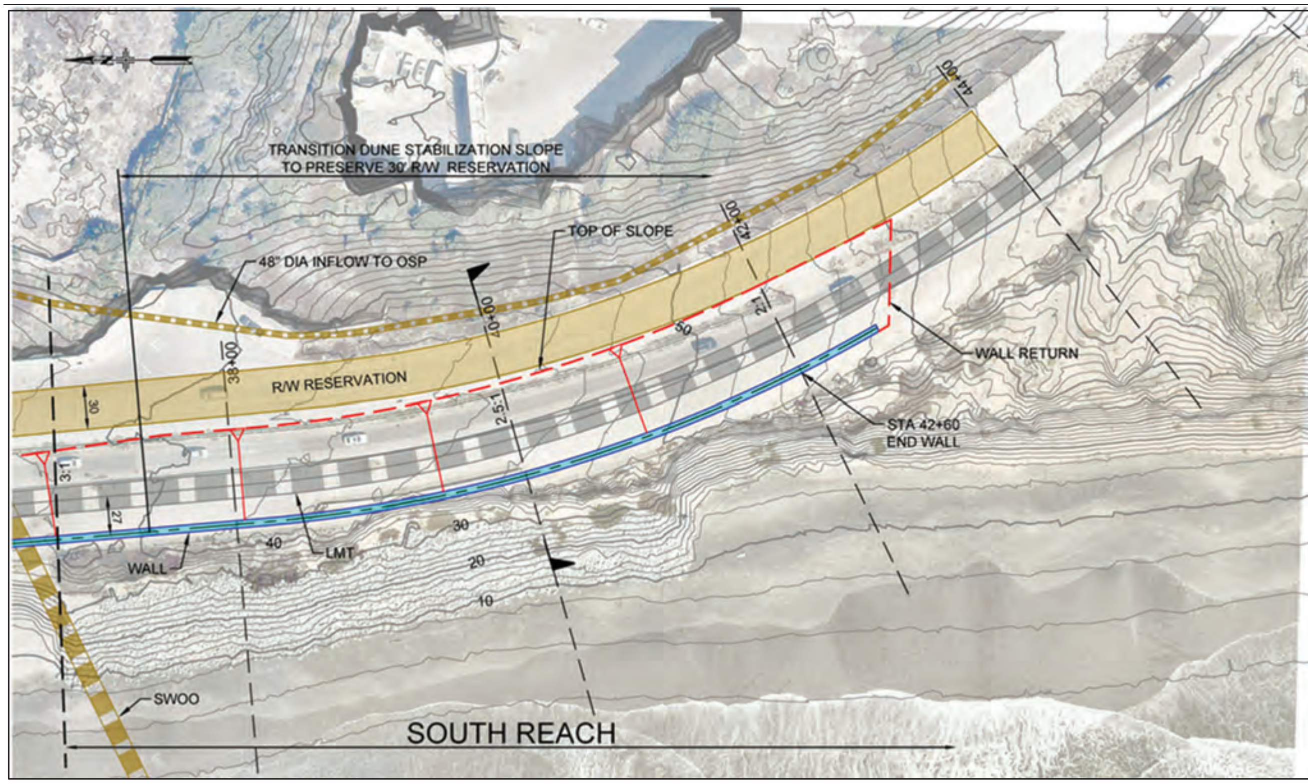

Figure 6-8: Wall Alignment Plan, 4 of 4.

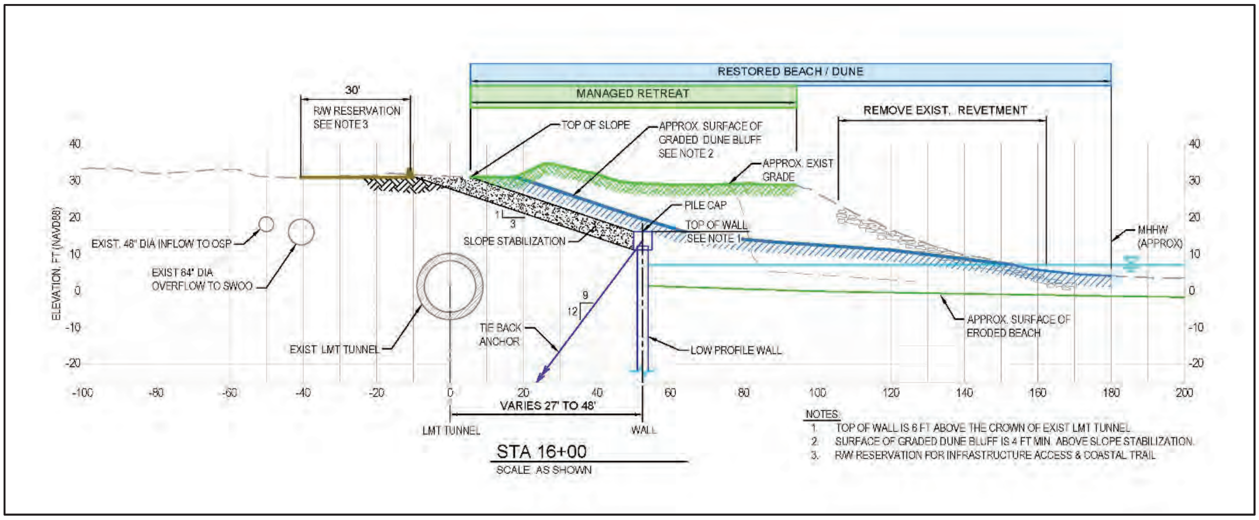

Figure 6-9: Low Profile Wall – North Reach Typical Section.

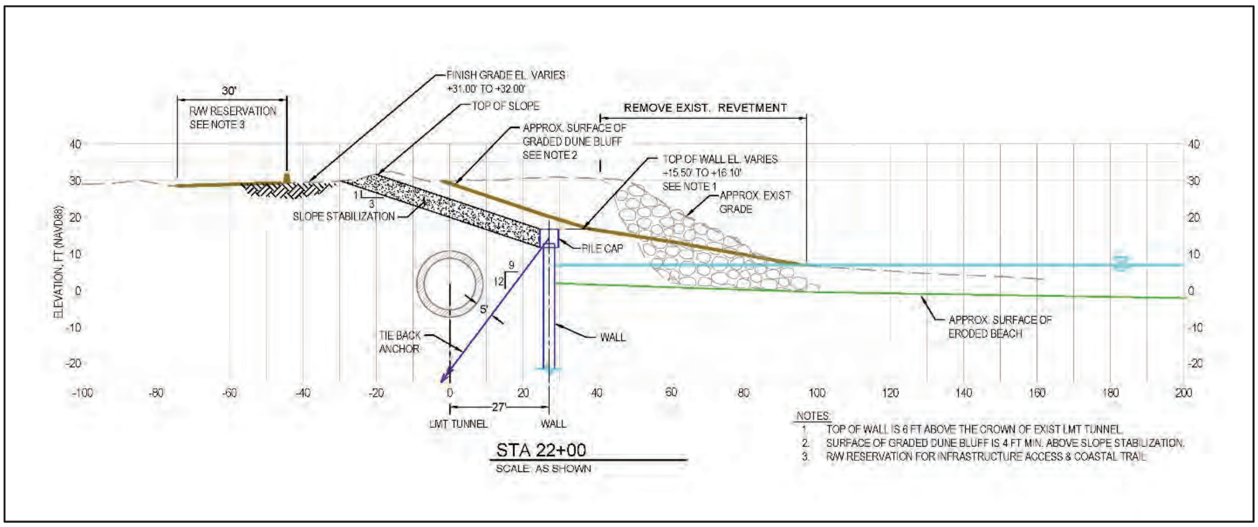

Figure 6-10: EQR Reach Typical Section.

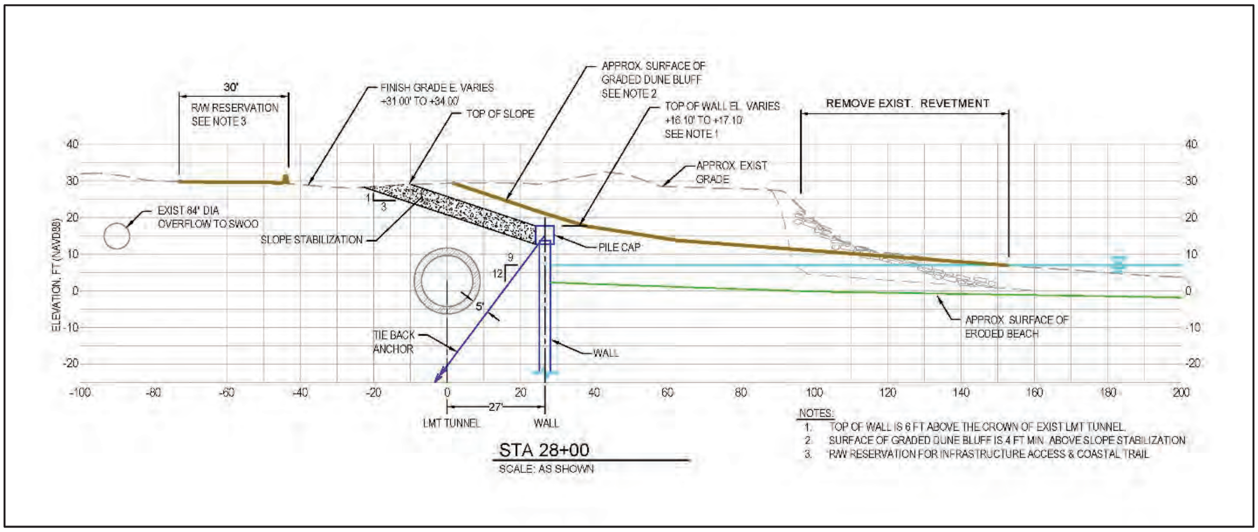

Figure 6-11: Rubble Reach Typical Section.

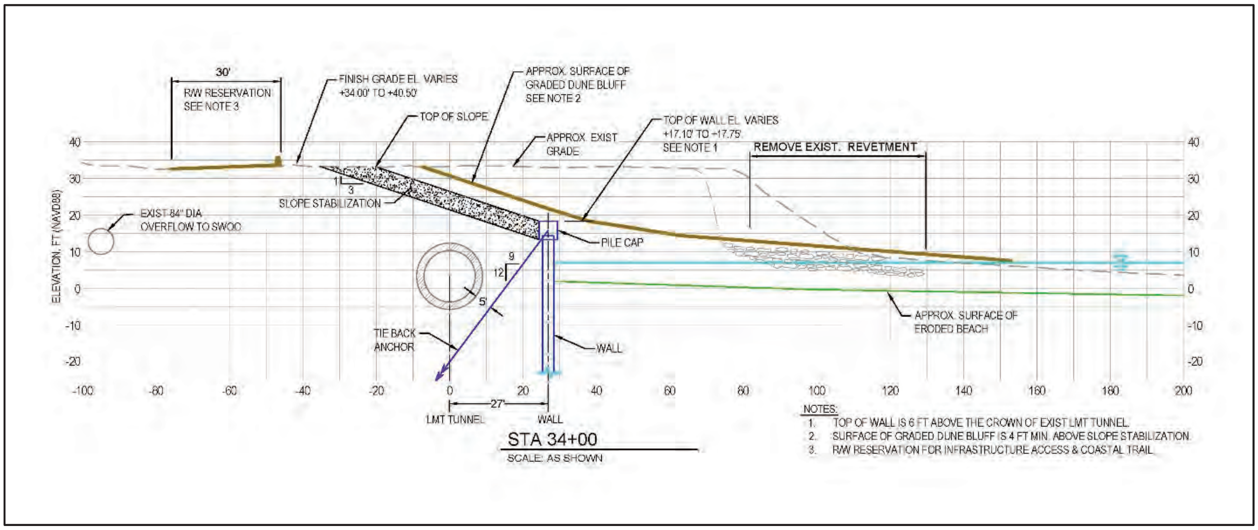

Figure 6-12: Bluff Reach Typical Section.

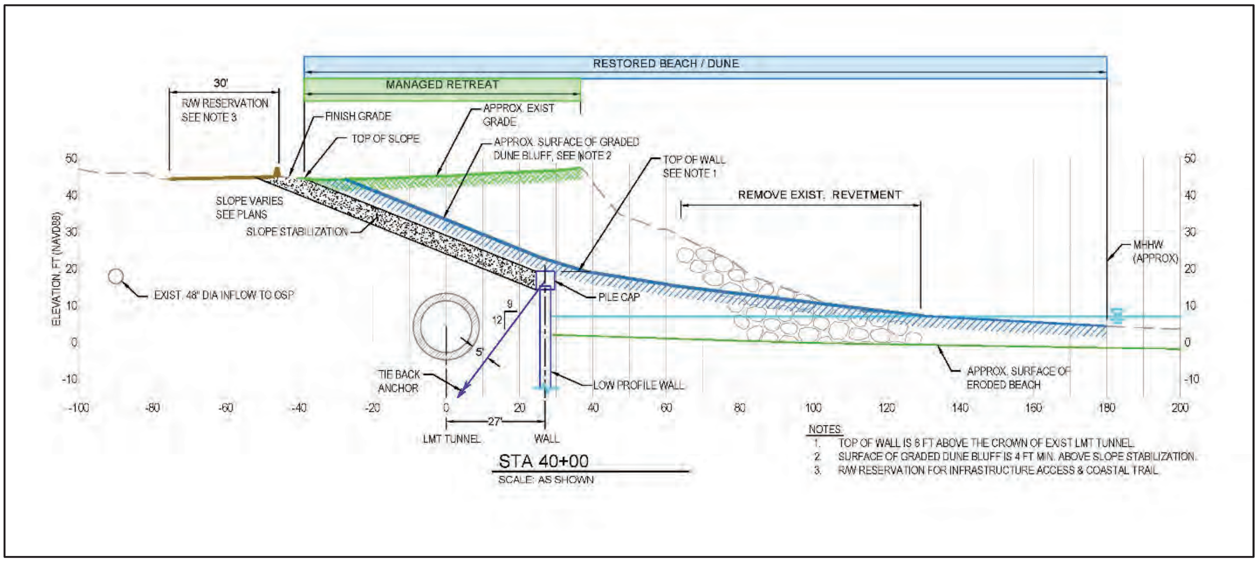

Figure 6-13: South Reach Typical Section.

6.5. Existing Utilities

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

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

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

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

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

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

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

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

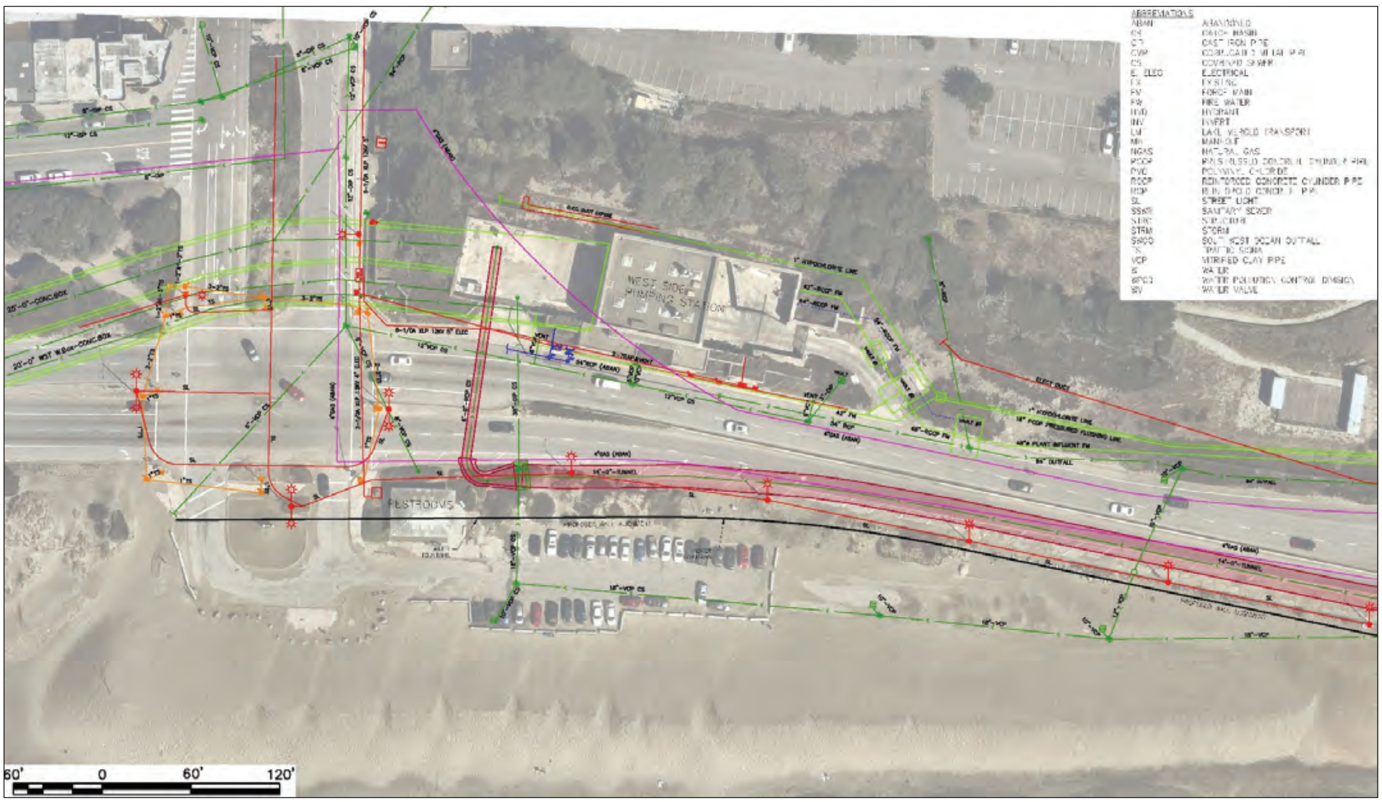

Figure 6-14: Existing Utilities Plan 1.

Figure 6-15: Existing Utilities Plan 2.

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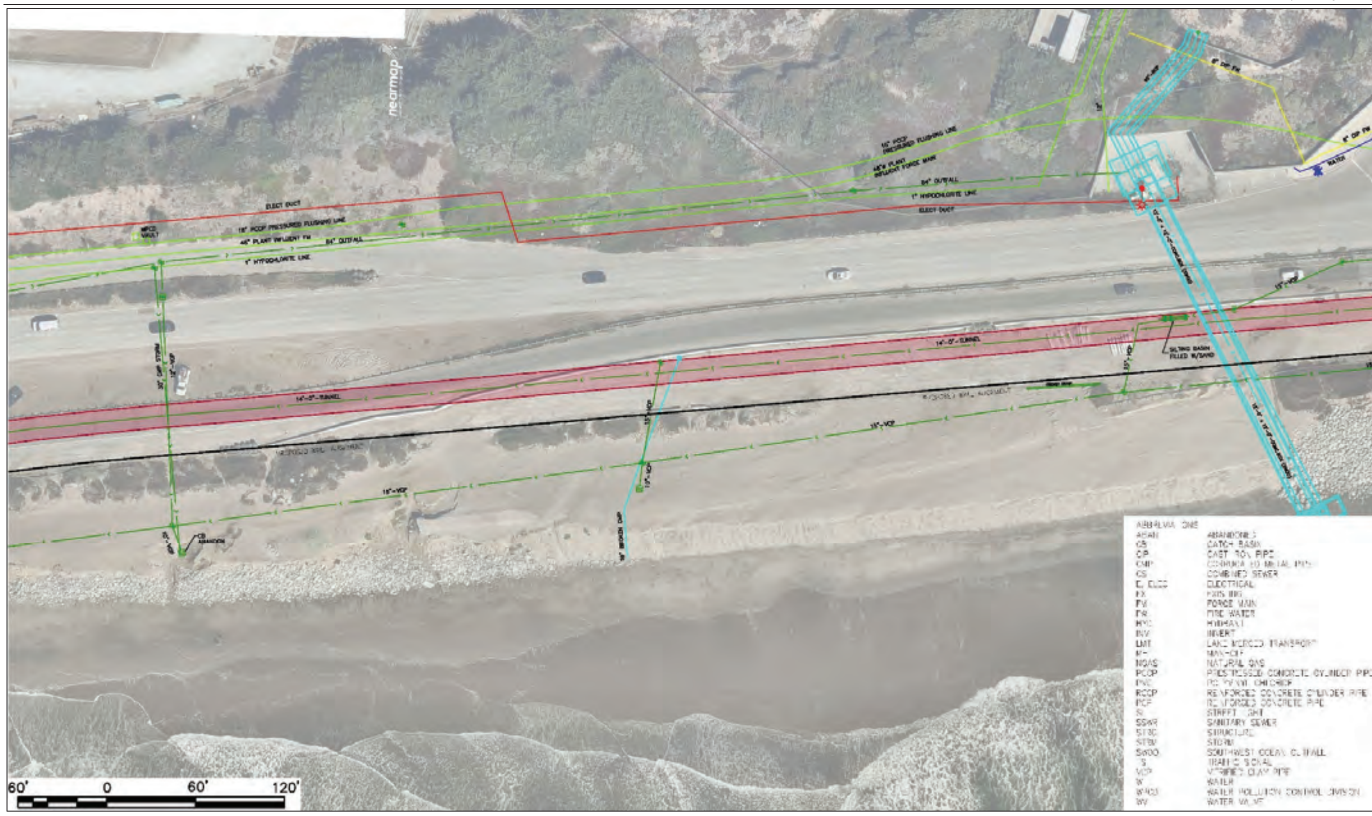

Figure 6-16: Existing Utilities Plan 3.

Figure 6-17: Existing Utilities Plan 4.

7. Traffic and Intersection Layout

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

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

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

7.1. Existing Conditions

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

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

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

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

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

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

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

7.2. Great Highway at Sloat Boulevard Intersection Redesign

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

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

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

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

7.2.1. Intersection Layout

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

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

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

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

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

Intersection Option 1: Zoo Access Expanded to Three Lanes

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

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

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

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

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

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

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

<u>Muni Bus Operations</u>

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

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

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

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

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

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

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

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

7.3. Great Highway at Skyline Boulevard Intersection

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

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

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

7.3.1. Intersection Layout

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

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

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

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

7.4. Activate Zoo Road

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

7.4.1. Access Concept

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

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

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

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

7.5. Design Criteria for Traffic and Intersections

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

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

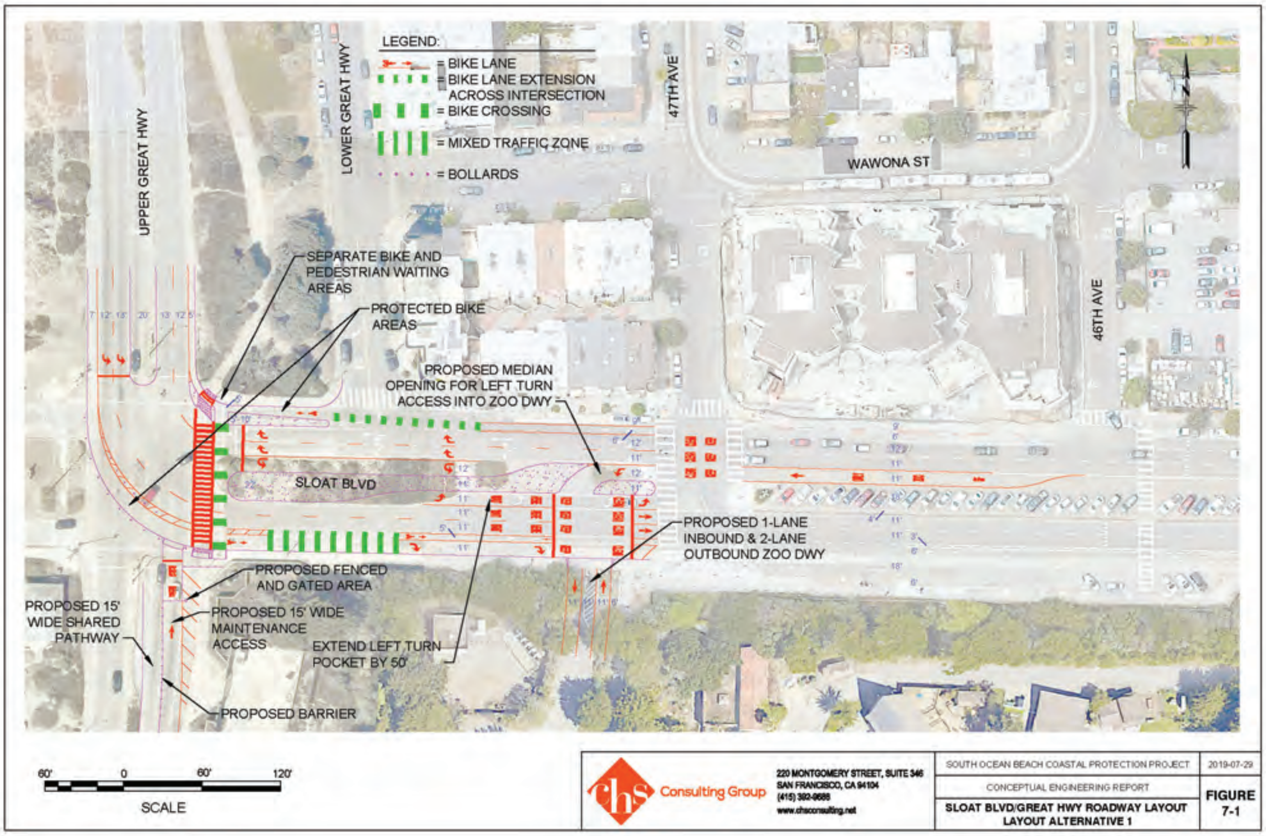

Figure 7-1: Great Highway at Sloat Boulevard Intersection – Alternative 1

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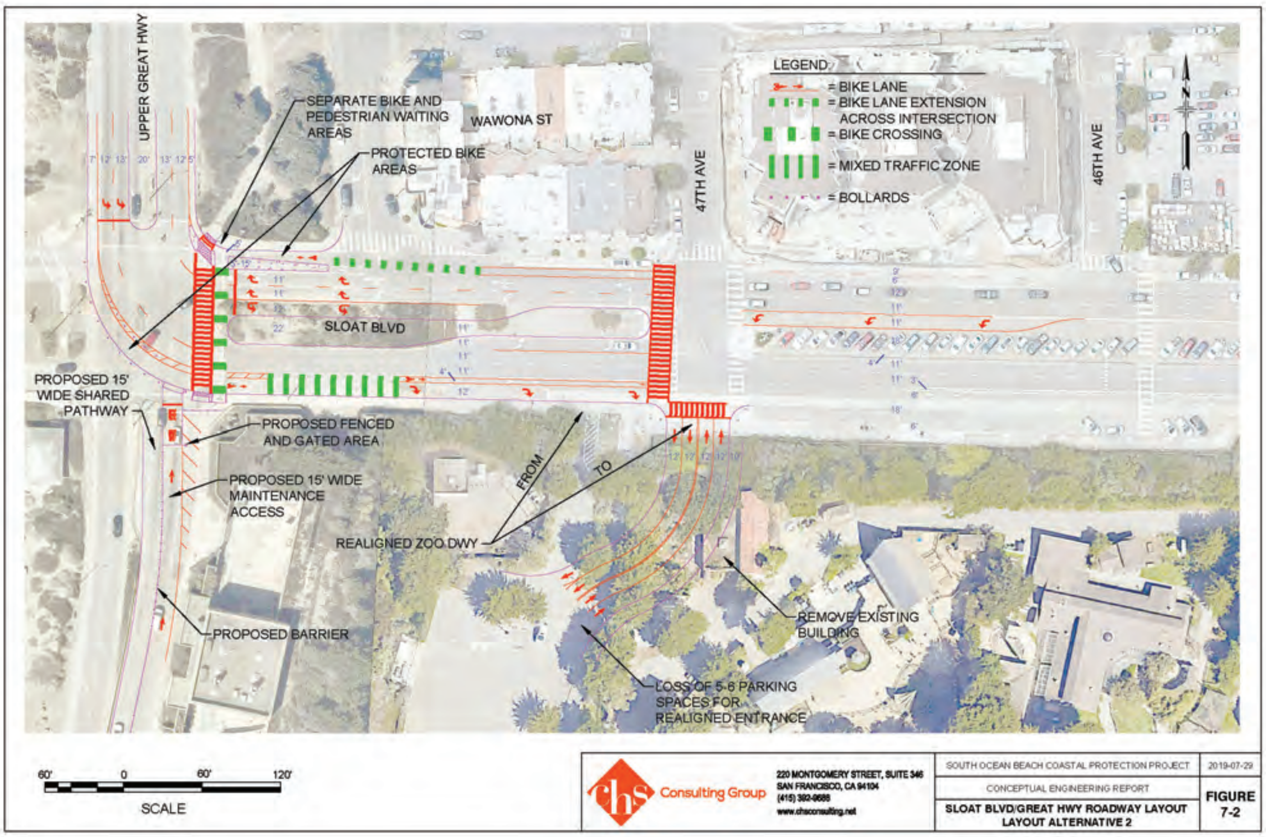

Figure 7-2: Great Highway at Sloat Boulevard Intersection – Alternative 2

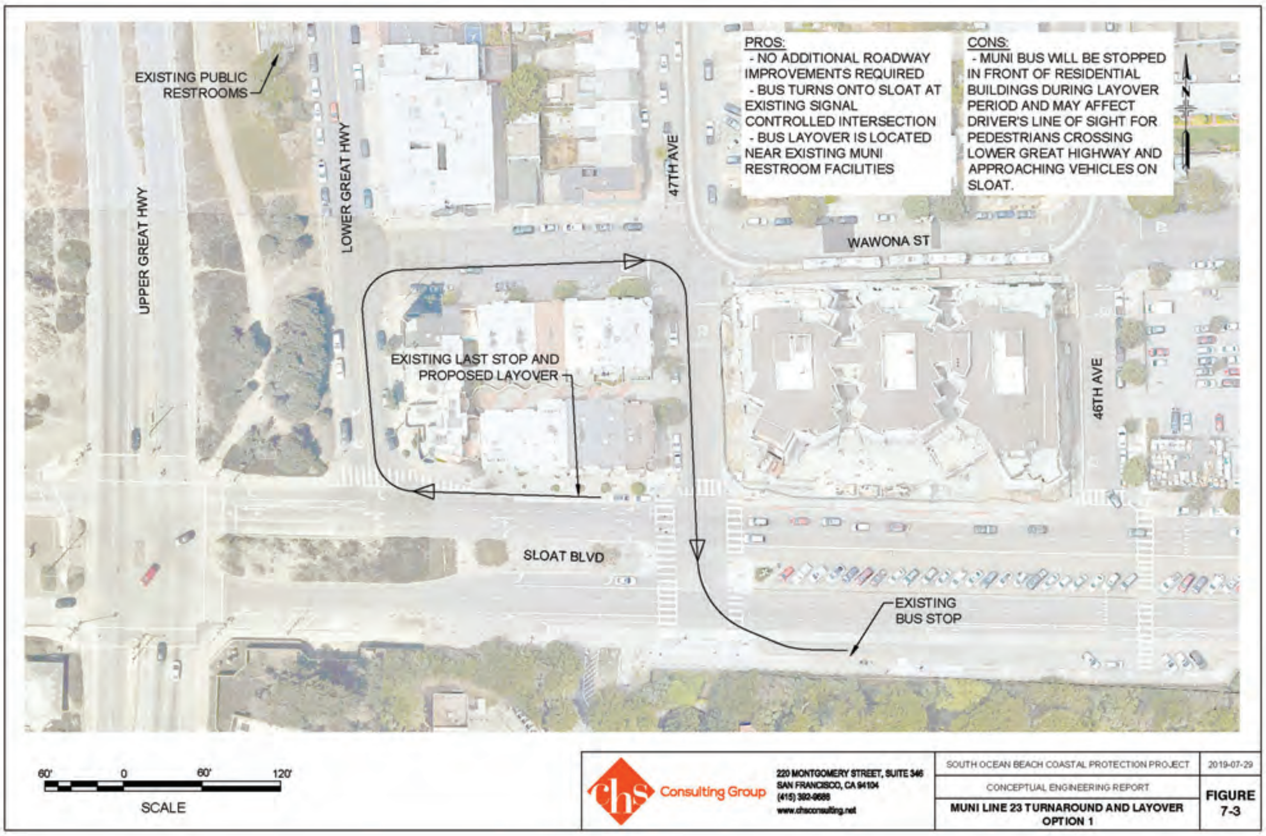

Figure 7-3: Muni Line 23 Turnaround and Layover – Option 1

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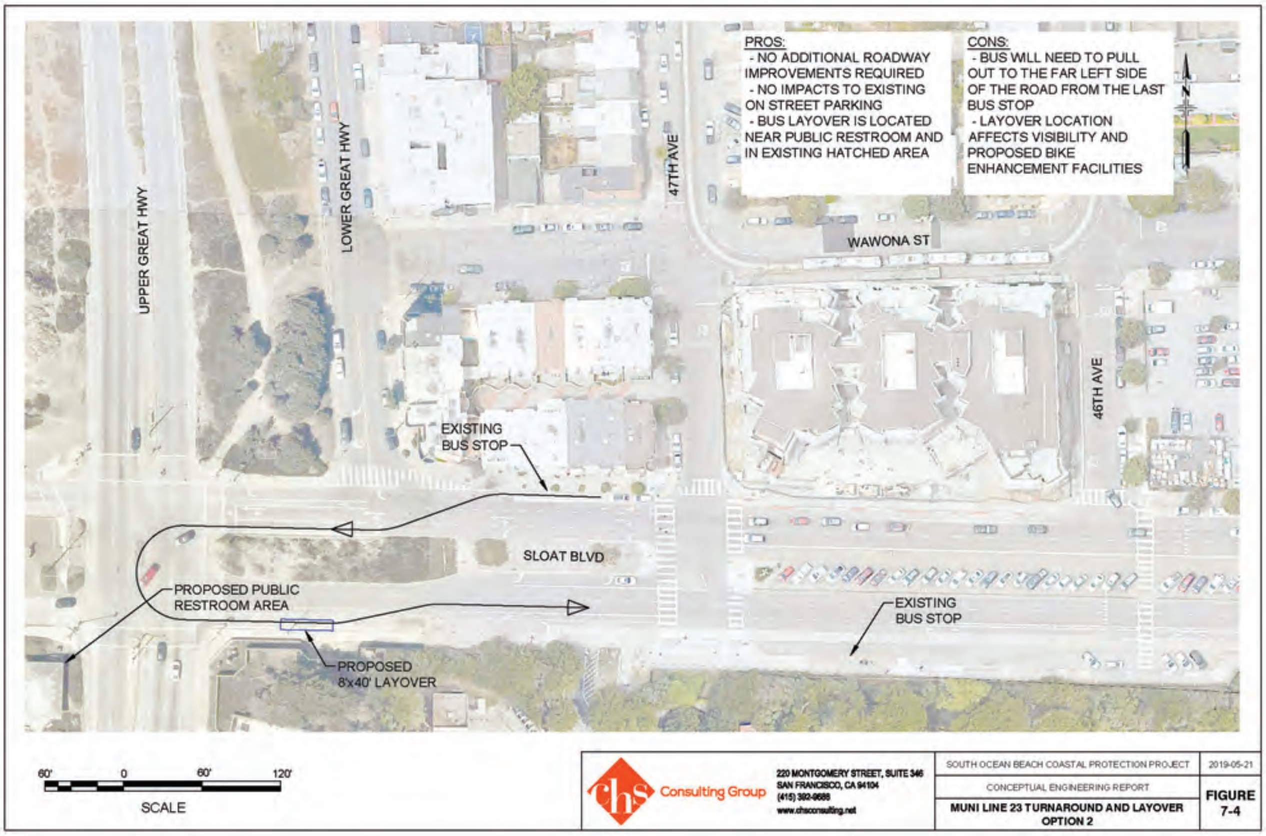

Figure 7-4: Muni Line 23 Turnaround and Layover – Option 2

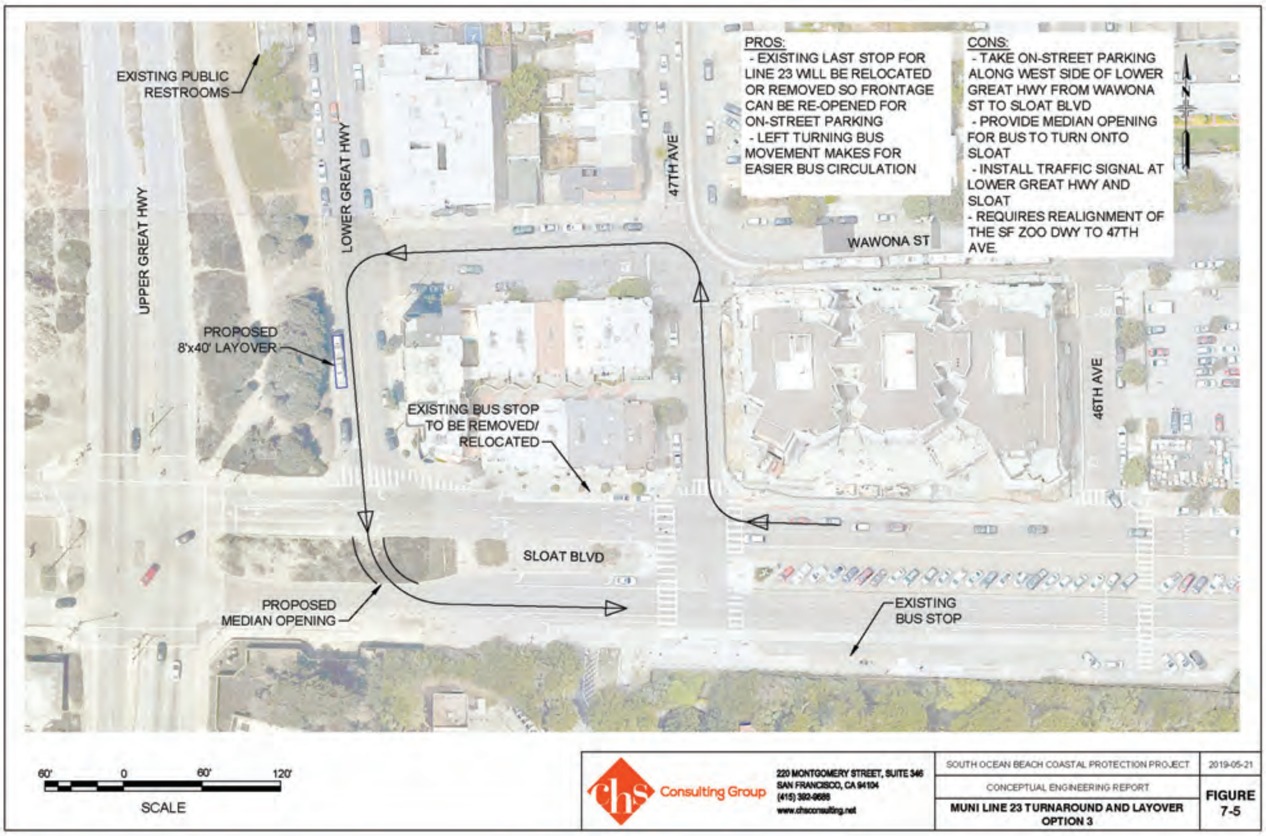

Figure 7-5: Muni Line 23 Turnaround and Layover – Option 3

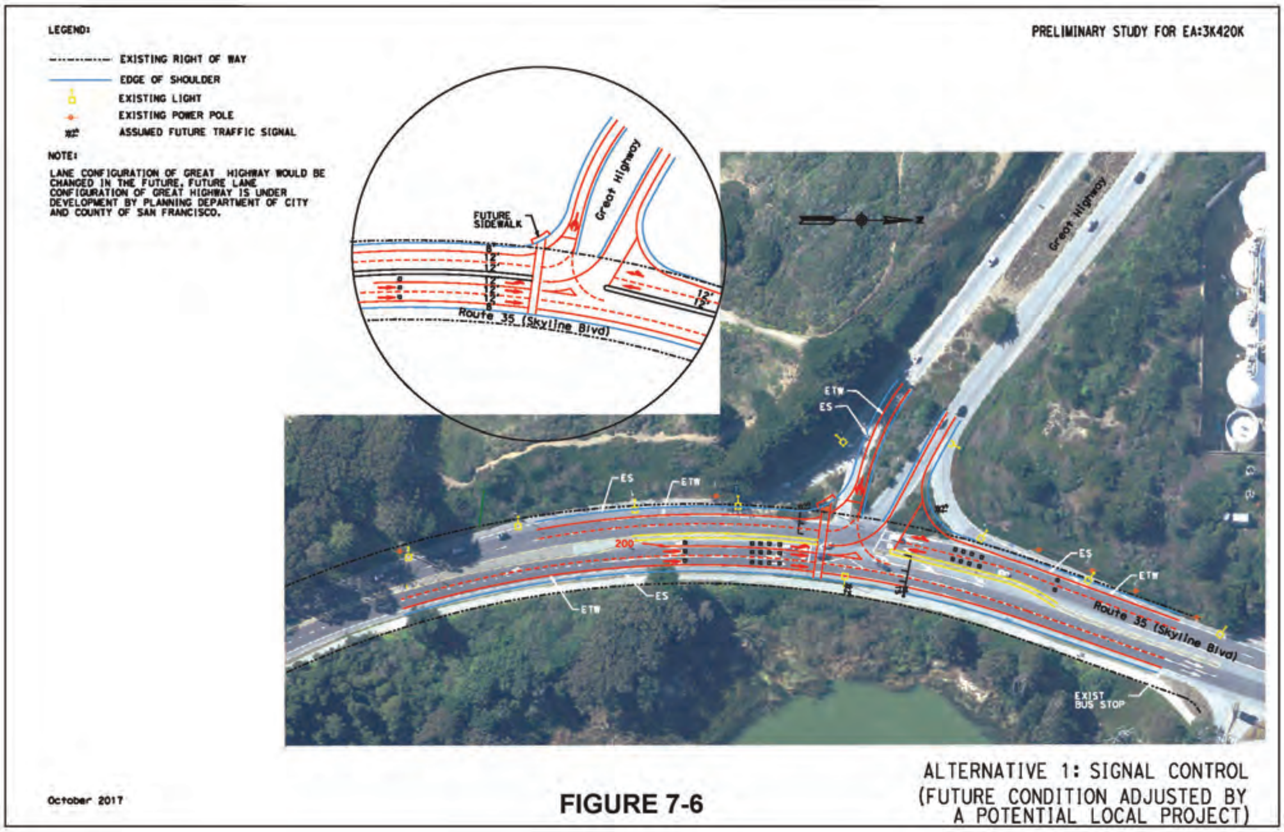

Figure 7-6: Great Highway at Skyline Boulevard Intersection (Caltrans/SFDPW Concept)

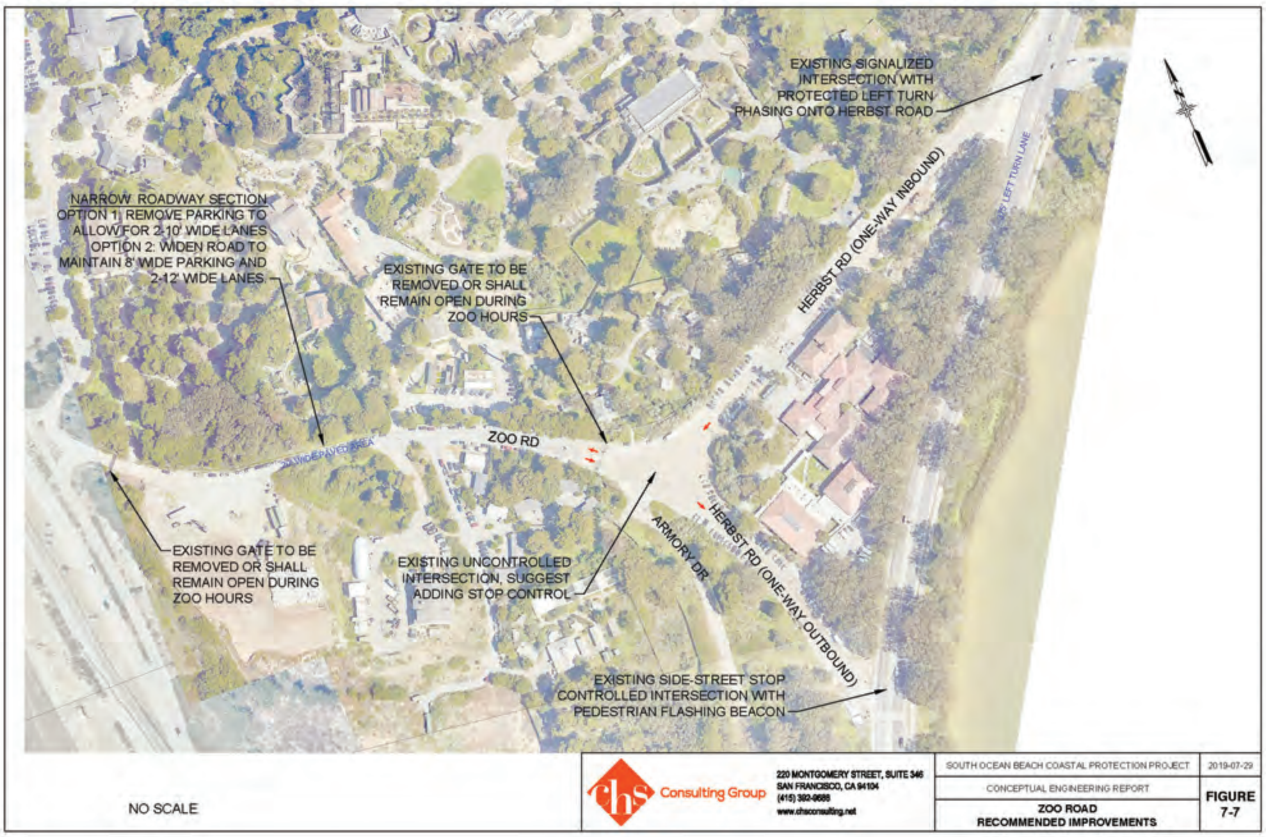

Figure 7-7: Zoo Road Access Concept

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

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

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

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

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

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

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

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

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

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

Trail Width, Striping and Vertical Clearance

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

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

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

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

Trail Surfacing, Slopes, and Drainage

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

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

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

Trail Lateral Clearance and Shoulders

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

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

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

Trail Signage and Wayfinding [Under Development]

8.1.1. Concept Design – Beach Access and Amenities

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

Multi-Use Trail & Beach Access

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

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

Access Stairs

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

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

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

Restroom

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

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

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

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

8.2. Parking

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

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

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

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

8.3. Service Road

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

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

8.3.1. Concept Design – Service Road

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

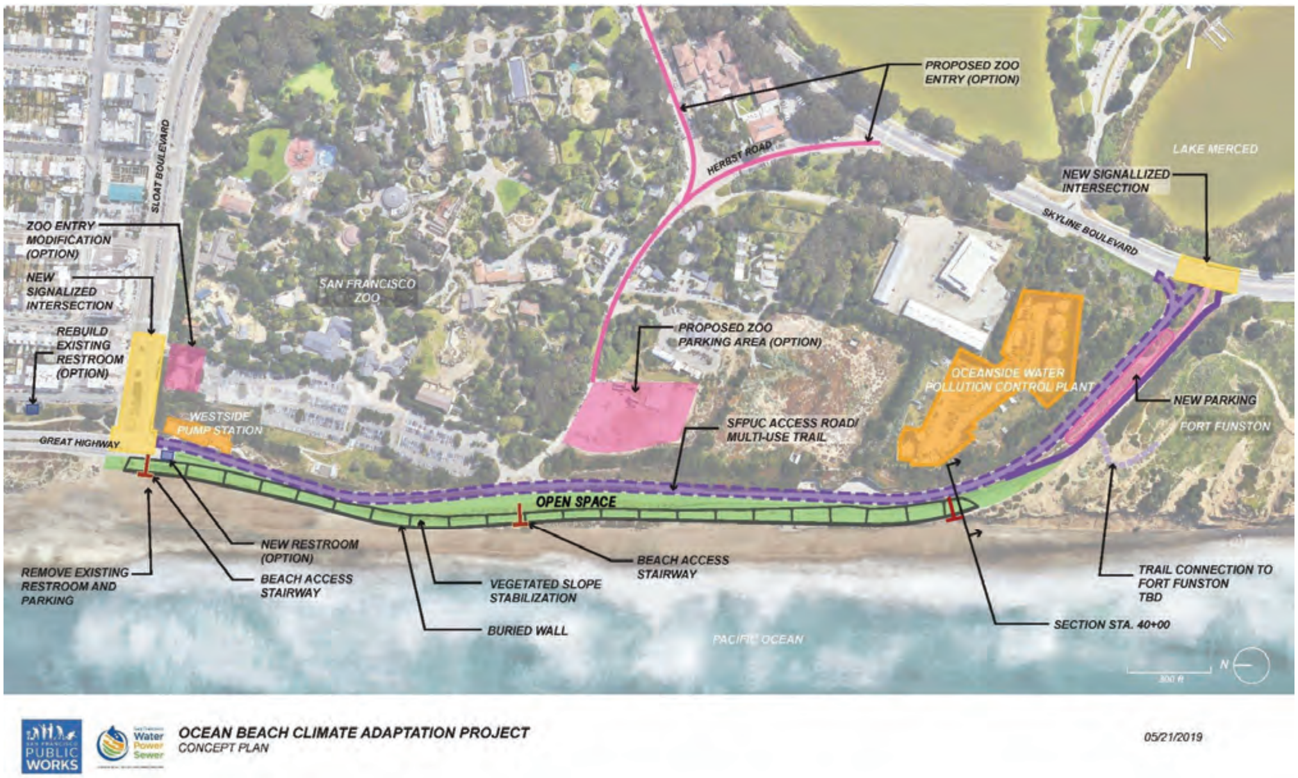

Figure 8-1: Overall Concept Plan

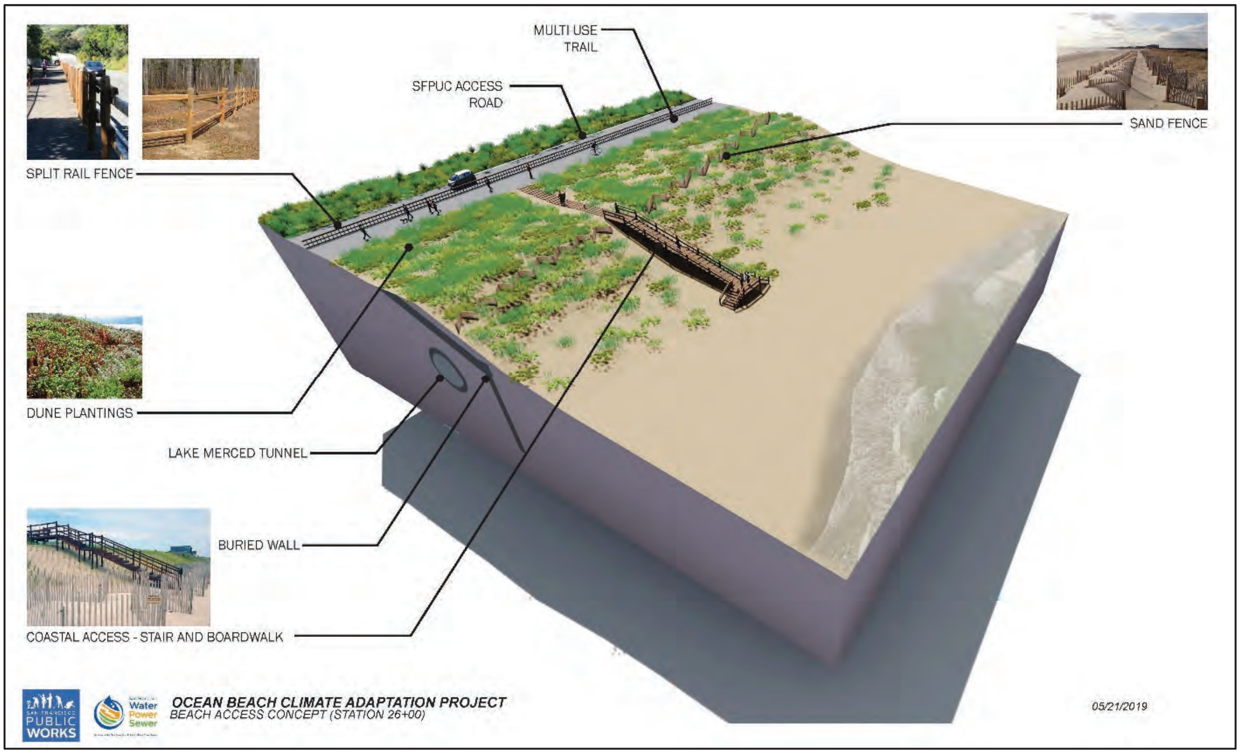

Figure 8-2: Beach Access Concept

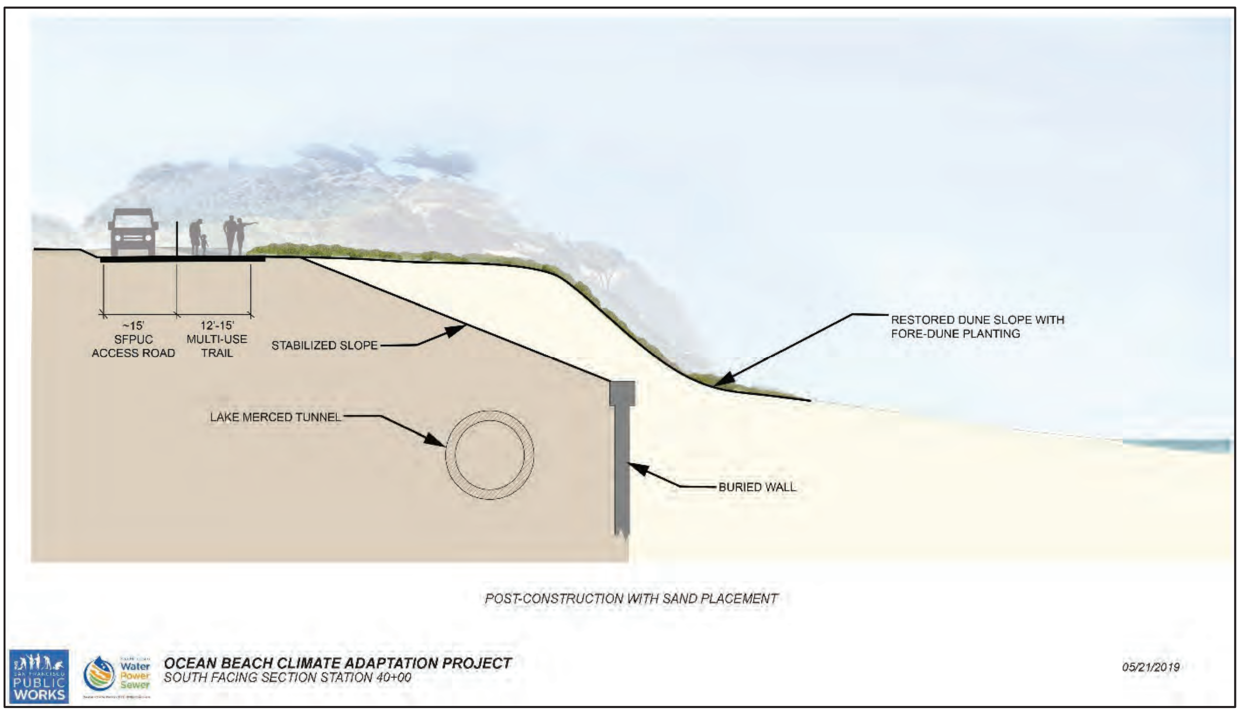

Figure 8-3: Concept Section

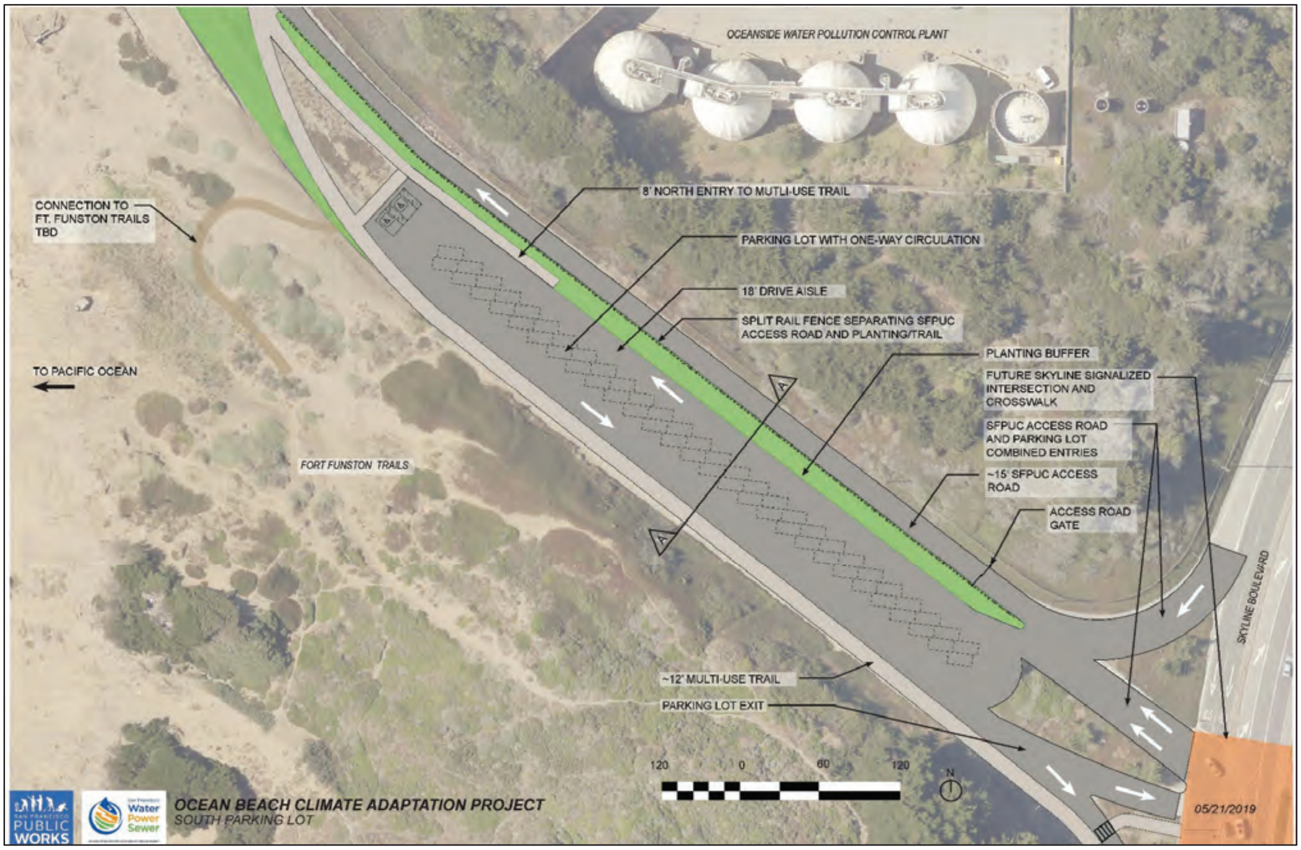

Figure 8-4: Parking Concept Layout

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9. Structural

9.1. Background

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

9.2. Wall Description

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

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

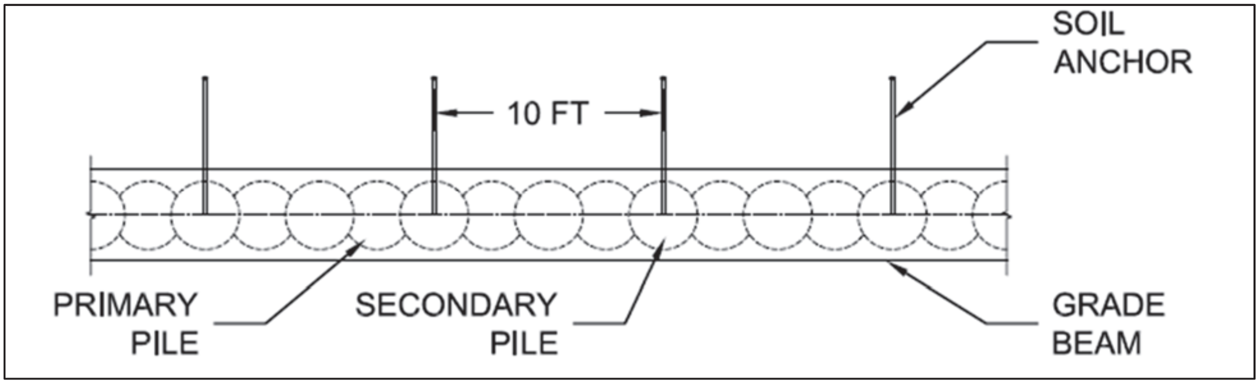


Figure 9-1: Typical Low-Profile Wall Plan.

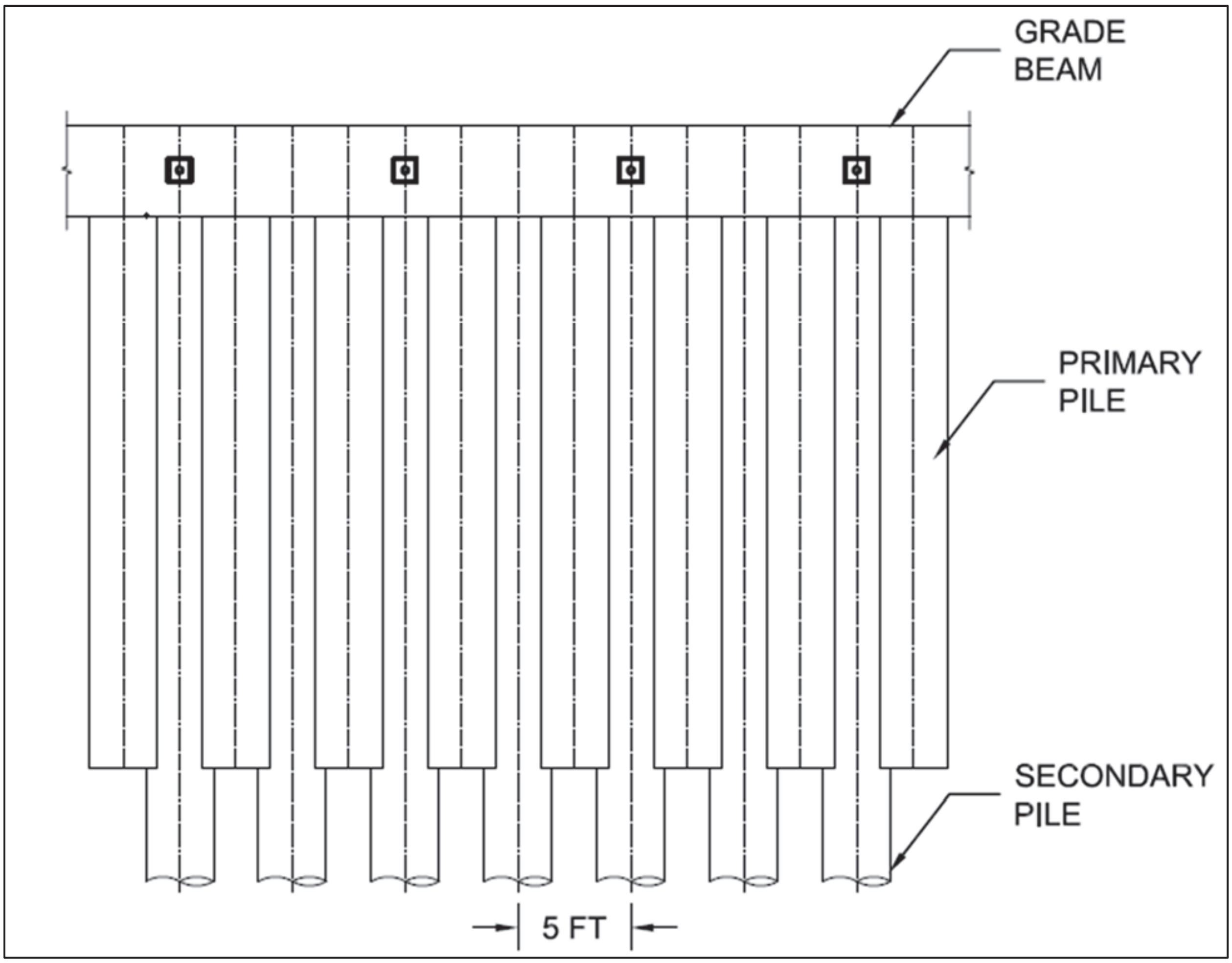


Figure 9-2: Typical Low-Profile Wall Elevation.

9.3. Wall Construction

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

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

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

9.4. Wall Reaches and Representative Sections

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

Table 9-1: Reach Descriptions.

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

9.5. Soils Properties for Structural Design

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

9.5.1. Soils Characterization

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

Table 9-2: Soil Properties for Wall Analysis.

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

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

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

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

9.5.2. Liquefaction

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

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

9.5.3. Water Table

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

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

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

9.6. Load Conditions and Design Load Conditions

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

9.6.1. Earth Pressure

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

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

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

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

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

9.6.2. Static Water Pressure

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

9.6.3. Surcharge

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

9.6.4. Seismic

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

- Non-yielding retaining (movement between 0.1%H and 0.2%H) – uniform rectangular with base of 32H where H is the height of the wall (taken from elevation -10-ft to top of wall).
- Rigid wall (movement less than 0.1%H) – straight triangular shape with base of 45H where H is height of the wall.

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

9.6.5. Liquefaction

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

9.6.6. Wave Forces

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

9.6.7. Design Loading Condition 1 – Static Condition

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

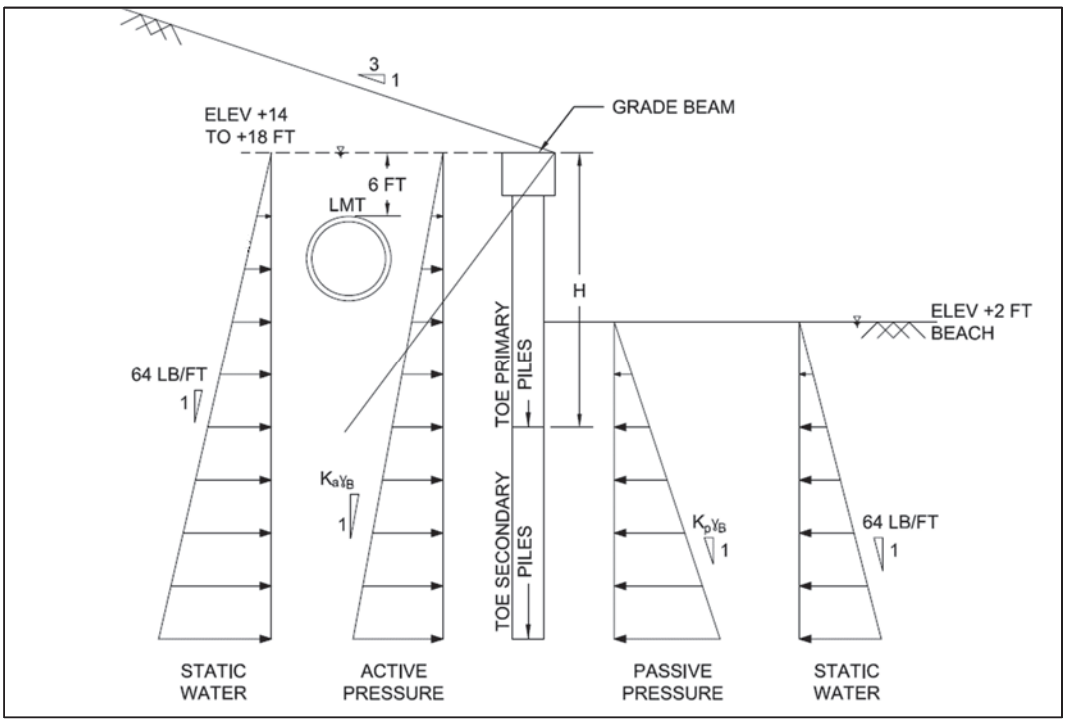


Figure 9-3: Design Loading Condition DLC-1.

9.6.8. Design Loading Condition 2 – Seismic Condition

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

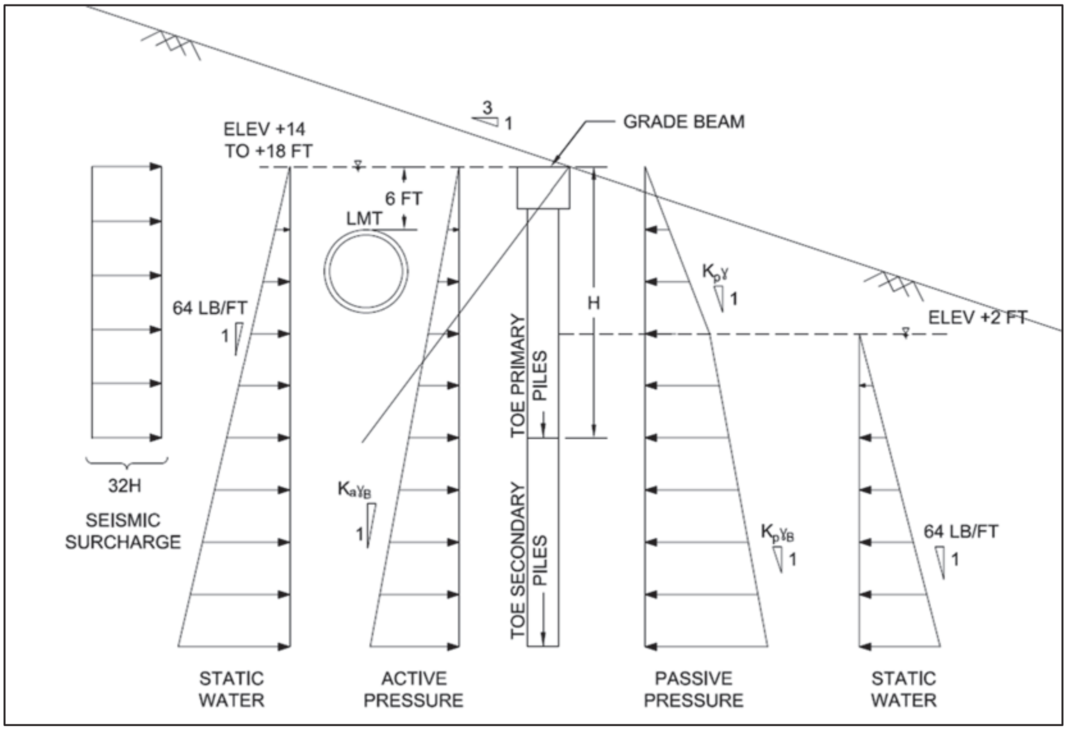


Figure 9-4: Design Loading Condition DLC-2.

9.7. Structural Analysis

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

9.7.1. SupportIT

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

9.7.2. DeepEx

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

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

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

9.7.3. SAP 2000

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

9.7.4. Analysis Summary

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

Table 9-5: Analysis Summary

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

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

9.8. Structural Design

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

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

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

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

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

Where:

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

9.9. LMT Structural Evaluation

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

- Cross sectional distortion
- Longitudinal differential displacement

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

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

9.9.1. Analysis Approach

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

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

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

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

9.9.2. Inputs to Analysis

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

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

Table 9-6: Soil layers and Groundwater elevations.

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

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

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

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

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

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

Table 9-9: Tieback parameters.

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

9.9.3. Key Assumptions

The following key assumptions were made regarding the ground behavior:

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

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

9.9.4. FLAC2D Models and Procedure

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

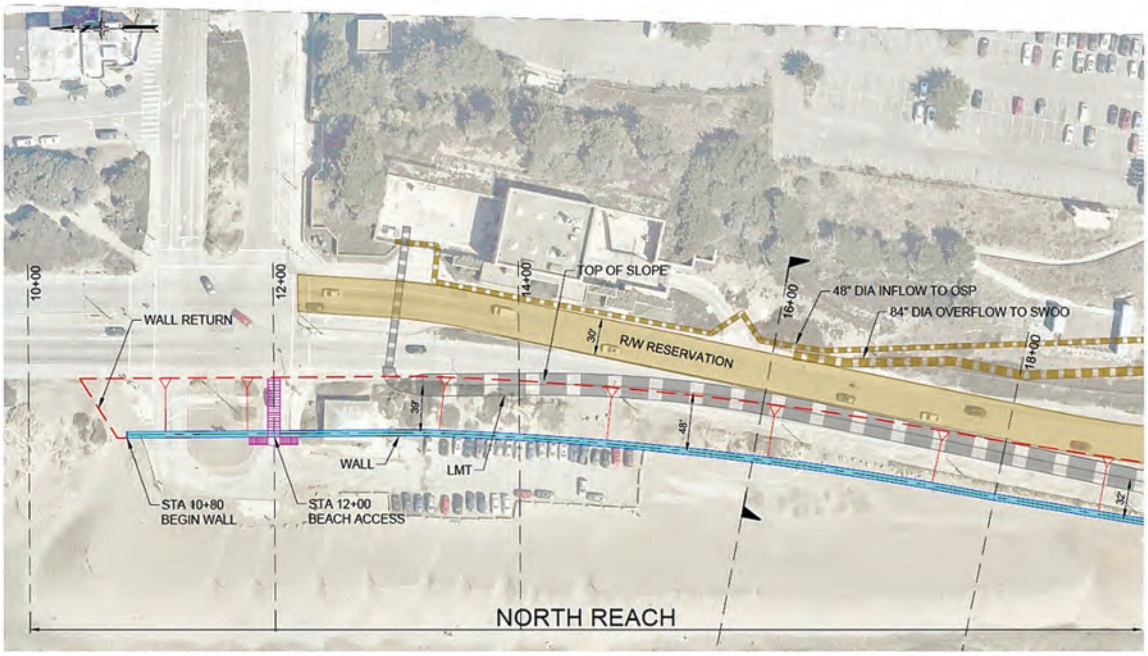
1. Gradual soil removal, as part of construction.
2. Assuming Groundwater elevation 2 feet above the AGS recommended elevation.
3. Assuming tunnel to be 50% full of effluent.

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

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

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

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



(a)

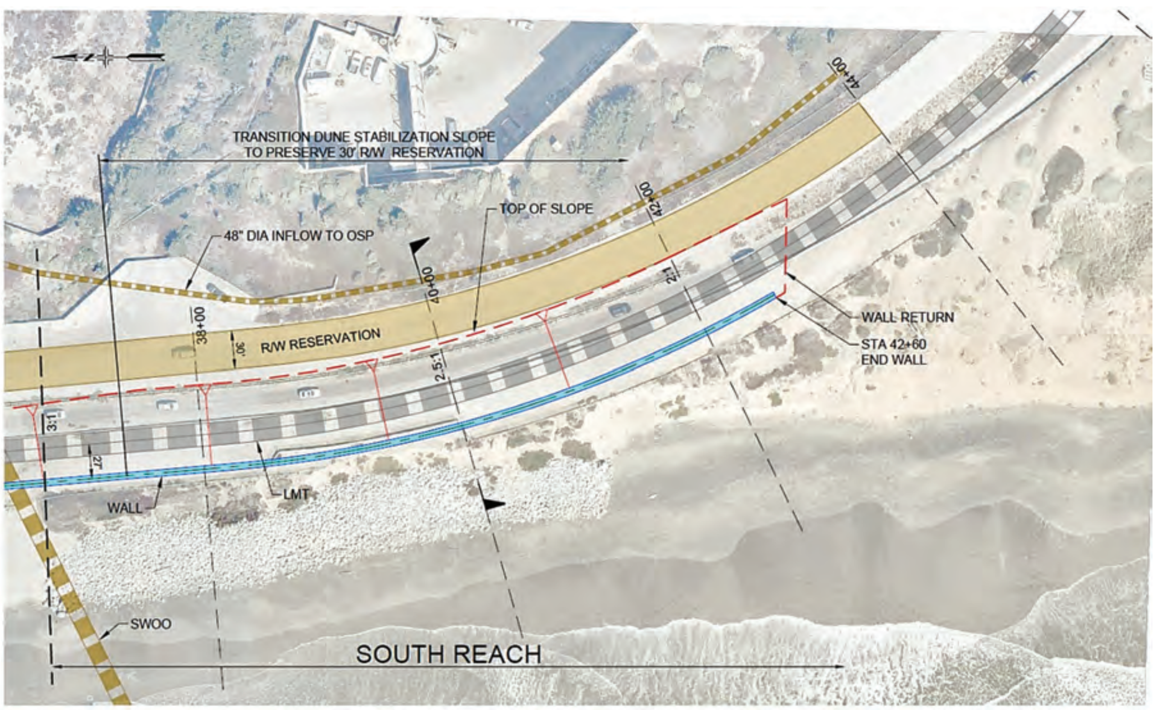
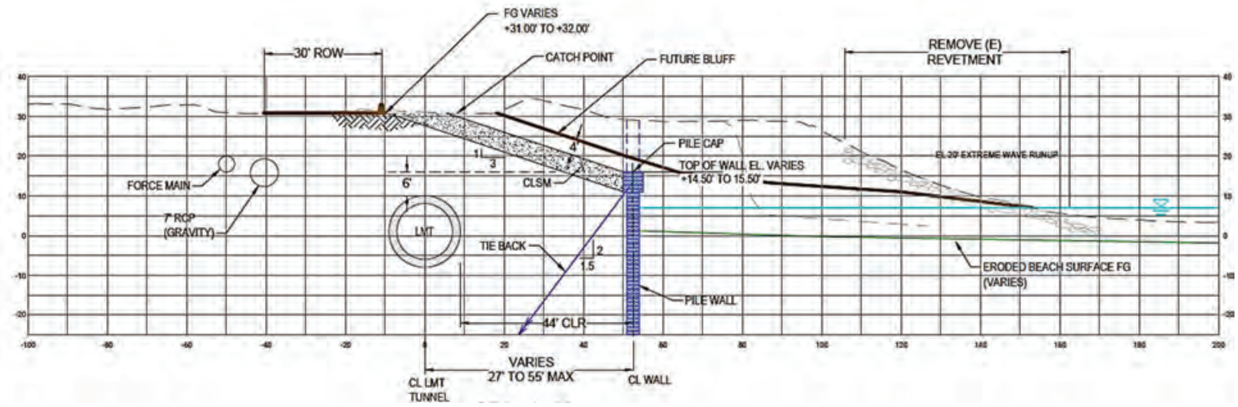
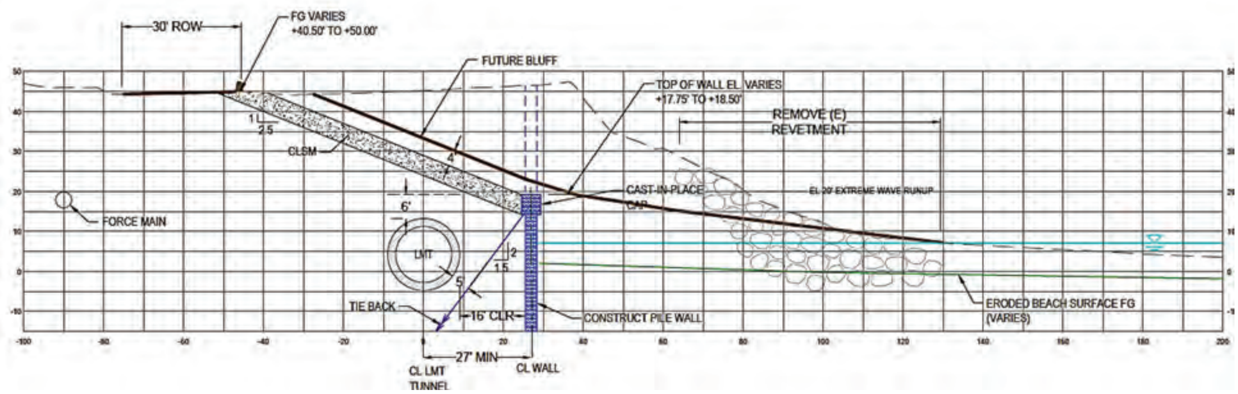


Figure 9-5: Tunnel plan view for (a) STA 16+00, (b) STA 40+00

(b)

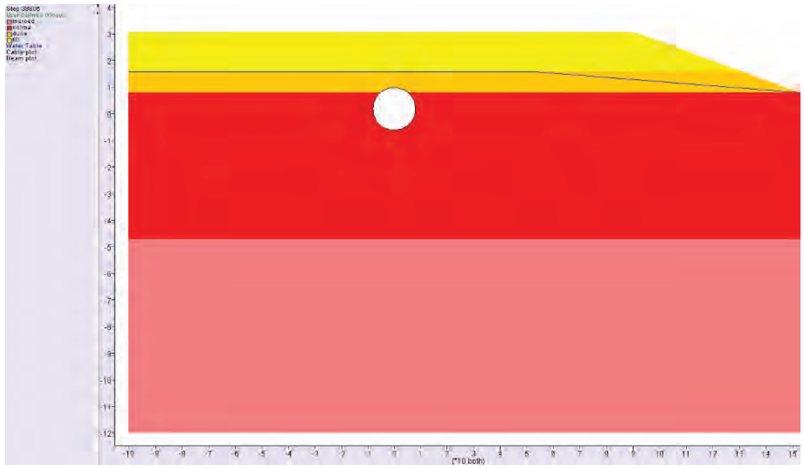


(a)

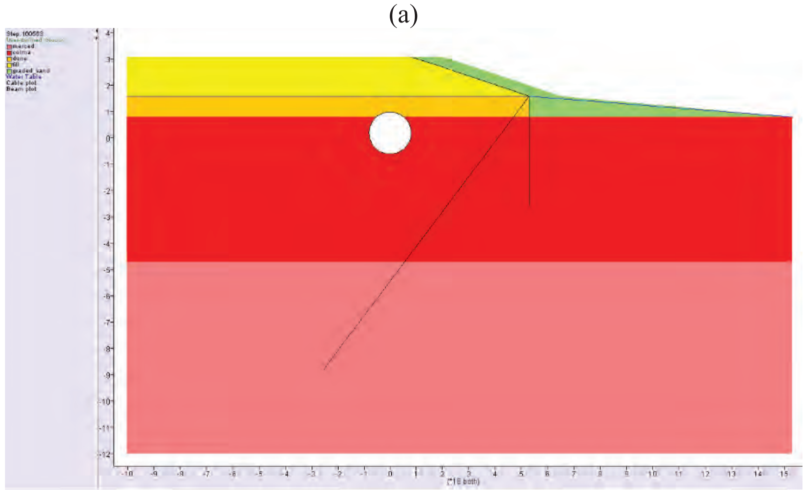


(b)

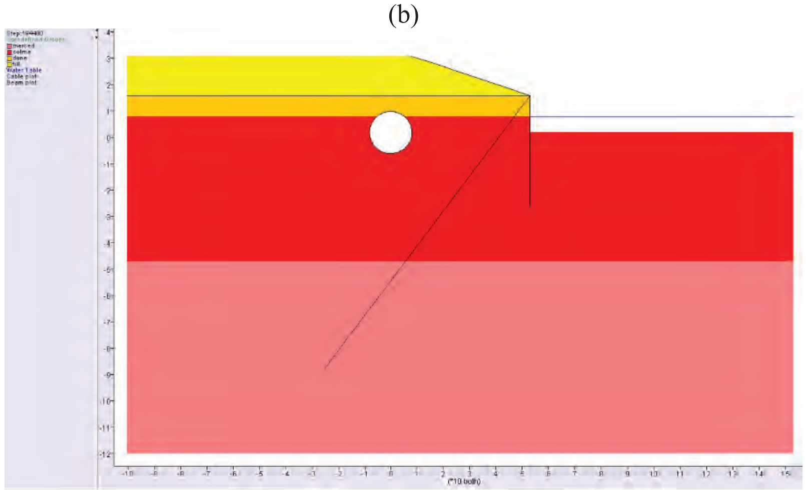
Figure 9-6: Cross sections for (a) STA 16+00, (b) STA 40+00.



(a)

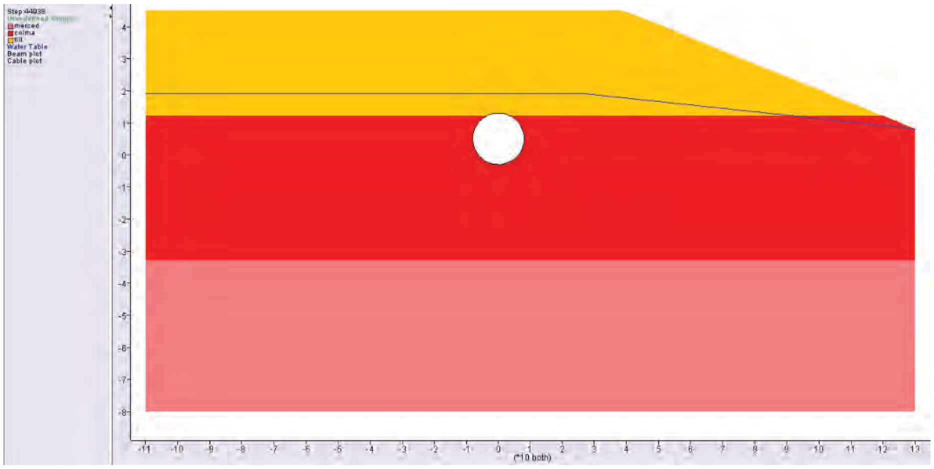


(b)

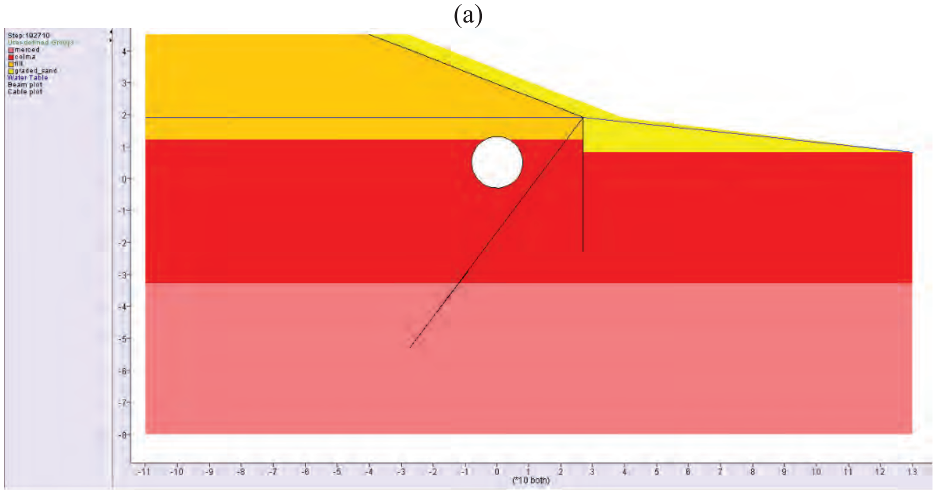


(c)

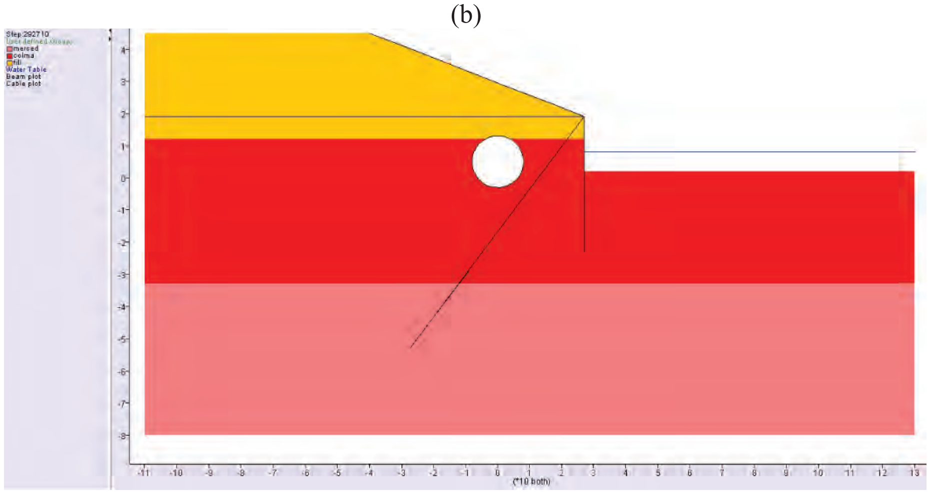
Figure 9-7: STA 16+00 model config. for (a) initial, (b) end of construction, and (c) long-term erosion conditions.



(a)



(b)



(c)

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

9.9.5. Results of Analyses

Cross Sectional Distortion

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

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

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

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

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

Longitudinal Deflection and Tunnel Lining Stresses

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

10. Constructability

10.1. Introduction

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

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

10.2. Obstructions and Constraints

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

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

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

10.3. Traffic Disturbance

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

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

10.4. Construction Activities and Sequencing

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

10.5. Construction Quantities

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

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

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

10.6. Construction Equipment

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

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

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

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

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

11. Operations and Maintenance

11.1. Introduction

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

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

11.2. Periodic Beach and Dune Nourishment

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

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

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

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

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

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

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

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

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

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

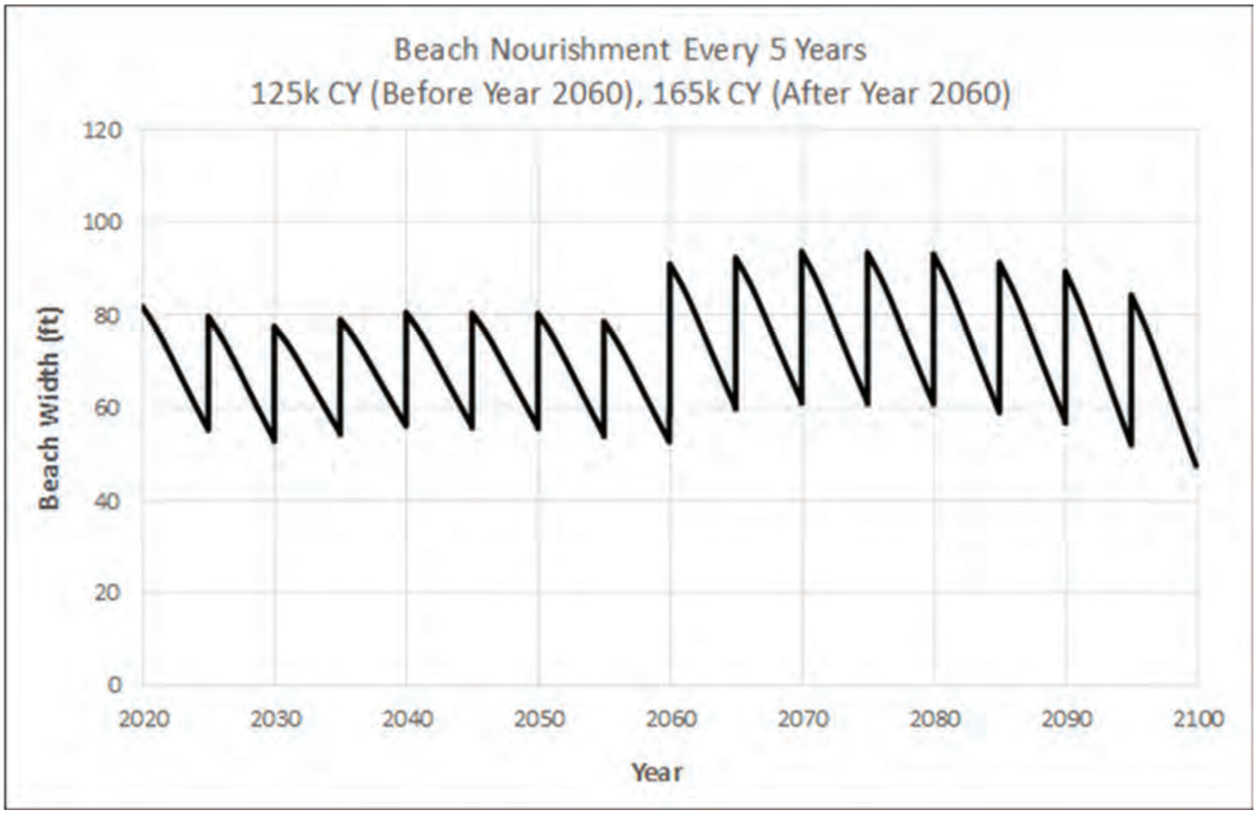


Figure 11-1: Beach Width Variation (RCP 8.5, Medium/High Risk Aversion Projection)

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

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

11.3. Public Access Features

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

11.4. Service Road and Parking Lots

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

12. Legal/Right-of-Way

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

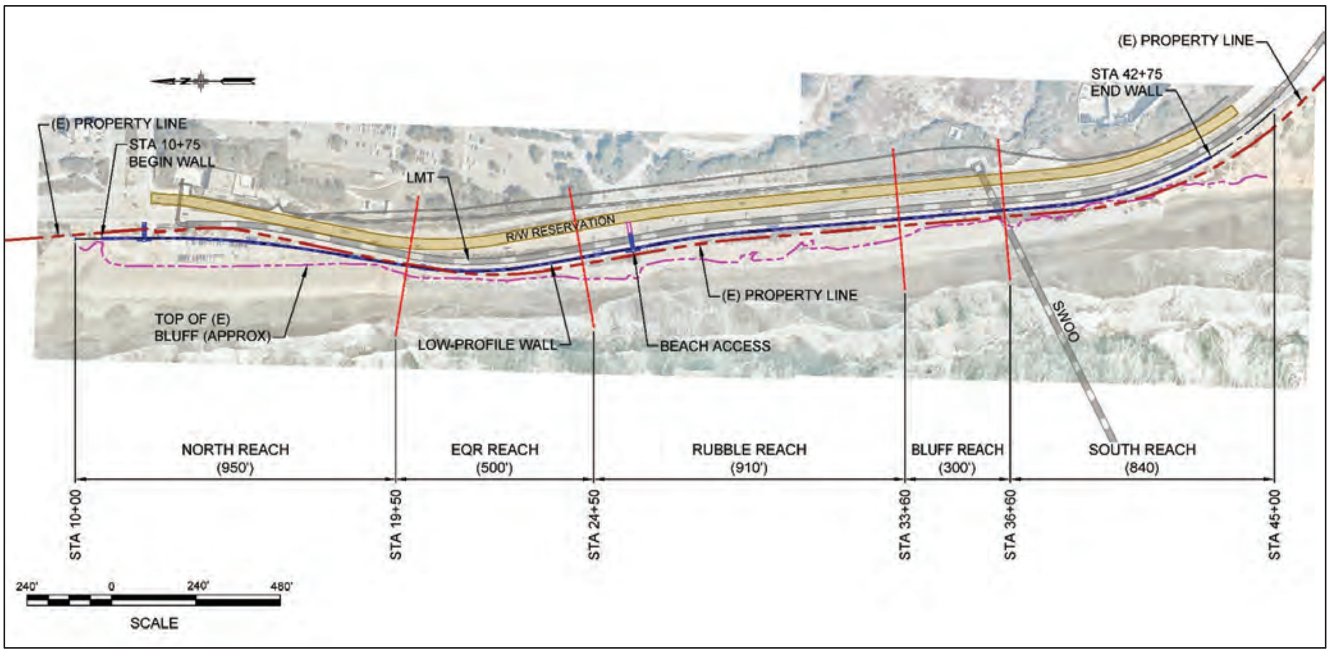


Figure 12-1: Interim Property Boundary Demarcation (GGNRA and CCSF)

13. Environmental Review

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

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

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

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

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

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

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

CEQA Documentation

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

NEPA Documentation

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

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

Environmental Permits

Permits anticipated for the project include:

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

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

14. Construction Duration and Schedule

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

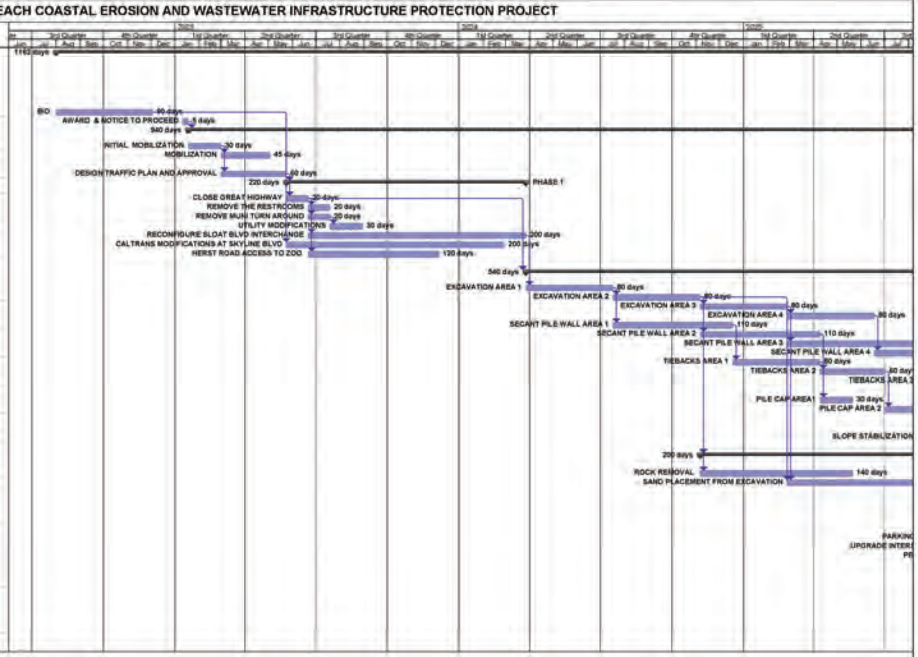
Construction could occur all year round, at least in different locations, and may only need to be suspended due to extreme weather conditions that may occur.

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

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

SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT

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



Project: PHASED CONSTRUCTION SCHEDULE A_190905 Date: Thu 9/5/19	Task	Rolled Up Task	External Tasks
	Split	Rolled Up Split	Project Summary
	Milestone	Rolled Up Milestone	External Milestone
	Summary	Rolled Up Progress	Inactive Task

Inactive Milestone	Manual Summary Rollup	Progress
Inactive Summary	Manual Summary	Deadline
Manual Task	Start-only	
Duration-only	Finish-only	

Page 1

Exhibit 7
2-21-0912
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SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT

ID	Task Name	Duration	Start	Finish	Predecessors
1	**SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT**	1162 days	Mon 8/1/22	Tue 1/12/27	
2	BID	90 days	Mon 8/1/22	Fri 12/2/22	
3	AWARD & NOTICE TO PROCEED	5 days	Wed 1/11/23	Tue 1/17/23	
4	**CONSTRUCTION**	840 days	Wed 1/18/23	Tue 8/25/26	3
5	INITIAL MOBILIZATION	30 days	Wed 1/18/23	Tue 2/28/23	
6	MOBILIZATION	45 days	Wed 3/1/23	Tue 5/2/23	5
7					
8	DESIGN TRAFFIC PLAN AND APPROVAL	60 days	Wed 3/1/23	Tue 5/23/23	5
9	**PHASE 1**	220 days	Wed 5/24/23	Tue 3/26/24	8
10	CLOSE GREAT HIGHWAY	20 days	Wed 5/24/23	Tue 6/20/23	8
11	REMOVE THE RESTROOMS	20 days	Wed 6/21/23	Tue 7/18/23	10
12	REMOVE MUNI TURN AROUND	20 days	Wed 6/21/23	Tue 7/18/23	10
13	UTILITY MODIFICATIONS	30 days	Wed 7/19/23	Tue 8/29/23	12
14	RECONFIGURE SLOAT BLVD INTERCHANGE	200 days	Wed 6/21/23	Tue 3/26/24	10
15	CALTRANS MODIFICATIONS AT SKYLINE BLVD	200 days	Wed 5/24/23	Tue 2/27/24	2
16	HERST ROAD ACCESS TO ZOO	120 days	Wed 6/21/23	Tue 12/5/23	10
17					
18	**PHASE 2**	540 days	Wed 3/27/24	Tue 4/21/26	10
19	EXCAVATION AREA 1	80 days	Wed 3/27/24	Tue 7/16/24	14
20	EXCAVATION AREA 2	80 days	Wed 7/17/24	Tue 11/5/24	19
21	EXCAVATION AREA 3	80 days	Wed 11/6/24	Tue 2/25/25	20
22	EXCAVATION AREA 4	80 days	Wed 2/26/25	Tue 6/17/25	21
23	SECANT PILE WALL AREA 1	110 days	Wed 7/17/24	Tue 12/17/24	19
24	SECANT PILE WALL AREA 2	110 days	Wed 11/6/24	Tue 4/8/25	20
25	SECANT PILE WALL AREA 3	120 days	Wed 2/26/25	Tue 8/12/25	21
26	SECANT PILE WALL AREA 4	110 days	Wed 6/18/25	Tue 11/18/25	22
27	TIEBACKS AREA 1	80 days	Wed 12/18/24	Tue 4/8/25	23
28	TIEBACKS AREA 2	60 days	Wed 4/9/25	Tue 7/1/25	24
29	TIEBACKS AREA 3	60 days	Wed 8/13/25	Tue 11/4/25	25
30	TIEBACKS AREA 4	60 days	Wed 11/19/25	Tue 2/10/26	26
31	PILE CAP AREA1	30 days	Wed 4/9/25	Tue 5/20/25	27
32	PILE CAP AREA 2	30 days	Wed 7/2/25	Tue 8/12/25	28
33	PILE CAP AREA3	30 days	Wed 11/5/25	Tue 12/16/25	29
34	PILE CAP AREA 4	30 days	Wed 2/11/26	Tue 3/24/26	30
35	SLOPE STABILIZATION	180 days	Wed 8/13/25	Tue 4/21/26	32
36					
37	**PHASE 3**	200 days	Wed 11/6/24	Tue 8/12/25	20
38	ROCK REMOVAL	140 days	Wed 11/6/24	Tue 5/20/25	20
39	SAND PLACEMENT FROM EXCAVATION	120 days	Wed 2/26/25	Tue 8/12/25	20,21
40					
41	**PHASE 4**	190 days	Wed 11/5/25	Tue 7/28/26	29
42	TRAIL CONSTRUCTION	90 days	Wed 3/25/26	Tue 7/28/26	34
43	ACCESS ROAD CONSTRUCTION	90 days	Wed 3/25/26	Tue 7/28/26	34
44	PARKING LOT AT SKYLINE	150 days	Wed 11/5/25	Tue 6/2/26	29
45	UPGRADE INTERSECTION AT SKYLINE BLVD	150 days	Wed 12/17/25	Tue 7/14/26	33
46	PEDESTRIAN BEACH ACCESS	100 days	Wed 12/17/25	Tue 5/5/26	33
47	GENERAL PROJECT CLEANUP	60 days	Wed 4/22/26	Tue 7/14/26	35,39
48	PHASE 1 LANDSCAPING	90 days	Wed 12/17/25	Tue 4/21/26	33,39
49					
50	CLOSE OUT CONTRACT	90 days	Wed 4/22/26	Tue 8/25/26	35,39
51					
52	**ALTERNATE CONTRACT**	100 days	Wed 8/26/26	Tue 1/12/27	50
53	DUNES RESTORATION	100 days	Wed 8/26/26	Tue 1/12/27	50
54	BEACH REFURBISHING	100 days	Wed 8/26/26	Tue 1/12/27	50
55	LANDSCAPING	100 days	Wed 8/26/26	Tue 1/12/27	50

Project: PHASED CONSTRUCTION SCHEDULE_A_190905
Date: Thu 9/5/19

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

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15. Cost Estimate

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

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

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

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

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

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

16. Specification List

DIVISION 00 – PROCUREMENT AND CONTRACTING REQUIREMENTS

INTRODUCTORY INFORMATION

00 01 02 ADVERTISEMENT FOR BIDS/INVITATION TO BID

00 01 03 KEY CONTACTS AND DETAILS

00 01 07 SEALS PAGE

00 01 10 TABLE OF CONTENTS

00 01 15 LIST OF DRAWING SHEETS

BIDDING REQUIREMENTS

00 21 13 INSTRUCTIONS TO BIDDERS

00 21 14 QUESTIONS ON BID DOCUMENTS

00 21 15 REQUEST FOR TRADE EXEMPTION FORM

00 21 16 RELEASE AND WAIVER

00 31 00 AVAILABLE PROJECT INFORMATION

00 40 13 BIDDING FORMS CHECKLIST

00 41 00 BID FORM

00 41 10 SCHEDULE OF BID PRICES

00 43 13 BID BOND

00 43 20 ACKNOWLEDGMENT OF RECEIPT OF ADDENDA

00 43 36 PROPOSED SUBCONTRACTORS FORM

00 43 37 PROPOSED SUBCONTRACTORS FORM FOR ALTERNATE WORK

00 45 13/PQ BIDDER'S QUALIFICATIONS

00 45 19 NON-COLLUSION AFFIDAVIT

00 45 57 CITYBUILD/FIRST SOURCE REFERRAL PROGRAM CERTIFICATION

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

00 45 58 INITIAL EMPLOYMENT PROJECTION

00 45 59 AGREEMENT TO HIRE APPRENTICES FROM PARTICIPATING REFERRALS AGENCIES

00 45 60 HIGHEST PREVAILING WAGE RATE CERTIFICATION

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

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

00 45 72 CHAPTER 12B EQUAL BENEFITS DOCUMENTATION GUIDE

00 45 82 CERTIFICATION OF BIDDER REGARDING DEBARMENT AND SUSPENSION

00 45 85 BUSINESS TAX REGISTRATION DECLARATION

00 45 86 BUSINESS REGISTRATION CERTIFICATE REQUIREMENT

00 45 87 CERTIFICATE OF BIDDER REGARDING APPRENTICESHIP TRAINING PROGRAM

00 45 88 CERTIFICATE OF SUBCONTRACTOR REGARDING APPRENTICESHIP TRAINING PROGRAM

00 49 01 CMD ATTACHMENT 1 – CONTRACT MONITORING DIVISION REQUIREMENTS FOR CONSTRUCTION CONTRACTS

00 49 04A CMD ATTACHMENT 4A – CONTRACT MONITORING DIVISION REQUIREMENTS FOR SFPUC REGIONAL MICRO-LBE SET-ASIDE

CONSTRUCTION CONTRACTS

00 49 06 CMD ATTACHMENT 6 – REQUIREMENTS FOR SFPUC REGIONAL CONSTRUCTION CONTRACTS

CMD2B FORM 2B: "GOOD FAITH OUTREACH" REQUIREMENTS FORM CMD3 FORM 3: CMD COMPLIANCE AFFIDAVIT

CMD6 FORM 6: CMD LBE SUBCONTRACTOR PARTICIPATION AFFIDAVIT

CMD6A FORM 6A: CMD LBE TRUCKING FORM

00 49 11 EQUIPMENT SUPPLIERS LIST

00 49 12 EXPERIENCE STATEMENT

00 49 14 CERTIFICATION OF SUBCONTRACTOR, LOWER-TIER SUBCONTRACTOR OR SUPPLIER REGARDING DEBARMENT AND SUSPENSION

00 49 16 DEPARTMENT OF INDUSTRIAL RELATIONS REQUIREMENTS

00 49 18 REQUEST FOR PRODUCT SUBSTITUTION

CONTRACTING REQUIREMENTS

00 52 00 AGREEMENT FORM

00 52 10 PROJECT LABOR AGREEMENT – LETTER OF ASSENT

00 61 13 PERFORMANCE AND PAYMENT BOND FORM

00 62 20 SUBCONTRACTING GOALS AND NON-DISCRIMINATION REQUIREMENTS

00 63 30 ESCROW AGREEMENT FOR SECURITY DEPOSITS IN LIEU OF RETENTION

00 64 00 CMD CONTRACT FORMS

CMD7 FORM 7: CMD PROGRESS PAYMENT FORM

CMD8 FORM 8: CMD EXIT REPORT AND AFFIDAVIT

CMD9 FORM 9: CMD PAYMENT AFFIDAVIT

CMD10 FORM 10: CMD CONTRACT MODIFICATION FORM

00 67 00 ESCROW BID DOCUMENTS

00 67 00/A ESCROW BID DOCUMENTS DECLARATION

00 72 00 GENERAL CONDITIONS

00 73 00 SUPPLEMENTARY CONDITIONS

00 73 00/APA APPENDIX A: PERMITS AND AGREEMENTS TO BE OBTAINED BY THE CITY

DIVISION 01 – GENERAL REQUIREMENTS

DIVISION 02 – EXISTING CONDITIONS

DIVISION 03 – CONCRETE

DIVISION 05 – METALS

DIVISION 06 THRU 30 (NOT USED)

DIVISION 31 – EARTHWORK

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

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

DIVISION 33 THRU 46 (NOT USED)

ACCESS ROAD AND COASTAL TRAIL

(Under Development)

LANDSCAPING AND SIGNAGE

(Under Development)

17. Drawing List

GENERAL

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

CIVIL

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

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

UTILITIES

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

TRAFFIC AND INTERSECTIONS

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

STRUCTURAL

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

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

ACCESS ROAD AND COASTAL TRAIL

(Under Development)

LANDSCAPING AND SIGNAGE

(Under Development)

REFERENCE DRAWINGS

TO BE DETERMINED

18. References

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FINAL

GEOTECHNICAL INTERPRETIVE REPORT (GIR)

SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA

AGS Job No. AGS-18-003

Prepared for:

SAN FRANCISCO PUBLIC UTILITIES COMMISSION

Prepared by:

AGS

JULY 2021

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1.0 INTRODUCTION

1.1 GENERAL

This Geotechnical Interpretive Report (GIR) presents the results of the geotechnical study conducted by AGS, Inc. (AGS) for the proposed South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection Project in San Francisco, California. The project alignment is located within the Great Highway alignment, between Sloat and Skyline Boulevards as shown on Plate 1 – Site Vicinity Map.

The purpose of this GIR is to provide geotechnical recommendations for use in design of the proposed project. AGS has reviewed existing geotechnical data available in the vicinity of the site and performed a field exploration and laboratory testing program. The findings from the existing data review and the field exploration and laboratory testing program were summarized in the Geotechnical Data Report (GDR) by AGS (2020) for the project.

The conclusions and recommendations presented in this report are based on the available geologic information for the area and subsurface conditions encountered previously by AGS and others and those encountered during our field exploration for this project. The conclusions and recommendations presented in this report should not be extrapolated to other areas or used for other facilities without prior review by AGS.

1.2 PROJECT DESCRIPTION

The proposed South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection Project is about 3,180 linear feet (from Station 10+80 to Station 42+60) and primarily includes the following two elements:

1. Structural protection of the Lake Merced Transport and Storage Tunnel (LMT); and
2. Strategic management of the coastal conditions.

Our geotechnical study for this project is focused on the first element (structural protection of the LMT). The scope of our geotechnical study on the second element (strategic management of the coastal conditions) is to characterize the ground conditions and soil properties at the beach and the bluff, which are addressed in the GDR.

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

The secant pile wall will consist of overlapping unreinforced and reinforced drilled, cast-in-place concrete piles (called "primary" and "secondary" piles, respectively) installed at approximately 5-foot center-to-center spacing between secondary piles. Both the primary unreinforced and secondary reinforced piles will be approximately 3 feet in diameter. The primary unreinforced piles will be drilled first and filled with concrete, followed by the secondary reinforced piles drilled between and partially cutting into the primary unreinforced piles. The toe of the primary unreinforced piles will be set at approximately Elevation -10 feet[1]. The secondary reinforced piles will be extended to greater depths as determined by structural analysis. An approximately 5-foot wide by 4-foot-thick continuous grade beam will be constructed for the secant pile wall with the top set at an elevation approximately 6 feet above the crown of the LMT. It is anticipated that the tiebacks will be installed at a spacing of 10 to 20 feet along the grade beam and at approximately 45 degrees to provide lateral restraint to the top of the wall. The distance between the centerline of the LMT tunnel and the secant pile wall is mostly about 27 feet and up to about 48 feet at the northern end near Sloat Boulevard.

Initially, the secant pile wall will be buried. However, over time, as beach recession occurs, the secant pile wall will be exposed (with the ground surface in front of the wall designed for a beach level of Elevation +2 feet). To provide resistance to erosion induced by wave run-up over the top of the wall, the upper 4 feet of soil cover for the ultimate backslope (above the LMT) will be either improved by in-situ soil-cement mixing or replaced with controlled low-strength material (CLSM) with a slope of 3H:1V. Ultimately, the landward side above the top of the secant pile wall will become a 3H:1V backslope. We also understand that 3 to 4 feet of sand deposition may occur on top of the protective cover.

Based on the 2012 Ocean Beach Master Plan, the Great Highway will be rerouted inland behind the San Francisco Zoo via Sloat and Skyline Boulevards. As currently proposed, a 30-foot wide right-of-way (R/W) reservation that consists of a 15-foot wide maintenance access road and a 15-foot wide multi-use trail will be provided behind the backslope.

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

Based on subsurface and coastal conditions, the proposed wall alignment is divided into five reaches as shown in Table 1. The representative stations serve as the worst-case scenario from the geotechnical and coastal conditions and were used for design of each reach.

TABLE 1
REACH DESCRIPTIONS

Name	Start Station	End Station	LMT Setback from Bluff (feet)	Depth of LMT Crown (Min/Max) (feet)	LMT Crown Elevation (Beginning / End) (feet)	Representative Station (feet)
North Reach	10+80	19+55	40	20/20	9.47 / 10.31	13+55
EQR Reach	19+55	24+55	38	20/20	10.31 / 11.15	22+30
Rubble Reach	24+55	33+70	80	20/22	11.15 / 11.88	27+40
Bluff Reach	33+70	36+65	35	22/30	11.88 / 12.55	35+05
South Reach	36+65	42+60	28	30/50	12.55 / 13.33	41+90

1.3 EXISTING DATA REVIEW

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

- Geotechnical Report, Westside Pump Station Reliability Improvements, San Francisco, California, by GTC, Inc., 2016;
- Draft Report Geotechnical Study, Slope Stability Hazard Evaluation, Great Highway Stabilization, San Francisco, California, AGS, Inc., 2010;
- Preliminary Engineering Study, Lake Merced Tunnel, The Great Highway, San Francisco, California, Treadwell & Rollo, 2002;
- Lake Merced Transport Tunnel Geotechnical Design Summary Report, Parsons Brinckerhoff Quade & Douglas, Inc., 1990;
- Geotechnical Data Report, Lake Merced Transport, San Francisco, California, AGS, Inc., 1989;

- Preliminary Geotechnical Investigation, Lake Merced Transport Project, San Francisco, California, Harding-Lawson Associates, 1981;
- Geotechnical Engineering Evaluation, Southwest Ocean Outfall Project, Woodward-Clyde Consultants, 1977;
- Geologic Exploration Studies, Southwest Ocean Outfall Project, Woodward-Clyde Consultants, 1977;
- Review and Evaluation of Existing Data, Southwest Ocean Outfall Project, Woodward-Clyde Consultants, 1977;
- Preliminary Report, Offshore Geophysical Survey, Southwest Ocean Outfall Project, Woodward-Clyde Consultants, 1977;
- Onshore Seismic Refraction Survey, Southwest Ocean Outfall Project, Woodward-Clyde Consultants, 1977; and
- West Side Transport Soil Investigation Phase I, Harding-Lawson Associates, 1976.

Relevant data from the above documents (including previous boring logs and locations) are provided in the GDR.

1.4 FIELD EXPLORATION PROGRAM

Our field exploration program for this project was performed between 2019 and 2021, and consisted of:

- Seven geotechnical soil borings: B-1 through B-5, B-6A and B-6B;
- Twenty (20) cone penetration tests: (CPT-1, CPT-2, CPT-3, SCPT-3, CPT-4 to CPT-19;
- Three monitoring wells (MW-1, MW-4 and MW-5 installed adjacent to B-1, B-4 and B-5, respectively);
- Twelve (12) vacuum potholes (PH-1A, PH-1B, PH-2A, PH-3A, PH-3B, PH-4A, PH-4B, PH-4C, PH-5A, PH-5B, PH-6A, PH-6B);
- Three (3) test pits (TP-1, TP-2, TP-3);
- Geophysical survey subsurface profiles (ML-1A, ML-1B, and ML-2 through ML-4); and

- Twelve (12) environmental borings (EB-1 through EB-6 and ET-1 through ET-6) and twenty (20) shallow borings for lead characterization. These borings can be seen on Plate 2 of the AGS report (AGS, 2021a) Environmental Report.

The findings from our field exploration program have been evaluated to develop geotechnical recommendations for this project. Full details of our field exploration program are provided in the GDR (AGS, 2020).

1.5 GEOTECHNICAL LABORATORY TESTING PROGRAM

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

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

The results of our geotechnical laboratory testing program have been evaluated to develop geotechnical recommendations for this project. Full details of our laboratory testing program are provided in the GDR (AGS, 2020).

1.6 ENVIRONMENTAL LABORATORY TESTING PROGRAM

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

- Total Petroleum Hydrocarbons – gasoline diesel and motor oil by EPA Method 8015B;
- California Title 22 Metals by EPA Methods 6010B and 7471A;
- Hexavalent Chromium by EPA Method 7196A;

- Volatile Organic Compounds (VOCs) by EPA Method 8260B;
- Semi-volatile Organic Compounds (SVOCs) by EPA Method 8270C; and
- Organochlorine Pesticides (OCPs) by EPA Method 8081A.

Twelve (12) samples were collected from the environmental borings ET-1 through ET-6 in January 2020. Two composite samples were collected from each boring, formed from soil collected between 1 and 5 feet below ground surface, and between 5 and 25 feet below ground surface. The samples were sent to Enthalpy Analytical in Berkeley, CA for the following tests:

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

Samples were also provided to Asbestos TEM Laboratories, Inc. of Berkeley, CA and analyzed for:

- Asbestos point count by California Air Resources Board Method 435

Upon receipt of the initial analytical results, selected samples were reanalyzed by Enthalpy for the following:

- Leachable Chromium by Soluble Threshold Limit Concentration (STLC) Waste Extraction Test (WET) Method 6010B

Twenty (20) samples were collected from the environmental borings Lead-1 through Lead-20 in April 2021. One composite sample was collected from each boring, formed from soil collected at 1 foot and 4 feet below ground surface. The samples were sent to Enthalpy Analytical in Berkeley, CA for the following test:

- Lead by EPA Method 6010B

The results of our environmental laboratory testing program are provided in the Draft Supplemental Environmental Investigation Report (AGS, 2021a).

1.7 CODES AND STANDARDS

The codes and standards applicable to our geotechnical study for this project include the following:

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

2.0 FINDINGS

2.1 SITE DESCRIPTION AND TOPOGRAPHY

The project alignment is located along the Great Highway, starting at Sloat Boulevard and continuing towards Skyline Boulevard to the south. The Westside Pump Station and the beach access parking lot are located at the northern end of the alignment, and the San Francisco Zoo is located east of the alignment. The southern end of the project alignment is approximately 1,000 feet south of the Oceanside Water Pollution Control Treatment Plant (OSP), which is located on the eastern side of the northbound lanes of the Great Highway.

Topographically, the site ground surface elevation is approximately +30 feet in the northern two-thirds of the site, from Sloat Boulevard at about Station 10+80 to about Station 32+00. From about Station 32+00 to about Station 42+60, the ground surface elevation gradually increases to approximately +60 feet.

Seaside bluffs in various stages of erosion, and some riprap stabilized seaside slopes, are located between 30 feet to 70 feet seaward of the western edge of the Great Highway at the time of this study. The bluffs range in height from 20 to 25 feet at the northern end of the site, up to greater than 50 feet in height at the southern end of the site. Based on our review of Google Earth and the topographic survey provided to us by the SFPUC (2015), these bluffs are sloping at approximately 3.5H:1V in the northern end of the site to 1.75H:1V in the southern end of the site.

A total of approximately 1,600 lineal feet of riprap improvements were installed along three reaches of the alignment in 2010 after the El Niño storm events of 2009-2010, which caused continued erosion and collapse of portions of the Great Highway. The supporting bluffs slipped out in some areas and the southbound lane was undermined and the pavement collapsed. At several locations along the alignment, the pavement of the former alignment of the Great Highway and its associated beach access parking lots, now decommissioned, are remaining on site, and overhanging on over-steepened slopes. A concrete k-rail barrier separates the abandoned southbound roadway from the current southbound lane.

2.2 REGIONAL AND SITE GEOLOGY

The geologic conditions of the project alignment are presented on Plates 3A and 3B – Regional and Local Geology Maps.

2.2.1 Regional Geology

The project study area is located along the coastal bluffs on the southwest side of San Francisco and is bounded to the west by the Pacific Ocean. The San Francisco Peninsula constrains the western side of the San Francisco Bay, a northwest-trending structural depression called a “bay block” that was submerged by rising sea level during the Holocene. This bay block is within the Coast Ranges geomorphic province, a region characterized by generally northwest-trending mountain ridges, valleys, and faults. The bay block is bounded by the San Andreas Fault to the west and by the Hayward fault to the east. The San Andreas fault crosses the coastline approximately three kilometers southwest of the project alignment. The San Francisco Peninsula is underlain by bedrock of the Franciscan Complex. In the project vicinity, the Franciscan bedrock surface is estimated to occur deeper than 300 feet (Schlocker, J., and Bonilla, M.G., 1972), and, based on Boring B-1 from Woodward-Clyde 1977, may be at a depth of approximately 400 feet.

Past episodes of tectonism have folded and faulted the rock of the Coast Ranges, creating northwest-trending ridges and valleys characteristic of this area. The project alignment lies on the northern end of a topographic depression, the Merced Basin: an ancient sedimentary basin bounded by the San Andreas Fault to the southwest and the Serra Fault to the northeast. The Serra is a northwest striking, southwest dipping thrust fault that is currently thought to be inactive, but may have been active during the Holocene (Kennedy, 2002). The Merced Basin is thought to be an extensional pull-apart basin, which became filled with sediments of Franciscan origin as the basin subsided and the rocks in the Franciscan subduction zone were uplifted.

The Merced Basin is also a subset of the Colma Channel, an ancient watercourse leading from San Francisco Bay to the Pacific Ocean. This northwest trending channel, which was formed during late Cretaceous and Tertiary time, is bounded to the west by the San Andreas Fault and to the east by the inactive San Bruno Fault and the present day San Bruno Mountains. The San Bruno Fault is thought to be an inactive, westward dipping normal fault.

During the early Pleistocene Sangamon interglacial, about 100,000 years ago, sea level was higher than it is today and much of the San Francisco shoreline, including the Merced Basin and the Colma Channel, was below water and connected to the Pacific Ocean. The Merced Basin and the Colma Channel were filled with marine as well as continental (Franciscan) sediments. During the middle Pleistocene Wisconsin glacial, approximately 15,000 years ago, the Merced Basin, Colma Channel and parts of San Francisco Bay, were above water. The shoreline of what we now call Ocean Beach stretched up to twenty miles westward, above sea level. During this time, the Sacramento River flowed to the ocean and deposited sand derived from the granitic, plutonic and andesitic materials of the Sierra Nevada at Ocean Beach.

As glaciers melted and sea level rose, approximately 10,000 years ago, the Merced Basin and Colma Channel were filled with alluvial fan and remains above sea level to the present day. The sediments filling the Merced Basin are up to 5,000 feet thick and are now called the Merced Formation.

The present-day bluffs at Ocean Beach are uplifted Merced and Colma Formation sedimentary units with a lithology reflecting the landward and seaward movement of the shoreline over episodes of glacial maximums and minimums. There are alternating layers of marine sediments, such as silts and clays, coarser sediments such as sand and gravel deposited in the surf zone, “backshore” sediments such as fine-grained sands, silts and muds deposited along coastal embayments, as well as nonmarine estuarine fine-grained sediments and wind-blown sands.

2.2.2 Site Geology

In the project vicinity, the major geologic units are: historical artificial fill (Qaf), Holocene-age dune sand (Qd) and beach sand (Qb), Pleistocene-age Colma Formation (Qc), Pliocene-age Merced Formation (Qm) and Jurassic and Cretaceous Franciscan Complex (KJf).

2.2.2.1 Artificial Fill (Qaf)

Artificial fill exists along the entire alignment. In the vicinity of Sloat Boulevard, Ocean Beach is separated from the Great Highway by a rubble wall about 100 feet wide and 20 to 23 feet above high water. This rubble wall was built in the late 1800s to early 1900s in order to provide a surface for the construction of the Great Highway. The rubble consists of angular fragments of red chert, sandstone and greenstone of the Franciscan Formation, in a mixture of sand and clay. Artificial fill that resulted from grading operations during development of the project alignment is derived from native sediments, making it difficult to distinguish from dune sands and weathered unconsolidated Colma Formation sands.

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

2.2.2.2 Dune Sand (Qd)

In the project vicinity, Holocene dune sand deposits (Qd) extend from the western edge of Lake Merced to the coast. These deposits were fed by sand blown east from Ocean Beach and were deposited in the post-glacial period within the last 10,000 years. The thickness of the dune sand ranges from light cover at the tops of the highest bluffs, to up to 50 feet inland of the coast in the project. Near-surface dune sands tend to be poorly graded, fine to medium grained clean sand, whereas sands at depth may have light cementation or laminations.

2.2.2.3 Beach Sand (Qb)

Beach sand in the project vicinity is comprised of loose, well-sorted quartz and feldspar sand, which grades fine to coarse depending on its location in the surf zone. The beach sand at Ocean Beach has heavy mineral laminations, as well as thick layers of sand comprised of magnetite at the beach surface at the toe of the bluffs. These sands primarily originated from the Sierra Nevada during the previous low-stand sea level, when the San Francisco Bay was above water. Some of the sands are also comprised of continental (Franciscan) sediment outwash.

2.2.2.4 Colma Formation (Qc)

The Colma Formation overlies the tilted beds of the Merced Formation at an angular unconformity. Inland units of the Colma Formation include poorly consolidated colluvial, stream and eolian deposits, whereas coastal deposits of the Colma are more likely to be marine (nearshore and backshore) and estuarine in nature. Like the Merced Formation, the facies of the Colma Formation were formed with similar sedimentary material as well as in a similar depositional environment created by the rising and falling sea level causing transgression and regression of the shoreline.

Yi (2005), McGuire (2009) and others have described the Colma as a an approximately 3-foot thick “thin erosional remnant” at Ocean Beach in the project vicinity, and up to about 40 feet thick progressing south to Thornton Beach. However, Woodward-Clyde (1977) indicated that the

Colma is up to 200 feet thick in the vicinity of the SWOO and overlies the Merced, and Bonilla (1988) maps the Colma as underlying the northern two-thirds of the site.

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

According to Bonilla (1998) and Kennedy (2002), the Colma Formation is of latest Pleistocene age and was deposited between about 70,000 to 130,000 years ago.

2.2.2.5 Merced Formation (Qm)

The Merced Formation overlies the Franciscan Complex in the project vicinity, and consists of an accumulation of poorly consolidated sand, clay, gravel and silt sediments, which were deposited almost continuously in the late to early Pleistocene. Clifton and Hunter (1988) mapped a sequence of approximately 40 facies in the variably tilted and uplifted Merced exposures in the seaside cliffs, from the southern edge of the project alignment near Boring B-6 and continuing south approximately 7 kilometers to Mussel Rock. These sequences consist of marginal marine sediments, such as shelf, nearshore, backshore, embayment and fluvial facies, and their arrangement is indicative of alternate transgression and regression of the sea during geologic time. Yi (2009) mapped exposures of the Merced Formation in the project vicinity, from Sloat Boulevard to Thornton Beach, which is approximately 4.5 kilometers south of Sloat.

According to Hall (1965), Clifton (1988), Yi (2009), Kennedy (2002) and McGuire (2005), the Sequence X and Y of the Merced Formation are exposed closest to the project vicinity. These facies are generally weakly lithified to well-cemented, thinly bedded silts, sands, clays, and pebbly shell hash deposited in a shallow marine environment. Merced Formation at depth in the project vicinity is assumed to be characterized by light gray to dark gray and black fine-grained sand high in heavy minerals such as magnetite, and dark bluish gray fat clays with silty interbeds. Micaceous material indicative of backshore deposits is also possible in the Merced.

Based on the tectonic history of the Serra Fault, the Merced Formation can show bedding ranging from near-horizontal in the project vicinity, to up to 25 degrees and striking northeast in the vicinity of Fort Funston and Mussel Rock. Based on Woodward-Clyde's Boring B-1, the Merced Formation in the project vicinity can extend to approximately 400 feet depth at the contact with the Franciscan Formation.

2.2.2.6 Franciscan Complex (KJf)

Franciscan Complex rocks underlying the project alignment and its vicinity include graywacke sandstone, siltstone, claystone, and shale.

2.2.2.7 Discussion

AGS met with Professor John Caskey of San Francisco State University (SFSU) on March 19th, 2019 to discuss the subsurface stratigraphy at the project alignment. Professor Caskey and his graduate students have studied the Colma and Merced Formations in the site vicinity and south of the site vicinity for the past two decades. Yi (2005) mapped outcrops of the Merced Formation on the cliff exposures starting at Sloat Boulevard and continuing south 7 kilometers (km) to Thornton Beach. Yi also tested samples from the Colma, Merced, and Dune Sand units for grain size distribution and petrography. Kennedy (2002) hypothesized that the Merced Formation has been folded by and is bounded by the Serra Fault in the project vicinity, and he used optical luminescence to date the units. In a personal communication dated March 2019, Caskey indicated that the project alignment is likely to be chiefly underlain by Merced Formation in the near surface. He added that it is extremely difficult to distinguish between the bottom-most units of the Colma and the uppermost units of the Merced formations, as they are comprised of nearly identical material. This is a point to which others have also alluded (Woodward-Clyde 1977, Clifton and Hunter 1988). In their 1977 report, Woodward-Clyde indicates that the project alignment is underlain in the near surface by dune sands, then Colma formation up to 200 feet depth, and by Merced Formation up to 5,000 feet depth.

2.3 <u>SUBSURFACE GEOTECHNICAL CONDITIONS</u>

2.3.1 <u>Subsurface Stratigraphy</u>

Based on the material encountered in our borings, CPTs, and potholes, as well as the results of the geotechnical and geological lab test results, a generalized site stratigraphy profile was developed. The site stratigraphy shown in Table 2 and on Plates 4A to 4E represents AGS's estimate of the thicknesses of the units. The subsurface stratigraphy along the SWOO cross section trending east to west towards the ocean, crossing the LMT, is presented on Plate 5.

Boring B-1 was drilled in the beach access parking lot at the intersection of Sloat Boulevard and the Great Highway. Boring B-1 encountered approximately 20 feet of brown, loose to medium dense poorly-graded sand, with lenses of silty sand. Approximately 10 feet of dense, black and gray poorly graded sand with silt, underlies the medium dense sand. Approximately 40 feet of dense to very dense, bluish gray poorly graded sand, underlies the dense sand, which is underlain by 30 feet of bluish gray, medium dense to very dense silty sand and poorly graded sand with silt. Boring B-1 encountered very stiff fat clay at 100 feet depth to the bottom of the boring at 101.5 feet depth.

Boring B-2 encountered approximately 12 feet of loose to medium material comprised of grayish brown, poorly graded sand with trace silt. This is underlain by approximately 13 feet of dark gray, medium to dense poorly graded sand. This is underlain by about 25 feet of reddish brown and gray, dense to very dense, poorly graded sand and 31.5 feet of dark gray, loose to very dense silty sand and sand with silt, or very soft sandy clay.

Boring B-3 encountered approximately 12 feet of loose to medium dense brown and grayish brown sand overlying approximately 5 feet of medium dense dark gray poorly graded sand. These layers are underlain by approximately 40 feet of dense to very dense poorly graded sand with silt and silty sand. This layer is underlain by 40 feet of very dense silty sand and poorly graded sand and very stiff silt with sand. At 100 feet depth, fat clay was encountered, to the bottom of the boring at 101.5 feet depth.

Boring B-4 encountered approximately 13 feet of loose to medium dense, poorly graded sand, underlain by about 5 feet of medium dense to dense poorly graded sand with silt and silty sand. Underlying this layer is 50 feet of dense to very dense silty sand and dense clayey sand, including a five-foot layer of very stiff fat clay. Abundant mica was identified in samples between about 35 feet to 45 feet depth. The dense silty sand layer is underlain by 5 feet of dense silty sand and 6.5 feet of fat clay up to the bottom of the boring at 81.5 feet depth.

Boring B-5 encountered approximately 9 feet of loose to medium dense, reddish brown silty sand, underlain by approximately 6 feet of medium dense, yellowish brown, poorly graded sand with silt. This layer was underlain by approximately 36.5 feet of dense to very dense brown poorly graded sand with silt and very dense reddish brown silty sand, to the bottom of the boring at 51.5 feet depth.

Boring B-6A encountered fill up to the bottom of the boring at about 38 feet depth. The fill was brown and reddish brown, dense to very dense, poorly graded sand with silt. Boring B-6 encountered refusal on concrete. Boring B-6B was drilled adjacent to Boring B-6A with rotary wash up to a depth of about 30 feet without sampling and continued to about 35.5 feet with sampling. Boring B-6B encountered reddish brown, dense, poorly graded sand with silt fill from 30 to 35.5 feet depth. Boring B-6B refused at about 35.5 feet depth.

TABLE 2
SUMMARY OF SUBSURFACE CONDITIONS

Reach	Representative Borings ID	Loose to Medium Dense Sand Thickness (feet)	Medium Dense Sand Thickness (feet)	Sand / Sand with Silt[1] Layer Thickness (feet)	Sand[1], Silty Sand[1], Silt[2], Clay[2] Layer Thickness (feet)	Maximum Depth of Exploration (feet)
North	B-1, R3-1[3], B-6[4], CPT-2	20	5-15	50-65	>31.5	101.5
EQR	B-2, CPT-3, CPT-4	10-20	0-10	30-40	>31.5	100.5
Rubble	B-3, B-4, B-5[4], CPT-7, CPT-9	5-15	5-10	35-55	>31.5	101.5
Bluff	B-5, R2-1[3], CPT-10	10-20	5-15	40-60	>6.3	79.7
South	B-6, R1-C2[3], R1-C3[3], R1-C1[3], R1-B3[3], R1-B1[3], R1-A1[3], CPT-11, CPT-12, CPT-13	30-40	0-10	>10	>40	76.3

Notes:
1. Sand, Sand with Silt, and Silty Sand were generally dense to very dense in these layers.
2. Silt and Clay were generally stiff to hard in these layers.
3. Boring from AGS 2010
4. Boring from AGS 1989

2.3.2 Groundwater Levels

Depth to groundwater was measured in monitoring wells MW-1, MW-2, and MW-3, companion holes to Boring B-1, B-4 and B-5, respectively. Groundwater was encountered at a depth of about 22 feet below grade in MW-1, at the approximate elevation of sea level at the time of our reading. The rotary wash method prevented groundwater readings in the remaining borings, Borings B-2, B-3 and B-6.

Table 3 presents depth to groundwater encountered in each of the exploration monitoring wells, as well as in borings from previous explorations.

TABLE 3
GROUNDWATER LEVEL DATA

Well ID	Date Measured	Ground Surface Elevation, NAVD88 (feet)	Groundwater Elevation, NAVD88 (feet)	Depth to Groundwater (feet)	Total Depth (feet)	Source
MW-1	3/15/19	31.4	9.4	21.96	25.09	AGS 2019
	5/31/19	31.4	8.7	22.70	25.04	AGS 2019
	6/28/19	31.4	8.1	23.31	26.02	AGS 2019
	7/27/19	31.4	8.2	23.18	25.08	AGS 2019
	10/2/19	31.4	9.0	22.41	25.05	AGS 2019
	11/6/19	31.4	8.3	23.08	25.05	AGS 2019
	12/10/19	31.4	8.9	22.46	25.05	AGS 2019
	2/5/20	31.4	9.1	22.28	25.05	AGS 2020
	4/1/20	31.4	8.6	22.80	25.05	AGS 2020
	4/8/20	31.4	8.9	22.55	25.05	AGS 2020
	5/29/20	31.4	8.9	22.48	25.05	AGS 2020
	6/26/20	32.4	9.1	23.34	25.05	AGS 2020
MW-2	3/15/19	30.4	8.4	22.01	27.70	AGS 2019
	5/31/19	30.4	7.8	22.58	26.66	AGS 2019
	6/28/19	30.4	7.4	23.02	27.63	AGS 2019
	7/24/19	30.4	7.6	22.81	27.28	AGS 2019
	10/2/19	30.4	8.6	21.76	27.64	AGS 2019
	11/6/19	30.4	8.1	22.35	27.58	AGS 2019
	12/10/19	30.4	8.4	22.04	27.57	AGS 2019
	2/5/20	30.4	8.0	22.42	27.57	AGS 2020
	4/1/20	30.4	7.7	22.75	27.34	AGS 2020
	4/8/20	30.4	8.0	22.39	27.34	AGS 2020
	5/29/20	30.4	8.6	21.85	27.34	AGS 2020
	6/26/20	31.4	8.4	23.05	27.34	AGS 2020
MW-3	3/15/19	28.9	5.6	23.33	28.30	AGS 2019
	5/31/19	28.9	4.2	24.75	29.32	AGS 2019

TABLE 3 CONTINUED
GROUNDWATER LEVEL DATA

Well ID	Date Measured	Ground Surface Elevation, NAVD88 (feet)	Groundwater Elevation, NAVD88 (feet)	Depth to Groundwater (feet)	Total Depth (feet)	Source
MW-3	6/28/19	28.9	5.1	23.79	28.22	AGS 2019
	7/24/19	28.9	5.5	23.41	28.22	AGS 2019
	10/2/19	28.9	5.8	23.07	28.23	AGS 2019
	11/6/19	28.9	5.4	23.55	28.20	AGS 2019
	12/10/19	28.9	5.4	23.50	28.22	AGS 2019
	2/5/20	28.9	5.4	23.51	28.22	AGS 2019
	4/1/20	28.9	5.3	23.62	28.22	AGS 2020
	4/8/20	28.9	5.3	23.64	28.22	AGS 2020
	5/29/20	28.9	5.9	23.05	28.22	AGS 2020
	6/26/20	28.9	5.1	23.84	28.22	AGS 2020
B-5 (AGS)	5/24/89	29.4	10.4	19.0	60.0	AGS 1989
B-6 (AGS)	5/24/89	31.4	8.4	23.0	70.0	AGS 1989
HLA-54	6/24/77	31.9	11.4	20.5	101.5	HLA 1977
WC-4	6/6/77	36.4	6.9	29.5	80.0	W-C 1977
WC-10	6/6/77	48.5	13.5	35.0	60.0	W-C 1977
B-2 (GTC)	10/8/15	32.4	20.1	12.3	111.5	GTC 2016

2.4 SEA LEVEL RISE SCENARIO

As indicated in Section 4.6 of the Conceptual Engineering Report (CER), Coastal erosion will increase with sea-level rise. Additional factors impacting coastal erosion events include high tides, storm surge, El Niño effects, and elevated groundwater tables.

2.5 FAULTING AND SEISMICITY

2.5.1 Surface Fault Rupture

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

2.5.2 Historical Seismicity

The project area is located in a seismically active region subject to periodic earthquakes causing strong to violent ground shaking of the site. The San Andreas Fault is about 2.6 kilometers (km) southwest of the site and is the major fault system in the region. Further from the project alignment are the San Gregorio Fault, which is 7.7 km southwest of the site, the Hayward Fault, which is about 27.4 km to the northeast; both are also significant seismic sources. Other major active faults considered capable of causing significant shaking at the project alignment by the United States Geological Survey (USGS) include the Point Reyes, Monte Vista-Shannon, Mount Diablo Thrust, Calaveras, Green Valley, West Napa, Greenville and Great Valley faults. Active fault traces and epicenters of recent earthquakes are shown on Plate 6 – Earthquake Epicenters and Fault Map. Historic earthquakes attributed to each fault are listed in Table 4 - Historical Earthquakes.

AGS

TABLE 4
HISTORICAL EARTHQUAKES

Date	Magnitude	Fault	Epicenter Area
June 24, 1808	6.0[5]	Unknown	Uncertain, San Francisco Bay Area
June 10, 1836	6.5[1], 6.8[5]	San Andreas	San Juan Bautista
June 1838	7.5[1], 7.0[5]	San Andreas	San Juan Bautista
Nov. 26, 1858	6.25[5]	Calaveras	San Jose Area
February 26, 1864	6.0[5]	San Andreas	South Santa Cruz Mountains
March 26, 1864	6.0[5]	San Andreas	Santa Cruz Mountains
October 8, 1865	6.3[2], 6.5[5]	San Andreas	South Santa Cruz Mountains
October 21, 1868	7.0[2,5]	Hayward	Berkeley Hills, San Leandro
February 17, 1870	6.0[5]	San Andreas	Los Gatos
April 12, 1885	6.25[5]	San Andreas	South Diablo Range
May 19, 1889	6.25[5]	Concord-Green Valley	Antioch
April 24. 1890	6.25[5]	San Andreas	Pajaro Gap
April 19, 1892	6.5[5]	Great Valley	Vacaville
April 21, 1892	6.25[5]	Great Valley	Winters
June 20, 1897	6.25[5]	Calaveras	Gilroy
March 31, 1898	6.5[5]	Rodgers Creek	Mare Island
April 18, 1906	8.0[3]	San Andreas	Golden Gate
July 1, 1911	6.6[4], 6.5[5]	Calaveras	Diablo Range, East of San Jose
October 22, 1926	6.1[5]	San Gregorio	Monterey Bay
April 24, 1984	6.1[5]	Calaveras	Morgan Hill
October 17, 1989	7.1[5]	San Andreas	Loma Prieta, Santa Cruz Mountains
August 24, 2014	6.0[6]	West Napa	South Napa, American Canyon

1) Borchardt & Toppozada (1996)
2) Toppozada, et al. (1981)
3) Petersen, et al (1996)
4) Real, et al (1978), Toppozada (1984)
5) Ellsworth, W.L. (1990)
6) GEER (2014)

2.5.3 Regional Active Faulting

The maximum moment magnitude earthquake (Mmax) is defined as the largest earthquake that a given fault is considered capable of generating. The Mmax earthquake on the San Andreas Fault will be a magnitude 8.05 event occurring approximately 2.6 km (1.6 miles) from the project alignment. The seismicity associated with each pertinent fault within 70 kilometers, including estimated slip rates, is summarized in Table 5 - Fault Seismicity.

TABLE 5
FAULT SEISMICITY

Fault Name	Distance to site [2] (km)	Maximum Moment Magnitude [1]	Contributing Segments [2]	UCERF 3 Slip Rate [1] (mm/year)
San Andreas	2.6	8.05	Peninsula (SAP) + Santa Cruz Mountains (SAS) + Offshore (SAO) + North Coast (SAN)	17.0 17.0 24.0 24.0
San Gregorio Connected	7.7	7.50	San Gregorio (North) + San Gregorio (South	7.0 3.0
Hayward-Rodgers Creek	27.4	7.33	Rodgers Creek (RC) + Hayward Northern (HN) + Hayward Southern (HS)	9.0 9.0 9.0
Point Reyes	37.3	6.90	Point Reyes	0.1
Monte Vista - Shannon	39.6	6.50	Monte Vista-Shannon (MVS)	0.6
Mount Diablo Thrust	44.2	6.70	Mount Diablo Thrust North Mount Diablo Thrust South	2.0 2.0
Calaveras	44.8	7.03	Calaveras North (CN) + Calaveras Central (CC) + Calaveras South (CS)	6.0 15.0 15.0
Green Valley Connected	49.1	6.80	Green Valley	4.0
West Napa	53.4	6.70	West Napa (WN)	1.0
Greenville Connected	61.6	7.00	Greenville North Greenville South	3.0 3.0
Great Valley 5, Pittsburg Kirby Hills	66.2	6.70	Great Valley 5, Pittsburg Kirby Hills	1.5

1) WGCEP (2003, 2008), Working Group on California Earthquake Probabilities
Map distance to the nearest segment, based on USGS Quaternary Fault and Fold Database (2006)
2) WGCEP (2008), Tables I-1 and I-3 of Appendix I. Parameters for Faults in California, 2008, “Documentation for the 2008 Update of the United States National Seismic Hazard Maps” and UCERF3 (2014).

2.6 LANDSLIDES

The project alignment is generally not located within a State of California designated Seismic Hazard Zone for earthquake-induced landslides (CGS, 2000). The sea cliff adjacent to the southern end of the project alignment (south of approximately Station 42+00) is mapped by the CGS to be in an area considered potentially susceptible to earthquake-induced landslides. Based on our review of the published geologic data, including Bonilla (1998) and Clifton and Hunter (1999), the materials mapped at the sea cliff mostly consist of the Merced Formation with favorably-oriented beddings (dipping into the slope of the sea cliff). As noted above, the landslide hazard associated with the project alignment is primarily due to coastal erosion. It is our opinion that, after the construction of the proposed secant pile wall in conformance with our geotechnical recommendations, the potential for future landslides to adversely affect the LMT will be low.

2.7 TSUNAMI

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

We note that tsunami risk, including the inundation zone, is increased with sea level rise.

2.8 CORROSION POTENTIAL

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

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

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 GENERAL

Based on the results of our data review, field exploration, laboratory testing, and engineering analyses, it is our opinion that the proposed South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection Project is feasible from a geotechnical point of view, provided the recommendations presented in this report are incorporated in the design and construction of the project. Conclusions and design recommendations for the secant pile wall, seismic design, site preparation and grading, and construction considerations are presented the following sections.

The following recommendations are for designing of a soil retention system which will resist lateral earth pressures based on the ultimate retaining condition when the bluff in front of the wall has resulted in loss of soils to a beach level of Elevation +2 feet for short term (equivalent to 72-year return period or 50 percent chance of occurrence in 50 years) and +10 feet for long-term (equivalent to 975-year return period or 5 percent chance of exceedance in 50 years).

3.2 DESIGN GROUNDWATER LEVEL

Groundwater levels recorded in previous borings and monitoring wells generally range from approximately Elevation +5.5 to +13.5 feet. SFPUC's 2017 Annual Groundwater Monitoring Report for Westside Basin indicates groundwater at approximately Elevation +10 feet at the site vicinity (based on the Groundwater Elevation Contours for Shallow Aquifer, Spring 2017). The construction of the secant pile wall may induce changes in groundwater conditions and therefore we recommend installation of a subdrain system at the top of the secant pile wall (behind the grade beam) discharging to a suitable free-drainage outlet. For design purposes, we recommend that groundwater levels at Elevation +10.7 feet (for the North Reach), Elevation +11.9 feet (for the EQR Reach), Elevation +12.6 (for the Rubble Reach), Elevation +13.6 feet (for the Bluff Reach), and Elevation +17 feet (for the South Reach) be considered behind the proposed secant wall.

3.3 SEA LEVEL RISE SCENARIO

As indicated in Section 4.6 of the Conceptual Engineering Report (CER), Coastal erosion will increase with sea-level rise. Additional factors impacting coastal erosion events include high tides, storm surge, El Niño effects, and elevated groundwater tables.

3.4 DESIGN GROUND MOTION

Ground motion design parameters at the Ocean Beach site were obtained through site-specific analyses for a Seismic Performance Class III facility per Section 2.2.3 of the SFPUC 2014 GSR. Per Section 2.2.3 of the SFPUC 2014 GSR, Design ground motion for facilities in Seismic Performance Class III should have a 5% probability of exceedance in 50 years (975-year return period). According to this section of the SFPUC 2014 GSR, design spectra for facilities in Seismic Performance Class III should be obtained from a site-specific evaluation. Procedure outlined in ASCE 7-16 with Supplemental 1 was used to develop site specific response spectra. Both 72-year and 975-year return period acceleration response spectra were obtained from a site-specific Probabilistic Seismic Hazard Analysis (PSHA) with the latest Western United States ground motion model called the 2014 Next Generation Attenuation West-2 (NGA-West2) relationships developed by Abrahamson, Silva, and Kamai (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), and Chiou and Youngs (2014). Based on the subsurface profile at the site, the site is classified as Site Class D – Stiff Soil with estimated shear wave velocity profiles in the upper 30 meters, or 100 feet, of the ground surface (Vs30) of about 220 meters per second (mps), or about 720 feet per second (fps).

Additionally, we developed ground surface spectral acceleration for 50% probability of exceedance in 50 years (72-year return period) for extreme wall-toe erosion at which beach level is at Elevation +2 feet. Table 6 shows probabilistic and 84th percentile deterministic accelerations for base level and 975-year return period. Based on Section 21.1.2 of ASCE 7-16, the site response model is permitted to be terminated where the soil stiffness is at least as great as the values used to define Site Class D provided that the very deep soil profiles make the development of a soil model to bedrock impractical. In such cases, the MCE_R response spectrum and acceleration time histories of the base motion developed shall be adjusted upward using site coefficients presented in Section 11.4.3 consistent with the classification of the soils at the profile base. Design level base and ground surface spectra accelerations for both 975-year and 72-year return periods are presented in Table 7 and Plate 7.

TABLE 6
RECOMMENDED 975-YEAR SPECTRAL ACCELERATIONS (5% DAMPING)

Structural Period (sec)	975-year Probabilistic (g)	84th-percentile Deterministic (g)	Lower of Probabilistic or Deterministic (g)	Deterministic Lower Limit (g)	Design Spectrum Acceleration (g)
0.01	0.96	0.99	0.96	0.65	0.96
0.02	0.97	0.99	0.97	0.69	0.97
0.03	1.01	1.01	1.01	0.74	1.01
0.05	1.00	1.04	1.00	0.83	1.04
0.08	1.23	1.23	1.23	0.96	1.23
0.10	1.55	1.41	1.41	1.05	1.41
0.15	1.90	1.68	1.68	1.28	1.68
0.20	2.03	1.86	1.86	1.50	1.86
0.25	2.07	2.03	2.03	1.50	2.03
0.30	2.05	2.13	2.05	1.50	2.05
0.40	2.00	2.25	2.00	1.50	2.00
0.50	1.92	2.23	1.92	1.50	1.92
0.75	1.52	1.84	1.52	1.50	1.52
1.00	1.24	1.51	1.24	1.50	1.50
1.50	0.85	1.06	0.85	1.00	1.00
2.00	0.65	0.78	0.65	0.75	0.75
3.00	0.44	0.53	0.44	0.50	0.50
4.00	0.31	0.38	0.31	0.38	0.38
5.00	0.25	0.29	0.25	0.30	0.30
7.50	0.13	0.15	0.13	0.20	0.20
10.00	0.08	0.09	0.08	0.15	0.15

TABLE 7
RECOMMENDED SPECTRAL ACCELERATIONS (5% DAMPING)

Structural Period	Design Spectrum Acceleration (72-year return period)		Design Spectrum Acceleration (975-year return period)	
	Base Ground Motion	Surface Ground Motion	Base Ground Motion	Surface Ground Motion
(sec)	(g)	(g)	(g)	(g)
0.01	0.26	0.25	0.98	0.96
0.02	0.26	0.25	0.99	0.97
0.03	0.28	0.25	1.03	1.01
0.05	0.33	0.28	1.20	1.04
0.08	0.42	0.36	1.48	1.23
0.10	0.5	0.43	1.68	1.41
0.15	0.6	0.54	1.99	1.68
0.20	0.63	0.61	2.23	1.86
0.25	0.63	0.64	2.38	2.03
0.30	0.59	0.66	2.41	2.05
0.40	0.52	0.64	2.35	2.00
0.50	0.44	0.59	2.17	1.92
0.75	0.30	0.52	1.68	1.52
1.00	0.20	0.52	1.28	1.50
1.50	0.12	0.19	0.85	1.00
2.00	0.07	0.14	0.57	0.75
3.00	0.04	0.09	0.36	0.50
4.00	0.03	0.07	0.26	0.38
5.00	0.02	0.06	0.20	0.30

Based on the methods of SFPUC General Seismic Requirements (SFPUC 2014 GSR), site specific spectral accelerations were developed for the project. According to Section 2.2.3 of the SFPUC GSR, design ground accelerations at the base below ground structures with Seismic Performance Class III were developed with 5 percent probability of exceedance in 50 years (975-year return period). The design ground acceleration at the base were capped to a deterministic limit taken as the 84th percentile level for the maximum earthquake with a lower bound of the deterministic Maximum Considered Earthquake (MCE) as defined in Section 21.2.2 of ASCE 7.

3.5 LIQUEFACTION

Soil liquefaction is a phenomenon in which saturated (submerged), loose to medium dense cohesionless soils lose their strength due to the build-up of excess pore water pressure, especially during cyclic loadings such as those induced by earthquakes. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movements, if not confined. Soils most susceptible to liquefaction are loose, clean sands. Silty sands and low-plasticity silts may also liquefy during strong ground shaking. Based on the assessment of Youd and Perkins (1978) on different soil susceptibilities to liquefaction, Pleistocene age (11,000 to 2 million years ago) sedimentary deposits, including alluvial fan and plain (Colma Formation) and marine terraces and plans (Merced Formation) generally have low to very low susceptibility to liquefaction.

3.5.1 Liquefaction Potential

The liquefaction potential of soils at the site was evaluated based on the method described in Idriss and Boulanger (2014) and the groundwater and modified peak ground acceleration (PGA_M). The maximum considered earthquake geometric mean peak ground acceleration (PGA_M) was developed for two hazard levels: 0.25g for 72-year return period and 0.96g for 975-year return period. The values were used to assess the potential for liquefaction and for dynamic seismic lateral earth pressure.

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

3.5.2 Consequences of Liquefaction

The main effects of liquefaction at the site may include settlement of the ground surface, lateral deformation, development of excess pore water pressure, buoyancy effects on the below groundwater structures, loss of allowable bearing pressure, downdrag force on the proposed secant pile wall, and increased lateral pressures on below grade structural elements, and foundations extending below the groundwater table.

3.5.3 Liquefaction-Induced Settlement

Liquefaction of the saturated, loose to medium dense sandy soils may occur during a major earthquake and result in liquefaction-induced settlement. The estimated liquefaction-induced settlements and the thickness of potentially liquefiable layers at the various boring and CPT locations are presented in Table 8.

TABLE 8

ESTIMATED LIQUEFACTION-INDUCED SETTLEMENTS

Boring or CPT	Reach	Ground Surface Elevation, NAVD88 (feet)	Depth to LMT Spring Line (feet)	Depths of Liquefiable Layers (feet)	Liquefaction-Induced Settlement (inches) 72-yr	975-yr
B-1	North	31	35	15-20	0	1
CPT-2	North	29.5	30	14-15 16-21 51-54	¼ ½ ¼	¼ ¾ ¼
CPT-14	North	5.5	N/A	1-3 10-14 20-21 25-27	¾ ½ ¼ ¾	¾ ½ ¼ ¾
B-2	EQR	30	27	14-15 50-56	0 1½	½ 2
CPT-3	EQR	30	30	14-17 18-23 32-35	0 ¼ ¼	½ 1 ¼
CPT-4	EQR	30.5	27	15-20 42-46	¼ 0	1 ¼
CPT-5	EQR	28	28	12-23 45-46 48-51	1¾ ¼ ¼	3½ ¼ ¼
CPT-15	EQR	4.5	N/A	1-6 22-52	1 8	1 8
B-3	Rubble	28.5	25	NL	0	< ¼
B-4	Rubble	29	27	NL	0	< ¼
AGS 1989 B-5	Rubble	29	28	15-20	0	1½
CPT-6	Rubble	29	26	13-19 31-38	1 ½	1½ 1
CPT-7	Rubble	29	26	13-22 28-31 38-40	½ 0 0	1½ ¼ ¼

TABLE 8 (CONTINUED)

ESTIMATED LIQUEFACTION-INDUCED SETTLEMENTS

Boring or CPT	Reach	Ground Surface Elevation (NAVD88) (feet)	Depth to LMT Spring Line) (feet)	Depths of liquefiable layers (feet)	Liquefaction-induced settlement (inches) 72-yr	975-yr
CPT-8	Rubble	29	25	13-14 21-22 25-26 32-34	¼ 0 0 ¼	½ ¼ ¼ ¼
CPT-9	Rubble	29	27	49-52	0	¼
CPT-16	Rubble	4.5	N/A	2-6 12-50	2 ½	2 ½
CPT-17	Rubble	4.5	N/A	2-8 22-29 33-36 41-66	2 1 ¾ 6	2 1 ¾ 6
B-5	Bluff	31	28	NL	0	< ¼
CPT-10	Bluff	36	31	20-24 32-33	0 0	¼ ¼
CPT-18	Bluff	4.5	N/A	1-5	1¼	1¼
AGS 2010 R1-C2	South	39	34	20-25	1	1
AGS 2010 R1-B3	South	43	38	25-38 38-50	1 ¼	3½ 1¼
AGS 2010 R1-B1	South	46	40	28-30 40-43	¾ ¾	¾ 1
AGS 2010 R1-A1	South	50	45	32-40	1¼	1¼
CPT-11	South	60	55	39-40	¼	¼
CPT-12	South	65	59	45-47	¼	¼
CPT-19	South	6.5	N/A	1-3	1	1

1) NL = Non-liquefiable
2) Elevation at beach level should be considered as approximate

It is noted that the majority of the potentially liquefiable soils are located above the spring line of the LMT. The estimated liquefaction-induced settlements below the spring line of the LMT are presented in Table 9. To assess the impact of liquefaction on the structural integrity of the LMT, a numerical modeling study using finite difference analyses (FLAC) was performed during the design phase. The results from FLAC numerical modeling study are presented in Appendix C.

TABLE 9
LIQUEFACTION-INDUCED SETTLEMENT BELOW LMT SPRING LINE

Reach	Liquefaction-induced Settlement below Spring Line of LMT (inches)
North	¼
EQR	¼ - 2
Rubble	¼ - 1
Bluff	¼
South	¼ – 1¼

3.5.4 Liquefaction-Induced Lateral Deformation

Liquefaction-induced lateral deformation (also referred to as lateral spreading) is lateral movement of surficial soil mass towards a free face (such as the coastal bluff and beach slopes) during earthquakes. It typically occurs when a continuous layer of sands liquefies during a major earthquake and the overlying non-liquefiable crust slides as large blocks over the liquefied soils, creating fissures and scarps. It is our opinion that, majority of the lateral spreading along the proposed secant retaining wall will be above the pile cap. For the area where liquefaction-induced lateral deformation extends below the pile cap, it is our opinion that after the construction of the proposed secant pile wall in conformance with our geotechnical recommendations in this report, the continuity of the potentially liquefiable soil will be interrupted. If the proposed secant pile wall is designed to withstand liquefaction-induced lateral forces, then the potential for lateral spreading adversely affecting the LMT will be low.

3.5.5 Liquefaction-Induced Lateral Earth Pressure

If the soils behind the secant pile wall liquefy during a major earthquake, the lateral earth pressure exerted on the wall would be momentarily increased. Our recommendations on the liquefaction-induced lateral earth pressure for design are presented in Section 3.8.2 - Lateral Earth Pressures.

3.5.6 Liquefaction-Induced Uplift Pressure

Soil liquefaction can also result in an increase in uplift pressure on buried structures surrounded by liquefiable soils during a major earthquake. Such condition generally occurs when saturated, loose to medium dense sandy soils around the buried structures liquefy and lose their shear resistance against uplift from buoyance of the buried structures. The majority of the potentially liquefiable soils encountered is either above the spring line of the LMT or relatively thin localized layers that are approximately 1 to 2 feet thick below the spring line of the LMT. The risk of uplift of the LMT during a major earthquake was evaluated by numerical modeling, taking into account the reduction in shearing resistance of liquefied soils during earthquakes. The results of our numerical analyses are presented in the Soil-Structural-Interaction Technical Memorandum (AGS, 2021b) included in Appendix C.

3.5.7 Liquefaction Mitigation

The consequences of liquefaction, such as liquefaction-induced settlement and lateral deformations, have been discussed above. If the consequences of liquefaction are not acceptable to the design team, possible mitigation measures are discussed below.

Ground improvement of potentially liquefiable soils may consist of either deep soil mixing (DSM) or grouting techniques (such as jet grouting). Other ground improvement techniques are also available; however, they are apparently not feasible due to the site constraints, specifically over the concern of potential impact to the LMT.

DSM is a technique that involves mixing cementitious materials and in-place soils with a hollow-stem auger and paddle arrangement. Augers up to about 6 feet in diameter are commonly used to create soil-cement cells, and overlapping soil-cement columns are required in order to be an effective ground improvement. DSM may generate a significant amount of spoils that must be controlled and handled.

Chemical grouting involves the injection of low viscosity liquid grout (such as silicates, resins, microfine cements or polyurethane) under pressure into the pore spaces of granular soils that results in hardening of the soils by binding the soil grains together.

Jet grouting is another method of ground reinforcement that uses high kinetic energy in the form of a high velocity jet of grout to breakdown the soil structure and simultaneously mix cement grout with the in-situ soil.

DSM and grouting techniques are typically constructed through a design-build contract and specific design recommendations will depend on the methods and equipment used by the specialty contractors. If liquefaction mitigation would be considered, AGS can provide further recommendations upon request.

3.6 SEISMIC SETTLEMENT IN DRY SAND

Loose, unsaturated sandy soils tend to compress and settle during strong ground shaking from a major earthquake (called “seismic settlement in dry sand”). For analysis based on SPT data, we used the Tokimatsu and Seed (1987) method to estimate the seismic settlement in dry sand. For analysis based on CPT data, we used the Robertson and Shao (2010) method. In both cases, we applied the correction factor for multidirectional shaking as recommended by Pyke et al. (1975) and the limiting volumetric strain as recommended by Ishihara and Yoshimine (1992) as well as Idriss and Boulanger (2008) and Boulanger and Idriss (2014).

Table 10 shows the estimated seismic settlement in dry sand based on the results of our analyses. It is our opinion that, from a geotechnical engineering standpoint, the impact of the estimated seismic settlement in dry sand on the LMT down below will be low. The strengthened soil cover of the ultimate backslope (consisting of in-situ soil-cement mix or CLSM) may crack locally. However, the surface of the future slope (called “graded dune bluff”) is expected to be covered with minimum 4 feet of sand (including the SFPUC’s beach replenishment program to restore lost sand on a seasonal basis) and, if that is the case, pedestrian tripping hazards would not be a concern. The settlement may impact the access road and multi-use trail. Repair of the access road and multi-use trail, such as surface re-grading, may be needed after a major earthquake.

TABLE 10
ESTIMATED SEISMIC SETTLEMENT IN DRY SAND

Reach	Boring or CPT	Estimated Seismic Settlement in Dry Sand (inches)	Remarks
North	B-1	6.3	
	AGS 1989 B-6	8.6	
	CPT-1	2.8	170 feet north of LMT
	CPT-2	4.5	at north parking lot
EQR	B-2	2.8	
	CPT-3	1.1	
	SCPT-3	0.7	
	CPT-4	1.8	
	CPT-5	3.9	80 feet east of LMT
Rubble	B-3	3.0	
	B-4	1.6	
	AGS 1989 B-5	1.4	
	CPT-6	3.0	80 feet east of LMT
	CPT-7	1.7	
	CPT-8	0.1	
	CPT-9	0.1	
Bluff	B-5	1.6	
	CPT-10	0.4	
South	B-6	0.5	
	AGS 2010 R1-B3	1.3	
	AGS 2010 R1-C2	3.8	
	CPT-11	2.1	
	CPT-12	0.4	
	CPT-13	5.6	beyond project alignment

3.7 CONSTRUCTION SEQUENCING CONSIDERATIONS

We understand that the following two construction sequencing alternatives are under considerations:

Alternative A: Construct the Strengthened Soil Cover by In-situ Soil Cement Mix

1. Install the primary unreinforced and secondary reinforced concrete piles of the secant pile wall by drilling from the existing ground surface;
2. Perform soil improvement (by deep soil mixing or jet grouting from the existing ground surface) for the upper 4 feet of soil cover for the ultimate backslope behind the secant pile wall by in-situ soil-cement mix;
3. Similarly, perform soil improvement (from the existing ground surface) for soils above the ultimate backslope to allow for reduced effort when excavating the grade beam;
4. Excavate down to the bottom of grade beam elevation with open cut excavations (with dewatering where necessary) on both sides of the secant pile wall;
5. Construct the grade beam;
6. Install tiebacks after the grade beam has reached sufficient strength; and
7. Backfill the excavations with properly compacted engineered fill.

Alternative B: Construct the Strengthened Soil Cover by Controlled Low Strength Material (CLSM)

1. Install the primary unreinforced and secondary reinforced concrete piles of the secant pile wall by drilling from the existing ground surface;
2. Excavate down to the bottom of grade beam elevation with open cut excavations (with dewatering where necessary) on both sides of the secant pile wall;
3. Construct the grade beam;
4. Install tiebacks after the grade beam has reached sufficient strength;
5. Construct the upper 4 feet of soil cover for the ultimate backslope with CLSM, which consists of a fluid, workable mixture of cement, aggregate and water (to be placed in sections with terraced wooden forms); and
6. Backfill the excavations with properly compacted engineered fill.

From a geotechnical engineering standpoint, some considerations that may influence the selection of construction sequencing are presented below.

TABLE 11
SEQUENCING ALTERNATIVES

Alternatives	Advantages	Limitations
Improve soil cover of ultimate backslope by in-situ soil-cement mixing (Alternative A)	• Landward side open cut slope steeper than 1½H:1V possible, if soils above the ultimate backslope also to be improved. Otherwise, condition of open cut slope no steeper than 1½H:1V will remain.	• Relatively high cost of cement deep in-situ soil-cement mixing • Difficult to QA/QC • Could result in uneven finished surface of the backslope that may be undesirable for the ultimate condition
Construct soil cover of ultimate backslope with controlled low strength material (Alternative B)	• Relatively low cost of CLSM • Reliable QA/QC • Relatively homogeneous product	• Flat landward side open cut slope affecting existing roadway • Requires CLSM placement in sections with terraced wooden forms

3.8 SECANT PILE WALL

As noted above, the upper 4 feet of soil cover for the ultimate backslope will be strengthened (by either in-situ soil-cement mix or CLSM) to provide resistance to wave run-up over the top of the wall. If the strengthened soil cover would be constructed as a continuous, impervious blanket running longitudinally along the entire length of the secant pile wall, it could potentially act as a barrier to groundwater flow and may cause the groundwater level behind the wall to rise above the design groundwater level. Therefore, adequate drainage should be provided behind the grade beam such as installation of a subdrain system discharging to a suitable free-drainage outlet. The discharge system should be designed properly to avoid causing any slope instability.

The subdrain should consist of a vertical layer of Class 2 permeable material and a 4-inch diameter perforated PVC pipe (SDR 35). The vertical layer of permeable material should be at least 12 inches thick and should be placed from the bottom of the grade beam to about 1 foot below the finished grade behind the grade beam (and no obstruction of water flow by the strengthened soil placed behind). Alternatively, the Class 2 permeable material may be replaced by a gravel layer wrapped in a suitable geotextile fabric to reduce the potential for fines migration.

The perforated pipe should be placed near the bottom of the grade beam to carry collected water to a suitable gravity discharge. The discharge system should be designed properly to avoid causing any slope instability in front of the secant pile wall.

Tiebacks will be installed at the grade beam, extending back into the landward side beneath the LMT with a minimum clearance of 5 feet. The geotechnical recommendations for tiebacks are presented in section 3.12.

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

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

3.8.1 Vertical Capacities

Vertical downward and uplift capacities of the secant pile wall were estimated using the methodology which originally was developed by O'Neill and Reese (1999) and presented in FHWA Geotechnical Engineering Circular (GEC) No.8 (2007) For Design and Construction of Continuous Flight Auger Piles.

The drilled piles for the secant pile wall should be designed such that the vertical, horizontal or rotational loads are within the design and operational limits. In addition to the weight of the wall, grade beam and backfill placed above, the vertical loads on the drilled piles should also include the downdrag load from the tiebacks. For vertical compression (downward) loads, the 3-foot diameter piles drilled to minimum depths of 60 feet should have the following minimum allowable end bearing capacities for dead plus live loads:

- Primary Pile Axial Downward Resistance = 30 kip
- Secondary Pile Axial Downward Resistance = 75 kip

For vertical compression (downward) loads, the drilled piles should be designed for an allowable downward skin friction of 500 pounds per square foot (psf) in dense soils for dead plus live loads in addition to the end-bearing capacities indicated above. Both end bearing capacities and skin friction values include a factor of safety of 2 and may be increased by 1/3 to include wind and seismic loads. Uplift resistance may be calculated to be 75 percent of the skin friction in compression. The drilled piles should extend to a depth below the potentially liquefiable zones with zero skin friction in the liquefiable soils and account for liquefaction-induced downdrag force of 20 tons.

3.8.2 Lateral Earth Pressures

Lateral earth pressures on the secant pile wall with tiebacks are based on apparent earth pressure diagrams (trapezoidal or rectangular pressure distribution) using the methods recommended in American Association of State Highways and Transportation Officials (AASHTO) Design Specifications (2012), California Department of Transportation (Caltrans) Memo To Designers (MTD) 5-12 (2012) and Federal Highway Administration (FHWA) Geotechnical Engineering Circular (GEC) No.4 (1999) for design of anchored walls. For design of the secant pile wall, lateral earth pressures were developed using the soil properties presented in Table 12 and seismic parameters presented in Table 7 for both 975- and 72-year return periods. The secant pile wall was designed to resist lateral earth pressures based on the ultimate retaining conditions (when the bluff in front of the wall has resulted loss of soils to a beach level of Elevation +2 feet for short term and +10 feet for long-term). In addition to the lateral earth pressure and hydrostatic water pressure for the static condition, seismic lateral earth pressures, hydrodynamic pressure, and liquefaction-induced pore water pressure were included in the design of the secant pile wall for seismic condition. The additional seismic lateral earth pressure increments were obtained in accordance with the 2014 SFPUC GSR guideline.

According to Section 7 of the 2014 SFPUC GSR, hydrodynamic water pressure was also considered using the method recommended in Ebeling et al. “The Seismic Design of Waterfront Retaining Structures” (1992).

Where the soils behind the secant pile wall liquefy during a major earthquake, the lateral earth pressures exerted on the wall will be momentarily increased due to liquefaction-induced excess pore water pressure. For those soils that will be subjected to liquefaction behind the wall, the liquefaction-induced pore water pressures were calculated and added to the design. The liquefaction-induced pore water pressures and the seismically-induced lateral earth pressures discussed above are two different scenarios that will not occur simultaneously. The secant pile wall design should be checked against both loading scenarios to see which loading scenario is more critical.

When vertical surcharge loads were anticipated within the zone above an imaginary 45-degree line projected up from the long-term exposed bottom of secant pile wall (Elevation -10 feet), the additional lateral earth pressures from the surcharge should be included in the secant pile wall design.

Additionally, we assumed that 3 to 4 feet of soil will deposit above the strengthen zone above the tunnel and behind the proposed secant wall for both short (72-year) and long-term (975-year) conditions, respectively.

Lateral earth pressures for design of the secant pile wall are presented on Plates 8A – 8L. The lateral earth pressures for design are presented in a general case manner for each reach, so that liquefaction-induced lateral pressure and dynamic earth pressure, liquefaction-induced lateral pressure and hydrodynamic force, liquefaction-induced lateral pressure and hydrostatic force should not be taken into account at the same time. Liquefaction-induced lateral pressure is only the excess pore water pressure since the liquefaction-induced lateral deformation only occurs higher than the elevation of the top of retaining wall.

TABLE 12
SOIL PROPERTIES FOR LATERAL EARTH PRESSURES

Reach[1] (STA)	Design Groundwater Elevation (feet)	Layer	Top of Layer Elevation, NAVD88 (feet)	Total Unit Weight (pcf)	Friction Angle (degree)	Cohesion (psf)
North (13+55)	+16	Fill	+31	120	33	0
		Dune Sand	+11	120	34	0
		Colma Formation	+1	125	36	0
		Merced Formation	-19	125	27	300
EQR (22+30)	+16	Fill	+30	120	33	0
		Dune Sand	+17	120	34	0
		Colma Formation	+6	125	36	0
		Merced Formation	-19	125	27	300
Rubble (27+40)	+16	Fill	+28.5	120	33	0
		Dune Sand	+15.5	120	34	0
		Colma Formation	+11.5	125	36	0
		Merced Formation	-6.5	125	27	300
Bluff (35+05)	+18	Fill	+37	120	33	0
		Dune Sand	+27	120	34	0
		Colma Formation	+17	125	36	0
		Merced Formation	-28	125	27	300
South (41+90)	+19	Fill	+56	120	33	0
		Dune Sand	+28	120	34	0
		Colma Formation	+18	125	36	0
		Merced Formation	-4	125	27	300

1) Reaches are presented in Table 1.

According to Section 7 of the 2014 SFPUC GSR, retaining walls should be designed for appropriate static and seismic soil pressure depending on the restraining conditions of the wall. For yielding walls, active soil pressure may be used for design. For non-yielding walls, at-rest pressure may be used for design. Since tieback is considered in the proposed secant retaining wall, apparent earth pressure should be used in the upper portion of the secant retaining wall. The methods recommended in National Cooperative Highway Research Program, Report 611 (2008), American Association of State Highways and Transportation Officials (AASHTO) Design

Specifications (2012), California Department of Transportation (Caltrans) Memo To Designers (MTD) 5-12 (2012) and Federal Highway Administration (FHWA) Geotechnical Engineering Circular (GEC) No.4 (1999) for design of anchored walls were used in developing lateral earth pressure diagrams for different reaches of the proposed retaining wall.

Based on the current concept plans, the tiebacks are being proposed to be installed at an inclination of up to approximately 45 degrees below the horizontal.

3.9 SOIL SPRING RECOMMENDATIONS

Structural analysis of the wall for stability, required toe depth, pile forces, cap forces and soil anchor forces are determined from two dimensional models of sections representative of the five reaches described in Table 12. We understand that the proposed secant wall is being analyzed using the following two approaches as described below.

3.9.1 Limit Equilibrium Analysis

Limit equilibrium is an analysis method where limit state conditions are assumed. For earth retaining structures this usually means that earth pressures are assumed on both the retained and excavated sides. The wall height will be determined by balancing applied loads, active soil pressure, passive soil pressure and tieback loads. In this approach the soil structure interaction is not properly captured. However, it provides moments and forces on the wall. During the screening process to determine critical sections, the designer used limit equilibrium analyses and AGS provide geotechnical input parameters.

3.9.2 Simplified p-y Spring Analyses

After the limit-equilibrium analyses are completed, a simplified p-y spring model was developed. AGS provided p-y data for the models. The p-y data are presented in graphical form in Appendix B. Methodology which originally was developed by O’Neill and Reese (1999) and presented in FHWA Geotechnical Engineering Circular (GEC) No. 8 (2007) was used in developing the soil reaction (p) resisting force per unit length along the pile as a nonlinear function of (y) lateral deflection under static and cyclic loadings.

This model uses p-y springs only applied to the wall below the beach elevation. These springs are defined as compression only springs that only capture the passive response of the soil. Active soil pressures were not modelled using springs, rather an externally applied active soil pressures (as determined from classic earth pressure theories) were applied to the bulkhead wall.

Additional pressures (such as hydrostatic, hydrodynamic, and seismic incremental active pressures) were applied as distributed forces on the wall. The tieback/anchor was modelled as a linear elastic spring with a stiffness equal to the tieback system.

For passive resistance of the drilled piles, only Dune sand, Colma Formation and Merced Formation were considered below beach level. The soil springs for design of the secant pile wall are presented in Appendix B.

3.10 PEDESTRIAN PATH RETAINING WALL

We understand that a retaining wall will be constructed along the proposed pedestrian path between approximately Stations 40+00 and 42+50. The proposed retaining wall may be founded on continuous shallow foundation. We anticipate that the foundation of the proposed pedestrian retaining wall will be on engineered fill. Therefore, we recommend that the footing be embedded at least 24 inches below the finished grade. The retaining wall should either be constructed behind the 3H:1V slope from the secant wall or embedded deep enough to be below the 3H:1V line. The proposed pedestrian path retaining wall should be designed for maximum allowable bearing capacity of 3,000 psf for dead plus live load with minimum 3 feet width. This value may be increased by one third for all loads including wind and seismic.

We estimate that the maximum settlement of the proposed pedestrian retaining wall constructed in accordance with our recommendation to be ¾ inch with maximum deferential settlement of ½ inch along 50 lineal feet. Due to granular nature of the bearing layer, we anticipate that the majority of the settlement will occur during construction stage.

The proposed pedestrian retaining walls should be designed to resist lateral pressures exerted from a material having an equivalent fluid weight as follows:

Active Condition	=	35 pcf for horizontal backslope
At-rest Condition	=	50 pcf
Passive Condition	=	300 pcf
Coefficient of Friction	=	0.30

For a non-horizontal backslope, the active condition for equivalent fluid weight should be increased by 1.5 pcf for each 2 degree rise in slope from the horizontal. For a non-horizontal frontslope, the passive condition for equivalent fluid weight should be decreased by 10 pcf for each 2 degree fall in slope from the horizontal.

Active conditions occur when the top of a retaining wall is free to move outward. At-rest conditions apply when the top of wall is restrained from any movement. It should be noted that the effects of any surcharge and/or compaction loads behind the walls must be accounted for in the design of the walls.

The above criteria are based on fully drained conditions. If drained conditions are not possible, then hydrostatic pressure must be included in the design of the wall. In this case, an additional lateral fluid pressure of 63 pcf must be included

In order to achieve fully-drained conditions, a drainage filter blanket should be placed behind the wall. The blanket should be a minimum of 12 inches thick and should extend the full height of the wall to within 12 inches of the surface. If the excavated area behind the wall exceeds 12 inches, the entire excavated space behind the 12-inch blanket should consist of compacted engineered fill or blanket material. The drainage blanket material may consist of either granular crushed rock and drain pipe fully encapsulated in geotextile filter fabric or Class-II permeable material that meets CalTrans Specification, Section 68, with drainage pipe but without fabric. A 4-inch perforated drain pipe should be installed in the bottom of the drainage blanket and should be underlain by at least 4 inches of filter type material.

As an alternate to the 12-inch drainage blanket, a pre-fabricated strip drain (such as Miradrain) may be used between the wall and retained soil. In this case, the wall must be designed to resist an additional lateral hydrostatic pressure of 30 pcf.

If the pedestrian retaining wall is higher than 6 feet, as measured from the top of the footing, the retaining wall should be designed to resist lateral pressure induced by earthquakes. We recommend that an additional rectangular uniform distribution pressure equal to 24H in psf for level backfill, where H is the height of wall in feet to be considered in this case. Details of the retaining wall and adjacent grades (backslope or front slope) was not available to us during preparation of this report, If the backfill slope is not horizontal, we will provide supplemental recommendations for dynamic earth pressure. The above dynamic earth pressure does not include importance factor.

Piping with adequate gradient shall be provided to discharge water that collects behind the walls to an adequately controlled discharge system away from the structure's foundations.

3.11 BUOYANCY RESISTANCE

Based on our review of the 2015 Ocean Beach Master Plan Coastal Management Framework (CMF), we understand that Jacobs Associates (McMillan, Jacobs Associates, MJA) performed numerical modeling studies to assess the vulnerability of the LMT to bluff retreat and loss of existing overburden. The results of their numerical modeling studies were presented in a report (dated September 23, 2014) incorporated as Appendix 4 of the 2015 CMF. AGS performed numerical modeling for both static and dynamic conditions. The results of our numerical analyses are presented in Appendix C. MJA is performing static and pseudo-static numerical analyses for this project.

3.12 TIEBACKS

3.12.1 Design Criteria

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

Based on the current concept plans, the tiebacks are being proposed to be installed between 35 to 45 degrees from horizontal. It is understood that this relatively steep angle of installation is to meet the required clearance with the LMT and to keep the construction work within the project limits. Tiebacks are typically installed at an inclination between 15 and 30 degrees below the horizontal and inclination up to 45 degrees below the horizontal can generally be installed by most contractors. If possible, consideration should be given to moving the secant pile wall further seaward (perhaps by approximately 5 feet). This will allow easier installation of tiebacks at the more common 45 degrees (or less) to attract more qualified contractors and to increase in tieback efficiencies (with larger horizontal component of tieback load).

For preliminary design purposes, an allowable soil/grout bond strength of 2,700 psf (beyond the active zone defined by a plane sloping up at 60 degrees with the horizontal and from a point H/5 away from the Elevation +2 feet, where H is the long-term exposed height of the wall) may be considered. This preliminary allowable soil/grout bond strength includes a factor of safety of 2. This value may be increased after performing a pilot test program. It should be noted that the bond strength of tiebacks will depend on the construction method used by the contractors. The project specifications should allow for modification of the bond strength based on values that are demonstrated from field verification testing.

The potential impact of high grouting pressure in close proximity to the LMT should be considered. The following are some options that may be utilized to address this concern:

- moving the secant pile wall further away from the LMT (if possible);
- extending the length of the unbonded zone of the anchor to a point beyond the potential impact zone of the LMT; and
- limiting the grout pressure at a safe level where it is closest to the LMT.

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

3.12.2 Testing and Acceptance Criteria

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

3.12.3 Tieback-induced Downdrag Forces

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

3.13 CONTROLLED LOW STRENGTH MATERIAL (CLSM)

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

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

3.14 EARTHWORK

3.14.1 Site Preparation

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

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

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

3.14.2 Excavation Characteristics

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

3.14.3 Unshored Excavations

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

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

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

3.14.4 Fills and Backfills

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

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

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

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

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

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

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

3.15 DEWATERING AND GROUNDWATER CONSIDERATIONS DURING CONSTRUCTION

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

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

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

Water collected during dewatering should be tested for contamination prior to its disposal. Because the potential for contamination of groundwater was not explored in this study, recommendations are not given herein for proper disposal of collected water.

3.16 FLEXIBLE PAVEMENT

For the new SFPUC access road, the new asphalt concrete pavement should be designed based on the Caltrans Flexible Pavement Design Method with an assumed R-Value of 15 and Traffic Index (TI) as determined by the project civil engineer. Table 13 presents the Caltrans recommended sections.

TABLE 13

GEOTECHNICAL RECOMMENDATIONS FOR FLEXIBLE PAVEMENT SECTIONS

TI	AC Thickness (in)	AB Thickness (in)
4.5	2.5	8
5	3	8
5.5	3.5	9
6	3.5	10.5
6.5	4	11.5
7	4	13

Notes:
Caltrans Highway Design Manual, Chapter 630 (2020):
AC = Gravel Equivalent for Pavement Section;
AB = Aggregate Base (Min.R-Value = 78);

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

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

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

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

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

4.0 CLOSURE

This report has been prepared in accordance with generally accepted professional geotechnical engineering practice for the exclusive use of SFPUC for the proposed South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection Project in San Francisco, California. No other warranty, express or implied, is made.

The geotechnical findings and recommendations presented in this report are based on the data obtained from the borings and CPTs performed for this study, and other geotechnical information previously obtained by others in the project area. The nature and extent of variations between the borings and CPTs may not become evident until construction. In the event variations appear, it may be necessary to reevaluate the findings and recommendations presented herein.

The information in this report is primarily intended for use by design engineers. It is the responsibility of the owner or its representative to ensure that the applicable provisions contained herein are incorporated into the plans and specifications and that the necessary steps are taken to see that the contractor carry out such provisions in the field.

The use of this report or its contents requires prior consent of AGS. In addition, the use of any information contained in this report for purposes other than those expressly stated is at the user's own risk.

Respectfully submitted,

AGS, Inc.

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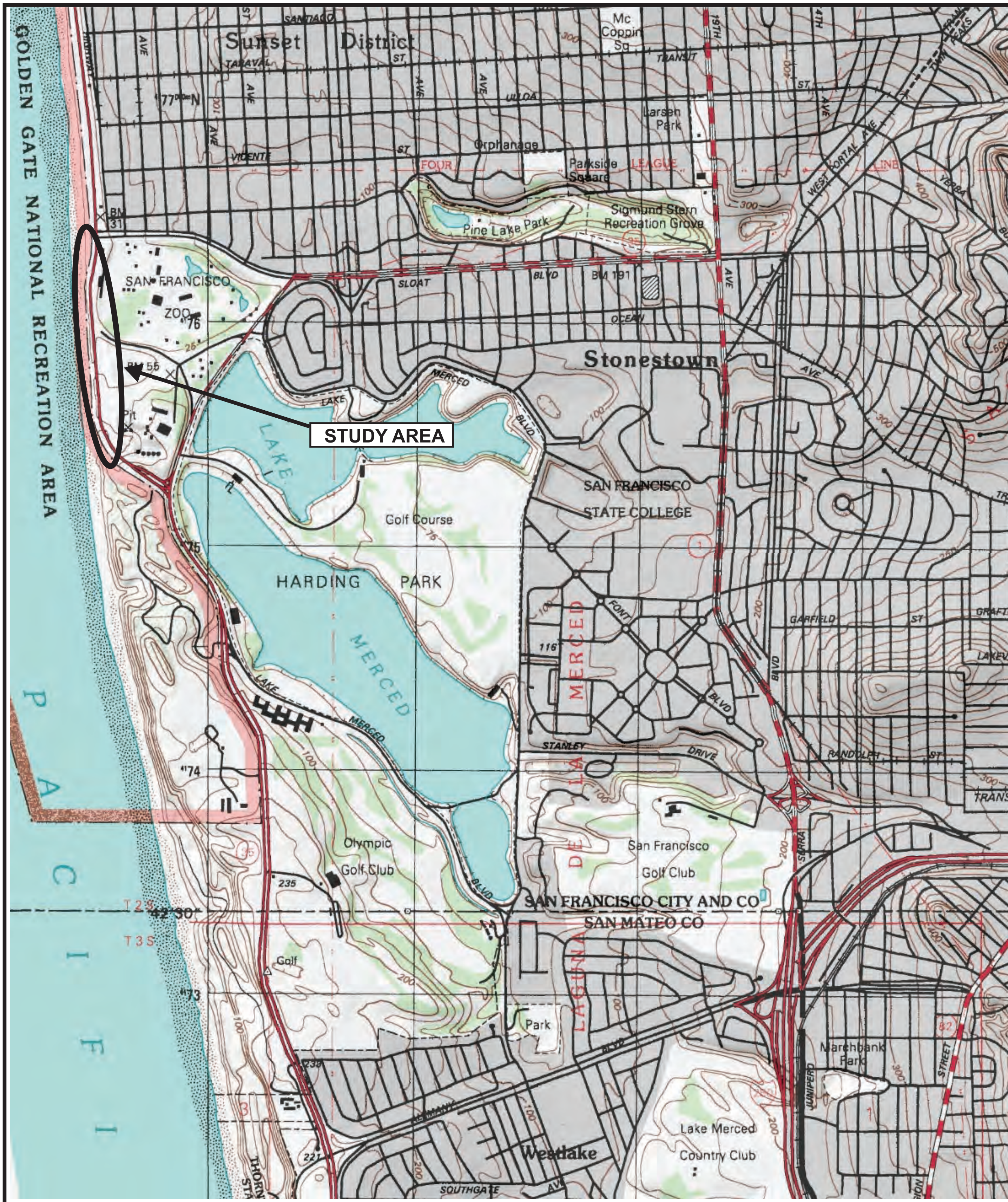
PLATES



Source: *USGS, 7.5 Minute Topographic Map of the San Francisco South (2018) Quadrangle.*

N

1000 0 1000 2000 ft

Scale

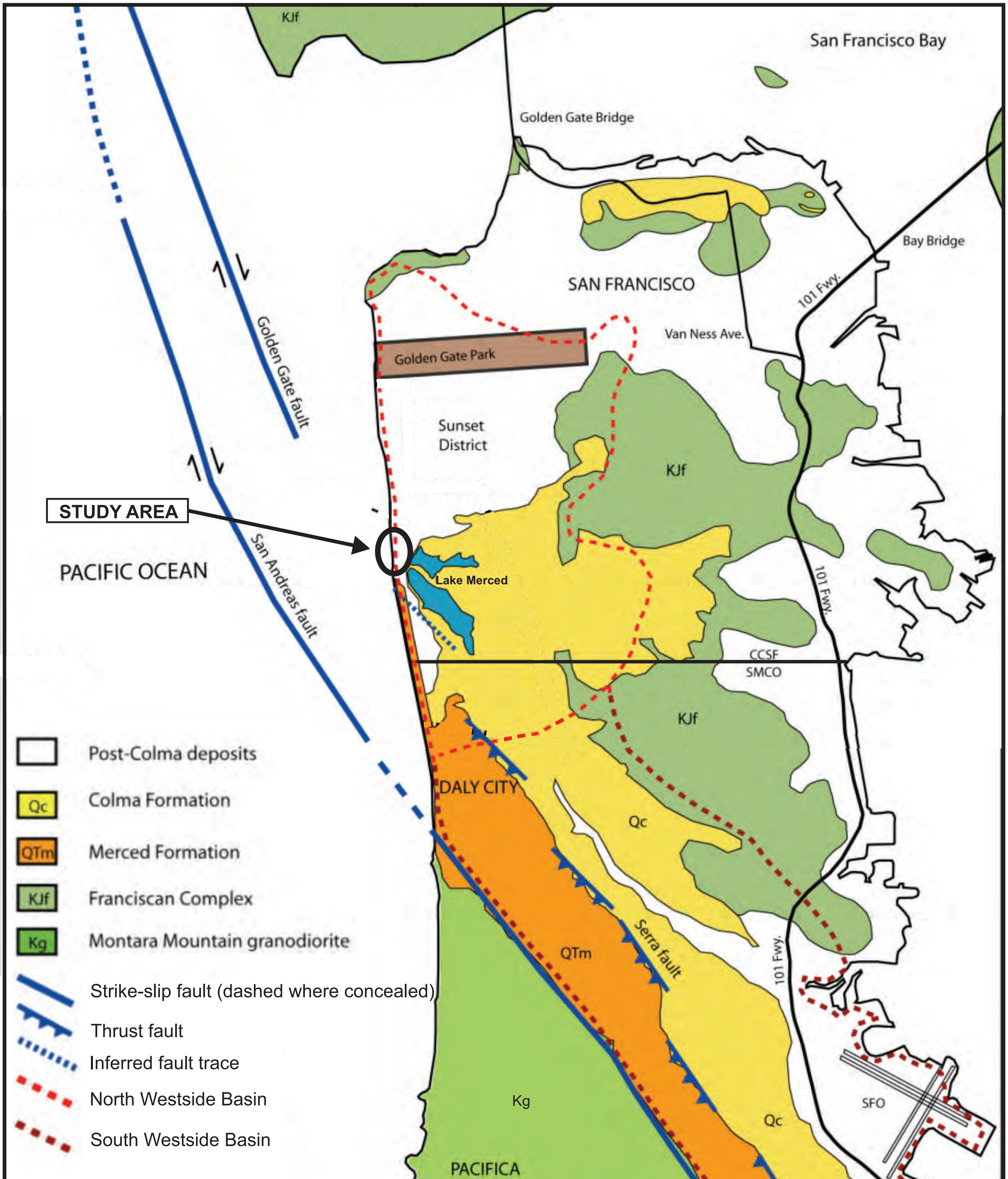
SITE VICINITY MAP

SOUTH OCEAN BEACH COASTAL EROSION AND WW INFRASTRUCTURE PROTECTION PROJECT SAN FRANCISCO, CALIFORNIA

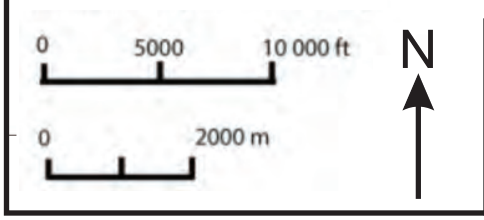
AGS

JOB NO. AGS-18-003	DATE: JULY 2019	PLATE 1

Exhibit 8
2-21-0912
Page 63 of 367



Source: McGuire (2009), Adapted from Clifton and Hunter (1999)



REGIONAL GEOLOGY MAP

SOUTH OCEAN BEACH COASTAL EROSION AND WW INFRASTRUCTURE PROTECTION PROJECT SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: JULY 2019	PLATE 3A

Exhibit 8
2-21-0912
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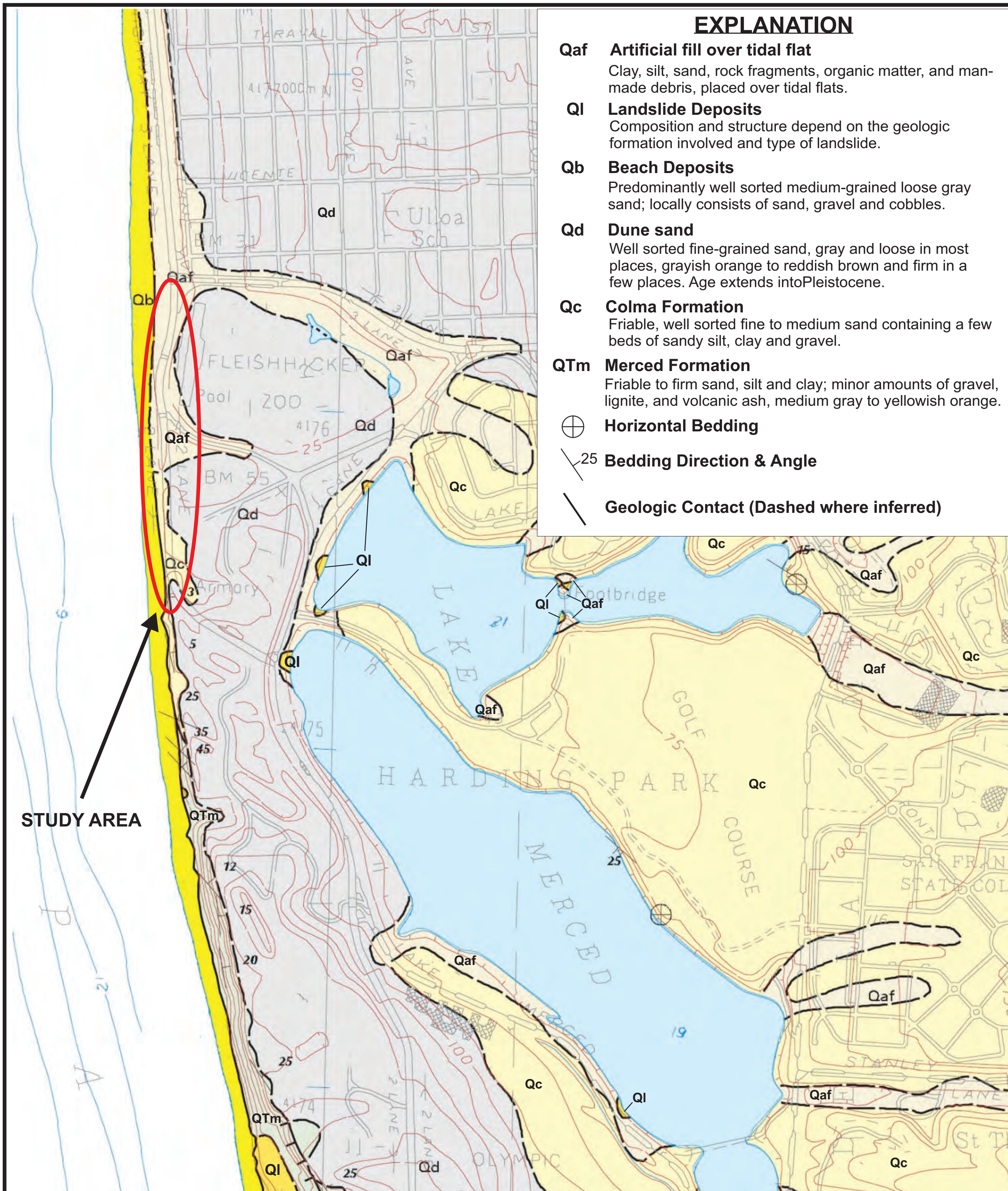


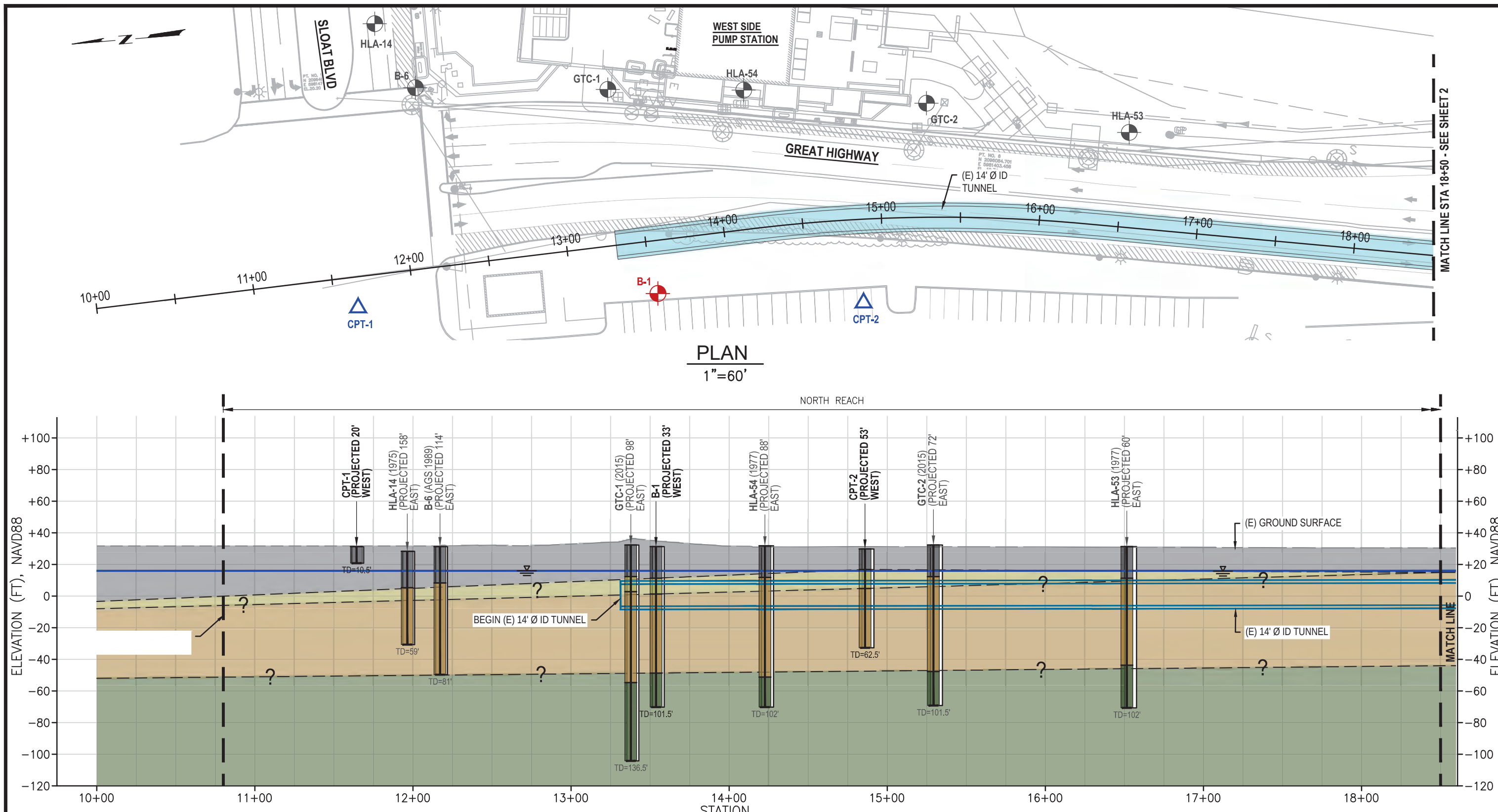
Source: Bonilla, 1998, USGS Preliminary Geologic Map of the San Francisco South 7.5' Quadrangle

LOCAL GEOLOGY MAP

SOUTH OCEAN BEACH COASTAL EROSION AND WW INFRASTRUCTURE PROTECTION PROJECT SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: JULY 2019	PLATE 3B



LEGEND

- ARTIFICIAL FILL
- DUNE SAND
- COLMA FORMATION
- MERCED FORMATION
- LMT ALIGNMENT
- ESTIMATED DESIGN GROUNDWATER
- B-1 TD=TOTAL DEPTH (FT)
- 0+00 LMT ALIGNMENT
- SOIL BORING (AGS 2019)
- CONE PENETROMETER TEST SOUNDING (AGS 2019)
- PREVIOUS SOIL BORINGS
- POTHOLES (AGS 2019)

PROFILE

HOR.: 1"=60'
VERT.: 1"=60'

NOTES

1. SFCD = NAV88−11.35 FEET.
2. ORIGINAL PLOT SIZE OF THIS SHEET IS 11"X17".

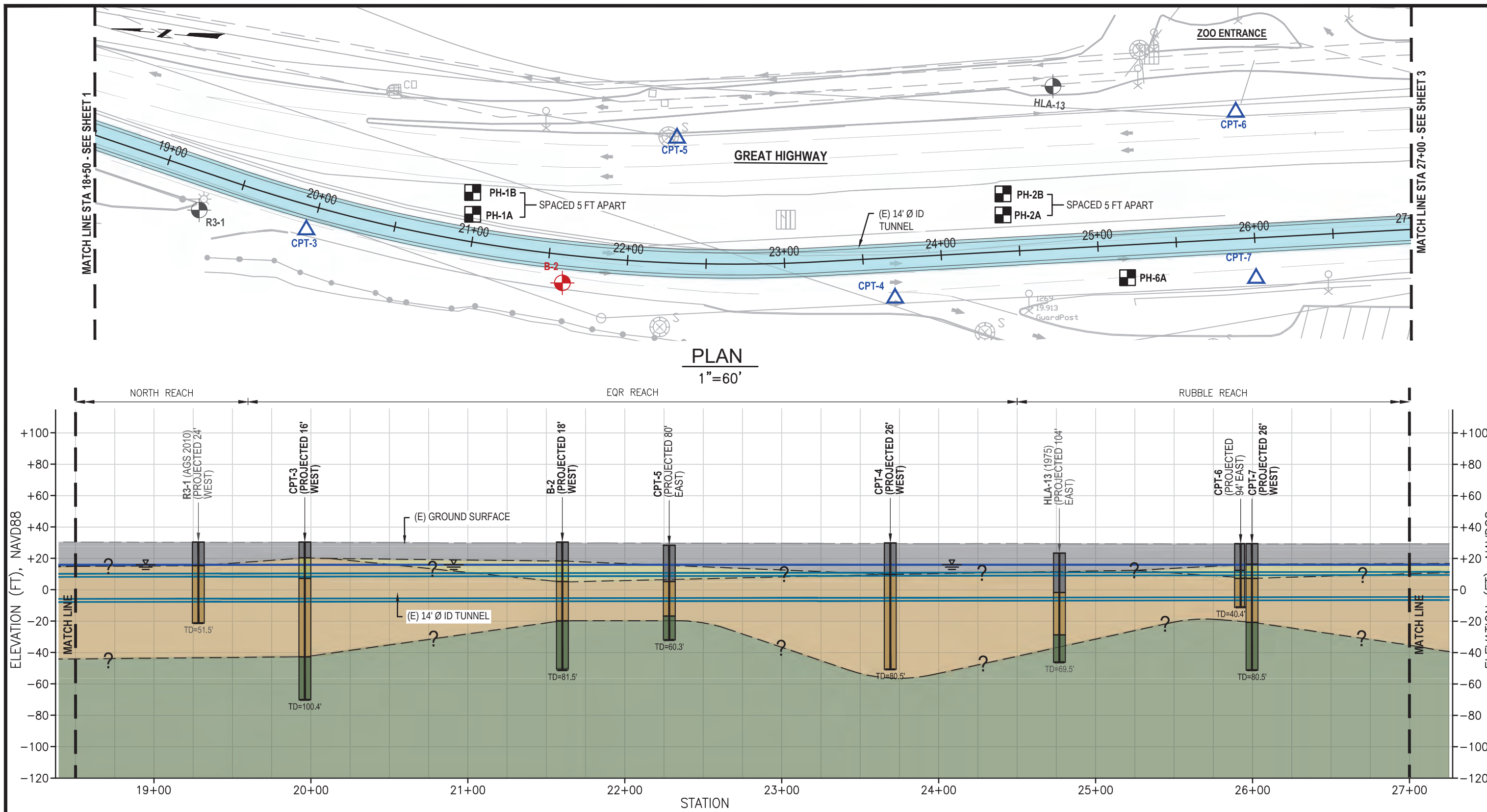
DISCLAMER: THE SUBSURFACE CONDITIONS WERE INTERPOLATED. THE INTERPOLATION AND EXTRAPOLATION OF THE SUBSURFACE SOIL AND GROUNDWATER CONDITIONS ARE BASED ON WIDELY SPACED BORINGS. VARIATION IN THE ACTUAL CONDITIONS FROM THOSE DEPICTED SHOULD BE ANTICIPATED. NEITHER BORING LOGS NOR THE GENERALIZED SOIL PROFILES ARE STAND-ALONE DOCUMENTS, AND THEY SHOULD ONLY BE USED WITHIN THE FULL CONTEXT OF THE SOILS REPORT. THIS PROFILE WAS PREPARED FOR THE SOLE PURPOSE OF DEVELOPING GEOTECHNICAL RECOMMENDATIONS. IT IS NOT INTENDED FOR USE IN ESTIMATING QUANTITY OF VARIOUS SOIL TYPES.

GENERALIZED PLAN AND PROFILE ALONG LMT ALIGNMENT

SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO: AGS-18-003	DATE: JUL 2019	PLATE 4A



LEGEND

- ARTIFICIAL FILL
- DUNE SAND
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- MERCED FORMATION
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PROFILE

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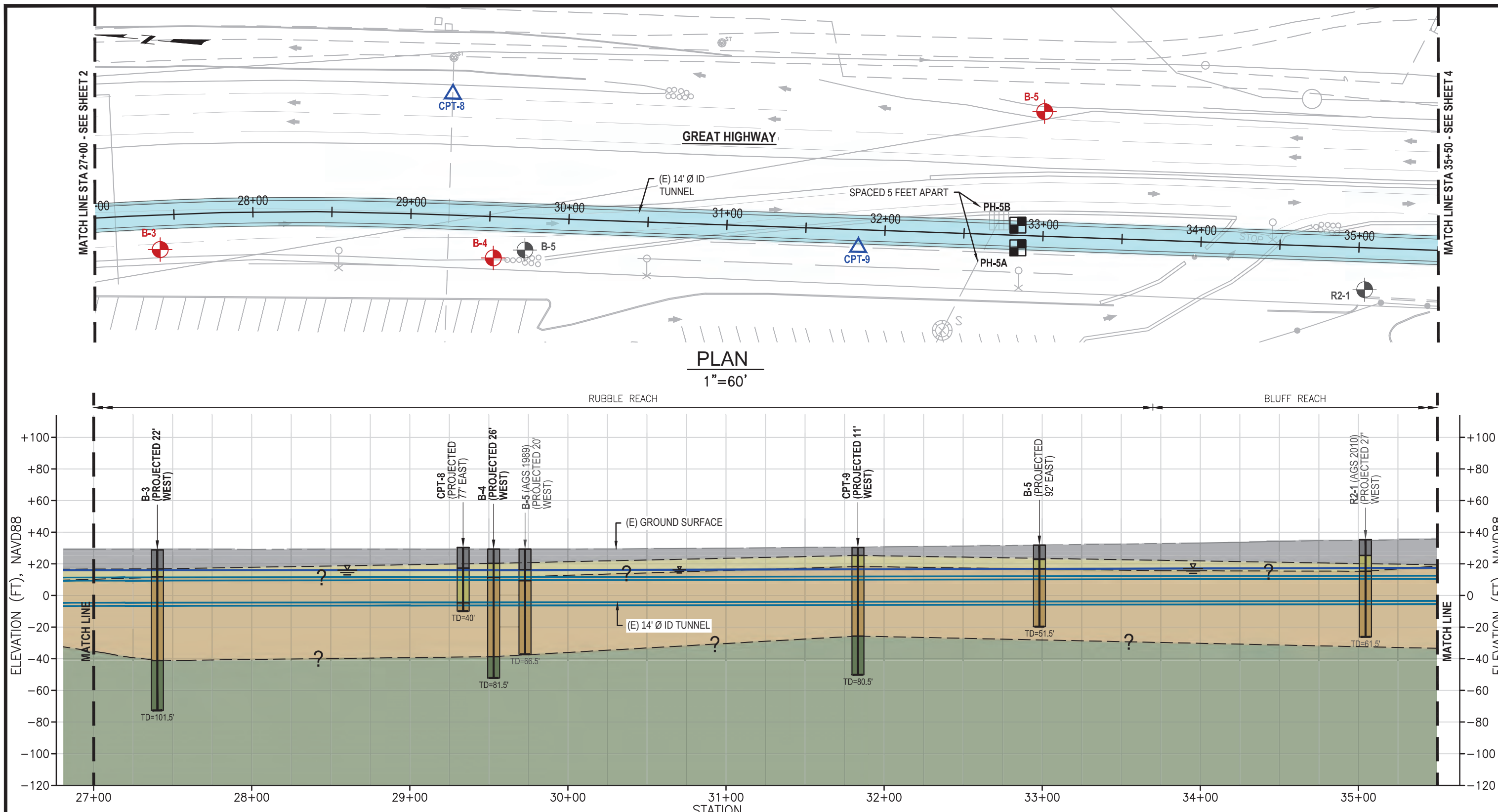
GENERALIZED PLAN AND PROFILE ALONG LMT ALIGNMENT

SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO: AGS-18-003	DATE: JUL 2019	PLATE 4B



LEGEND

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- MERCED FORMATION
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- SOIL BORING (AGS 2019)
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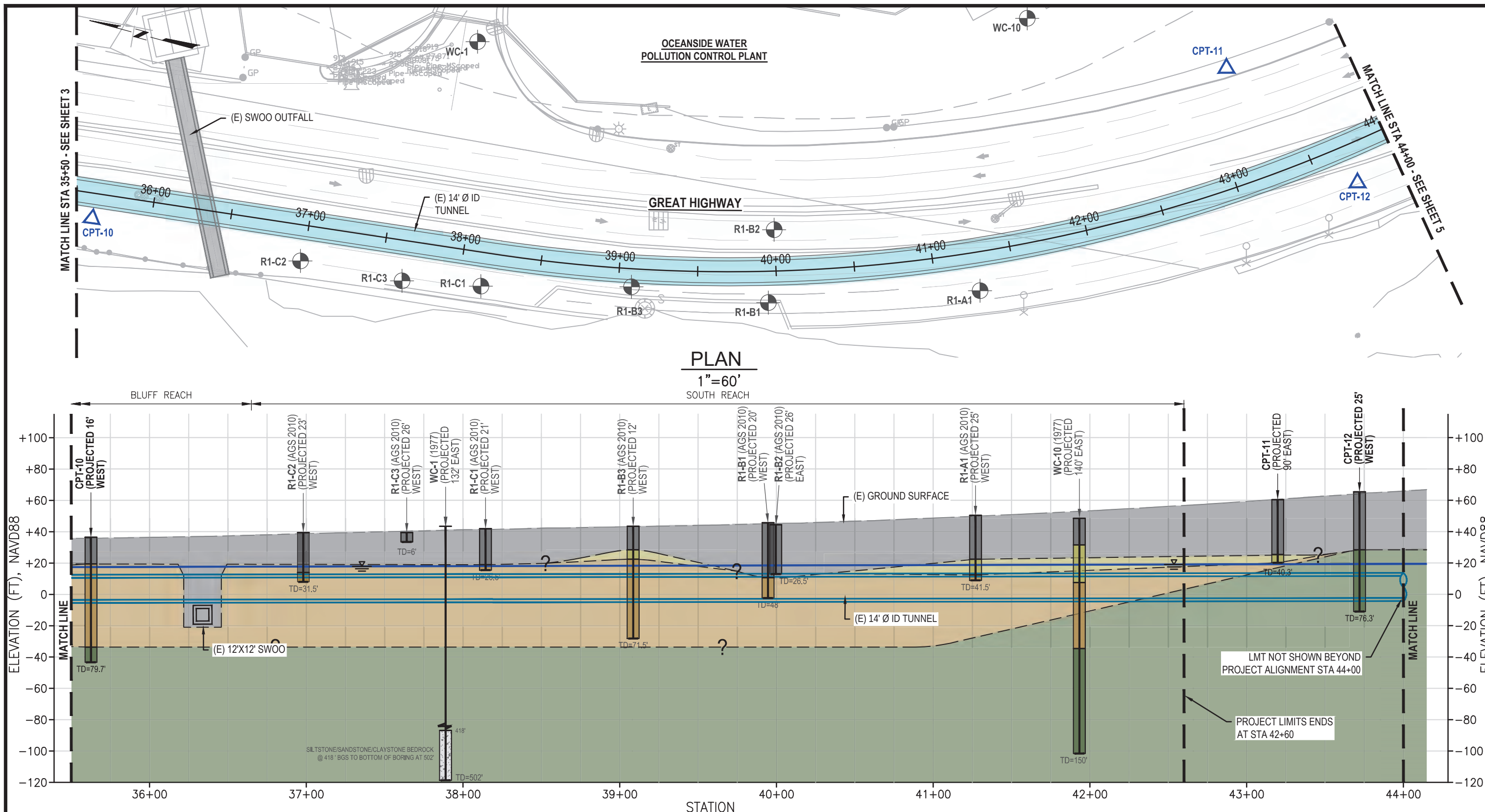
GENERALIZED PLAN AND PROFILE ALONG LMT ALIGNMENT SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT SAN FRANCISCO, CALIFORNIA		AGS
JOB NO: AGS-18-003	DATE: JUL 2019	PLATE 4C



LEGEND

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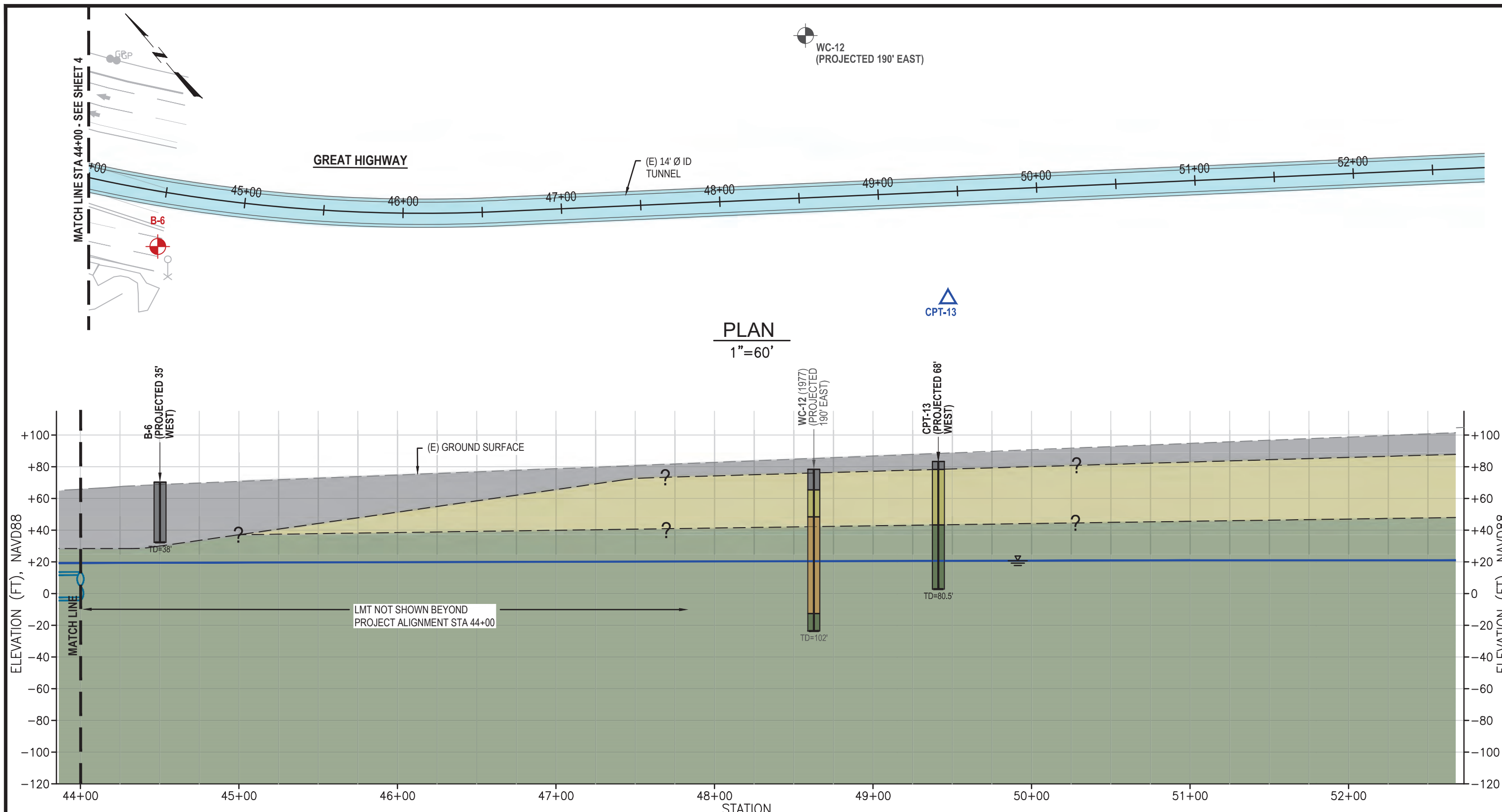
GENERALIZED PLAN AND PROFILE ALONG LMT ALIGNMENT

SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO: AGS-18-003	DATE: JUL 2019	PLATE 4D



LEGEND

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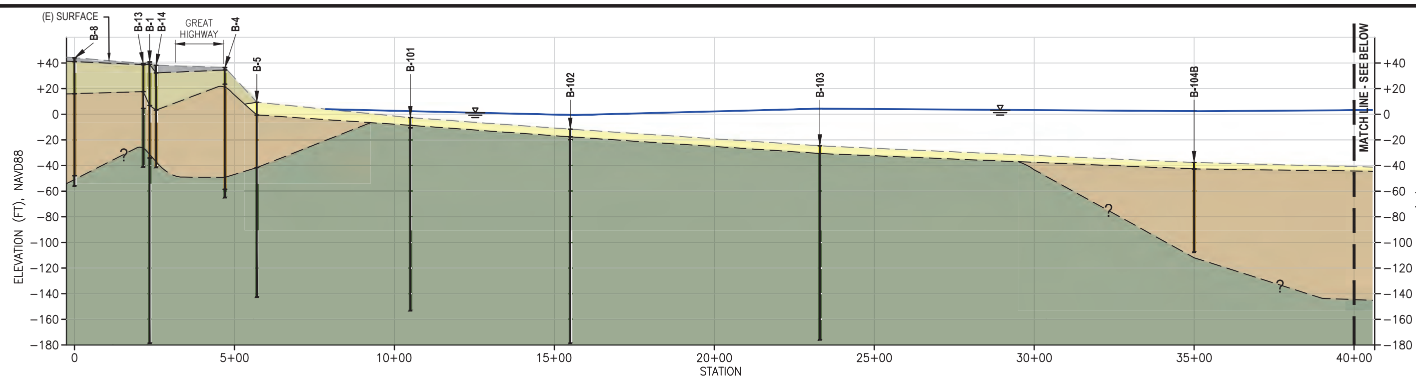
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT
SAN FRANCISCO, CALIFORNIA

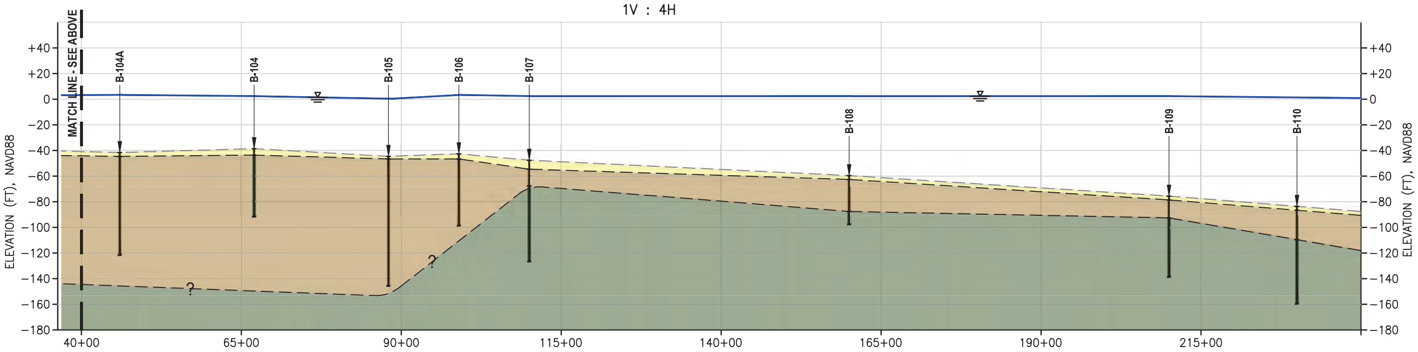
AGS

JOB NO: AGS-18-003	DATE: JUL 2019	PLATE 4E



PROFILE

HOR.: 1"=300'
VERT.: 1"=75'
1V : 4H



PROFILE

HOR.: 1"=1500'
VERT.: 1"=75'
1V : 20H

LEGEND

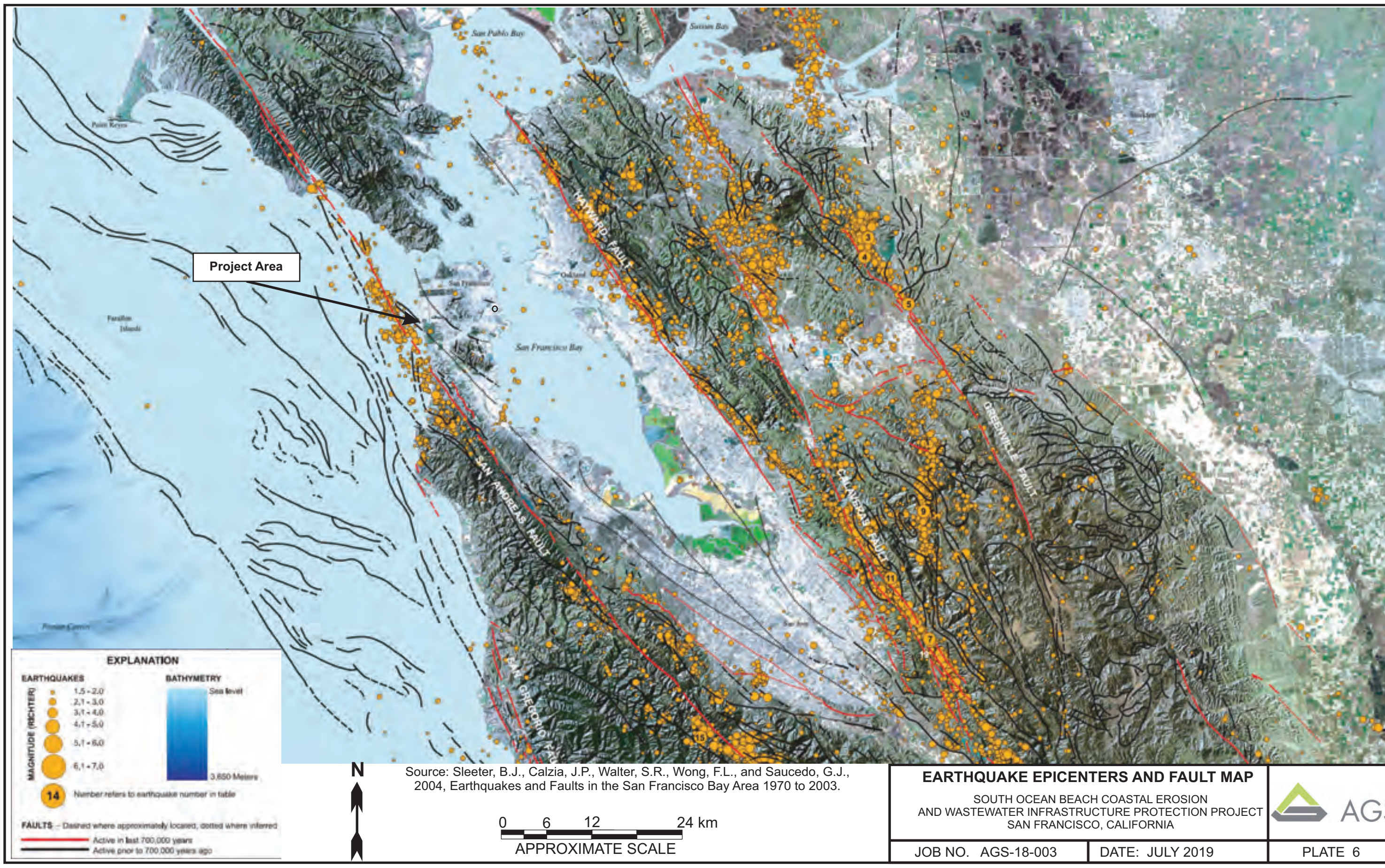
- ARTIFICIAL FILL
- DUNE SAND
- COLMA FORMATION
- MERCED FORMATION
- LMT ALIGNMENT
- ESTIMATED DESIGN GROUNDWATER
- B-1 TD=TOTAL DEPTH (FT)

DISCLAMER: THE SUBSURFACE CONDITIONS WERE INTERPOLATED. THE INTERPOLATION AND EXTRAPOLATION OF THE SUBSURFACE SOIL AND GROUNDWATER CONDITIONS ARE BASED ON WIDELY SPACED BORINGS. VARIATION IN THE ACTUAL CONDITIONS FROM THOSE DEPICTED SHOULD BE ANTICIPATED. NEITHER BORING LOGS NOR THE GENERALIZED SOIL PROFILES ARE STAND-ALONE DOCUMENTS, AND THEY SHOULD ONLY BE USED WITHIN THE FULL CONTEXT OF THE SOILS REPORT. THIS PROFILE WAS PREPARED FOR THE SOLE PURPOSE OF DEVELOPING GEOTECHNICAL RECOMMENDATIONS. IT IS NOT INTENDED FOR USE IN ESTIMATING QUANTITY OF VARIOUS SOIL TYPES.

NOTES

1. NAV88 = SFCD+11.35 FEET
2. PROFILE STATIONING BEGINS AT B-8.
3. SUBSURFACE SHOWN IS BASED ON WOODWARD AND CLYDE BORINGS PRIOR TO CONSTRUCTION OF SWOO.

GENERALIZED SUBSURFACE PROFILE - SWOO SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT SAN FRANCISCO, CALIFORNIA		AGS
JOB NO: AGS-18-003	DATE: JUL 2019	PLATE 5



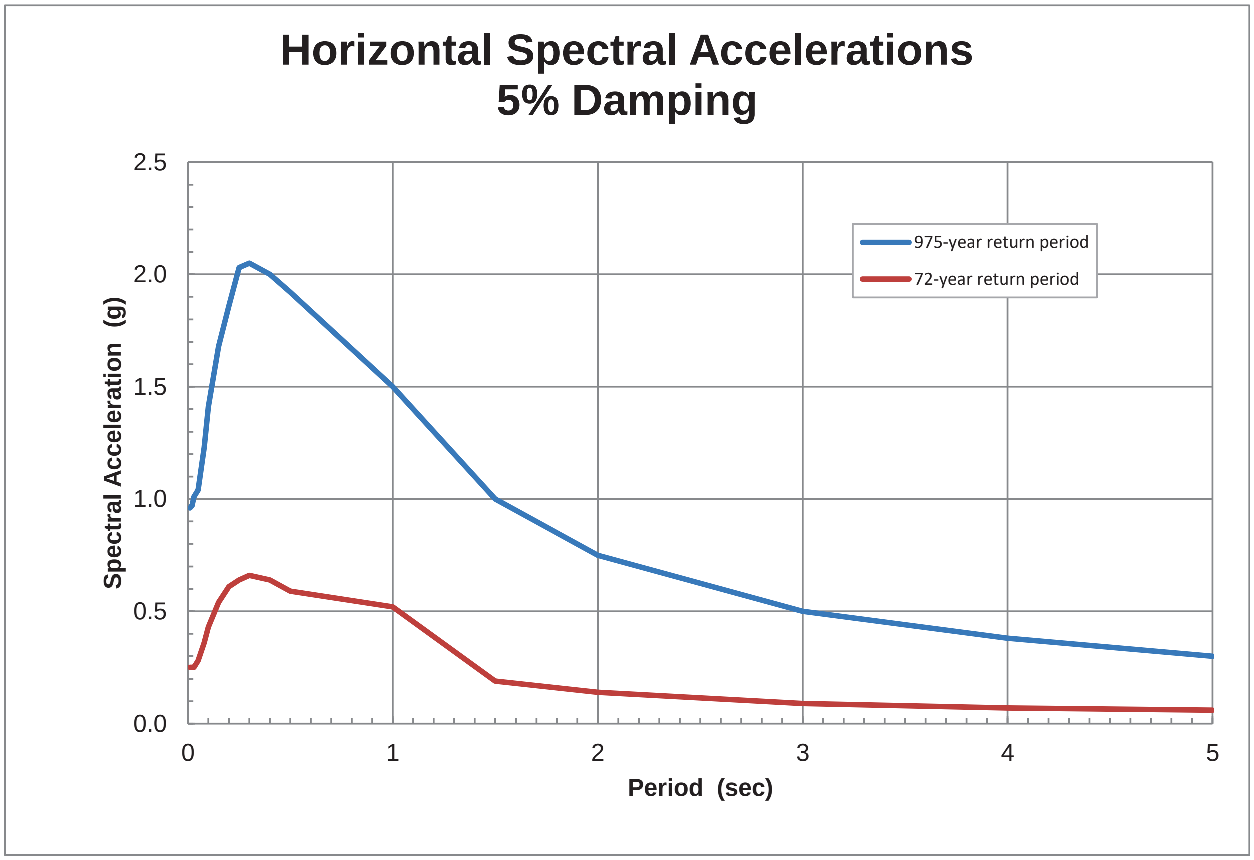
Source: Sleeter, B.J., Calzia, J.P., Walter, S.R., Wong, F.L., and Saucedo, G.J., 2004, Earthquakes and Faults in the San Francisco Bay Area 1970 to 2003.

EARTHQUAKE EPICENTERS AND FAULT MAP

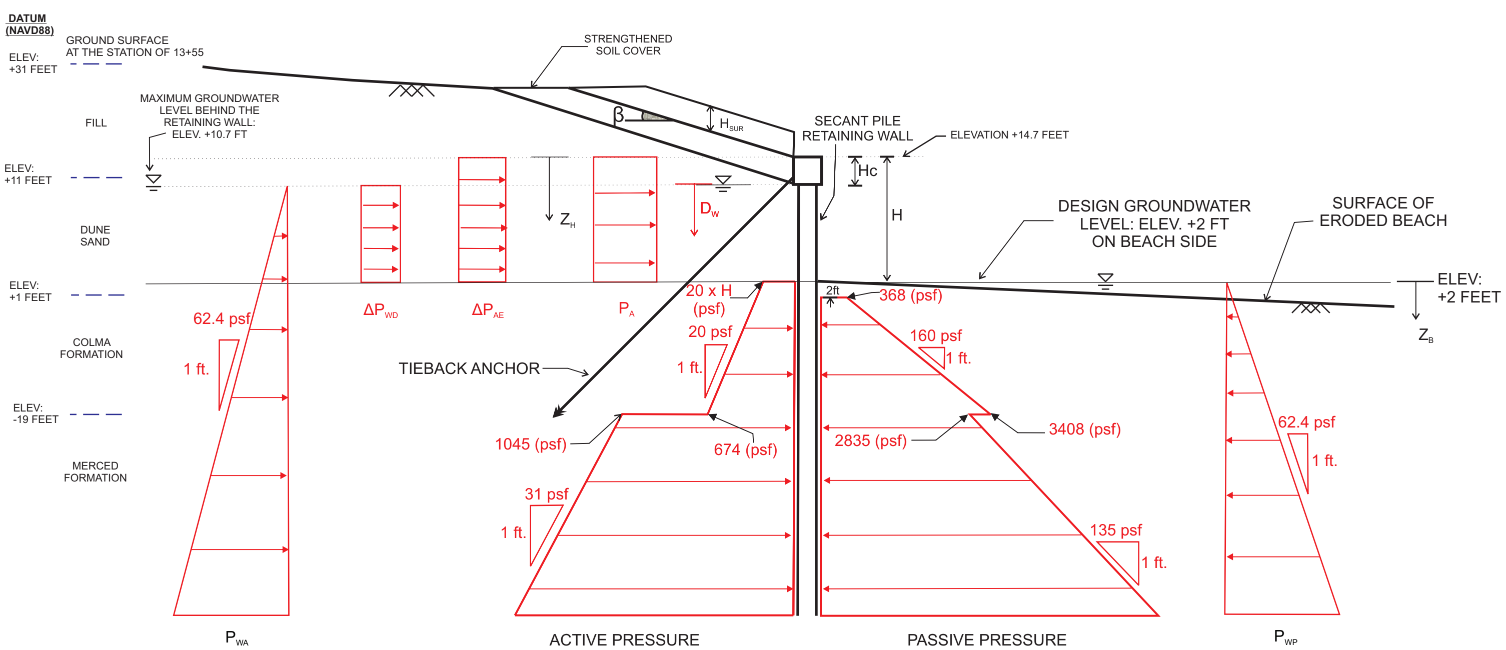
SOUTH OCEAN BEACH COASTAL EROSION
AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: JULY 2019	PLATE 6



AGS	**DESIGN ACCELERATION SPECTRUM** OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA	PLATE NO. 7
	Project No.: AGS-18-003 | Date: July 2021	



NOTES:

β = BACKSLOPE ANGLE = 3H:1V

H = WALL HEIGHT FROM ELEVATION +2 FEET TO TOP OF WALL (FEET)

Hc= HEIGHT OF PILE CAP (FEET)

Z_B = DEPTH BELOW BEACH LEVEL, IN ERODED CONDITION (FEET)

Z_H = DEPTH BELOW TOP OF WALL (FEET)

H_{SUR} = HEIGHT OF SAND LAYER SURCHARGE (FEET)

P_A = APPARENT EARTH PRESSURE = 35 x H_{SUR} + 18 x Hc + 17 x H (PSF)

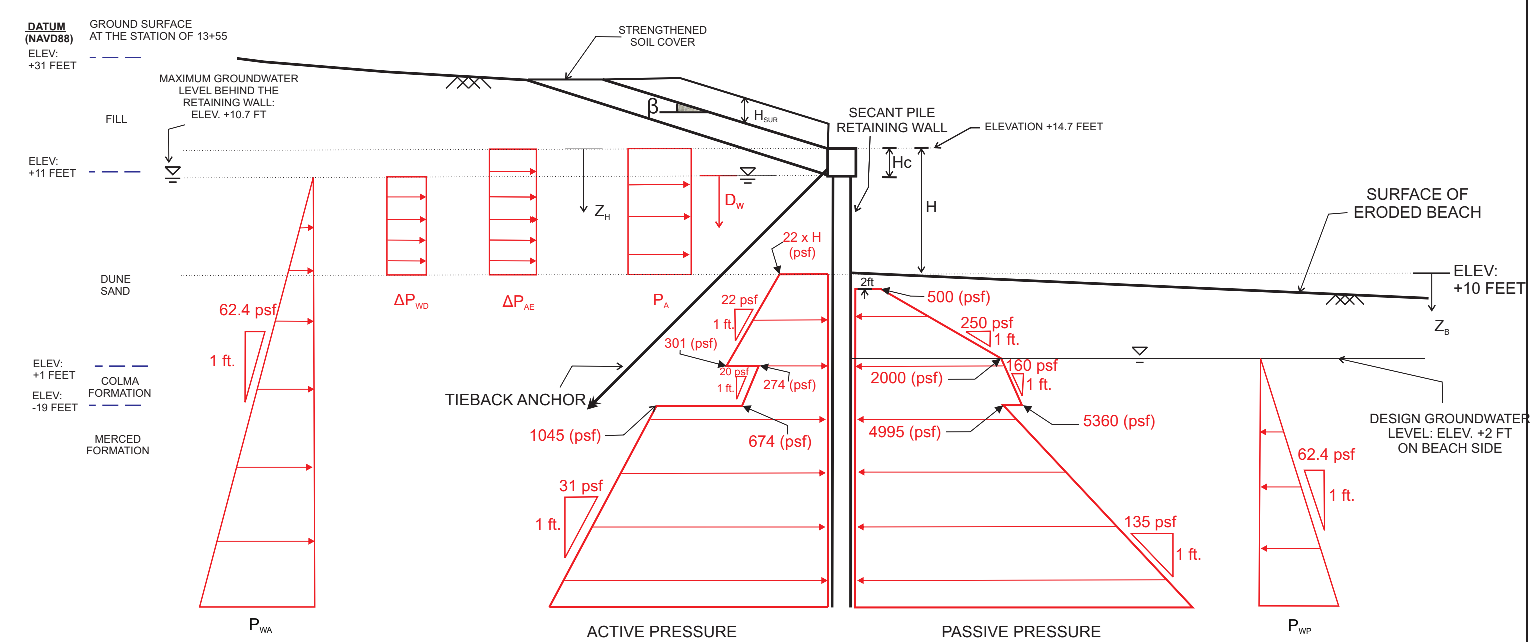
P_{WA} and P_{WP} = HYDROSTATIC PRESSURE (PSF)

ΔP_{AE} = SEISMIC EARTH PRESSURE = 10 x (H+½H_{SUR}) (PSF)

ΔP_{WD} = HYDRODYNAMIC WATER PRESSURE = 6 x (H-Hc) (PSF)

NOT TO SCALE

LATERAL EARTH PRESSURES FOR SECANT PILE WALL NORTH REACH Sta. 13+55 - BEACH ELEVATION +2 FEET SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JULY 2021	PLATE 8A



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P_{WA} and P_{WP} = HYDROSTATIC PRESSURE (PSF)

ΔP_{AE} = SEISMIC EARTH PRESSURE = 38 x (H+½H_{SUR}) (PSF)

ΔP_{WD} = HYDRODYNAMIC WATER PRESSURE = 23 x (H-Hc) (PSF)

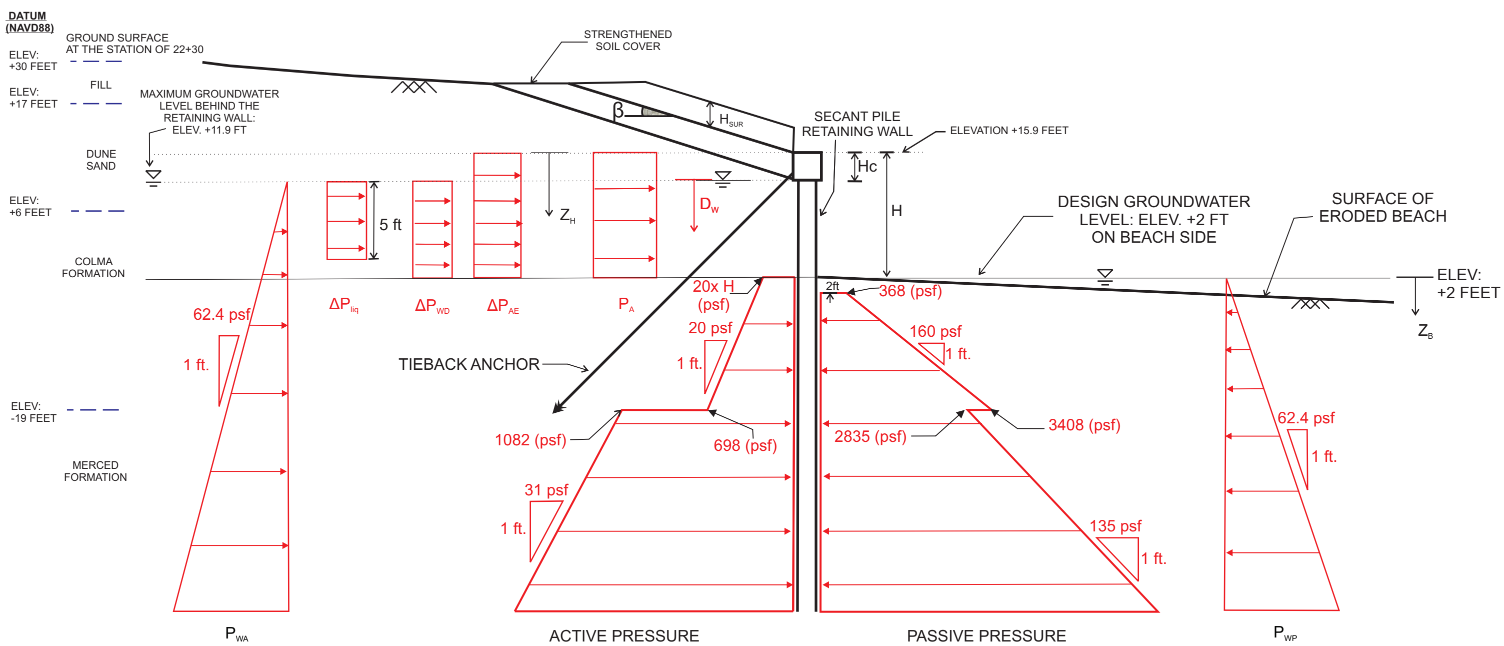
NOT TO SCALE

LATERAL EARTH PRESSURES FOR SECANT PILE WALL
NORTH REACH Sta. 13+55 - BEACH ELEVATION +10 FEET

SOUTH OCEAN BEACH COASTAL EROSION AND
WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: JULY 2021	PLATE 8B



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Hc= HEIGHT OF PILE CAP (FEET)

Z_B = DEPTH BELOW BEACH LEVEL, IN ERODED CONDITION (FEET)

Z_H = DEPTH BELOW TOP OF WALL (FEET)

H_{SUR} = HEIGHT OF SAND LAYER SURCHARGE (FEET)

P_A = APPARENT EARTH PRESSURE = 35 x H_{SUR}+ 18 x Hc + 17 x H (PSF)

P_{WA} and P_{WP} = HYDROSTATIC PRESSURE (PSF)

ΔP_{liq} = LIQUEFACTION INDUCED LATERAL PRESSURE (INCLUDES HYDROSTATIC PRESSURE) = 1100 x (Hc+H_{SUR}) (PSF)

ΔP_{AE} = SEISMIC EARTH PRESSURE = 10 x (H+½H_{SUR}) (PSF)

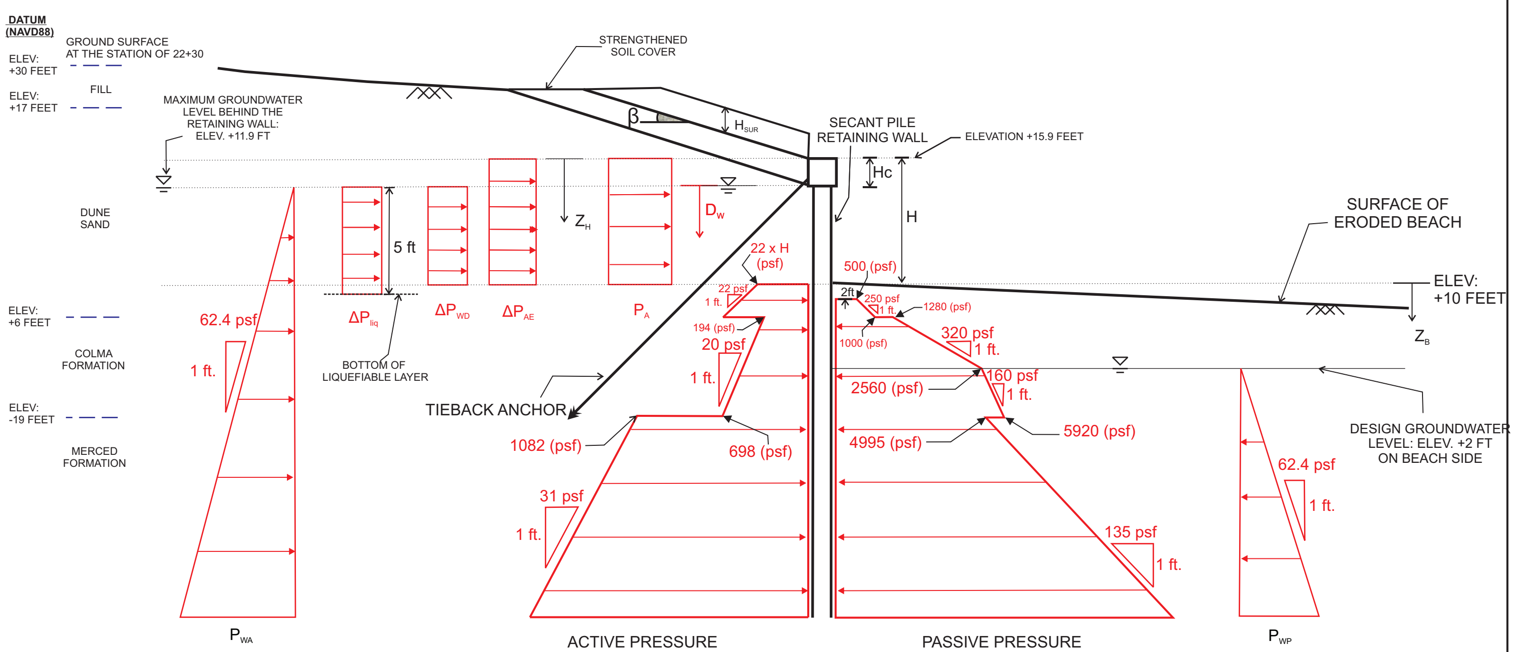
ΔP_{WD} = HYDRODYNAMIC WATER PRESSURE = 6 x (H-Hc) (PSF)

NOT TO SCALE

LATERAL EARTH PRESSURES FOR SECANT PILE WALL
EQR REACH Sta. 22+30 - BEACH ELEVATION +2 FEET
SOUTH OCEAN BEACH COASTAL EROSION AND
WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: JULY 2021	PLATE 8C



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P_A = APPARENT EARTH PRESSURE = 35 x H_{SUR} + 18 x Hc + 17 x H (PSF)

P_{WA} and P_{WP} = HYDROSTATIC PRESSURE (PSF)

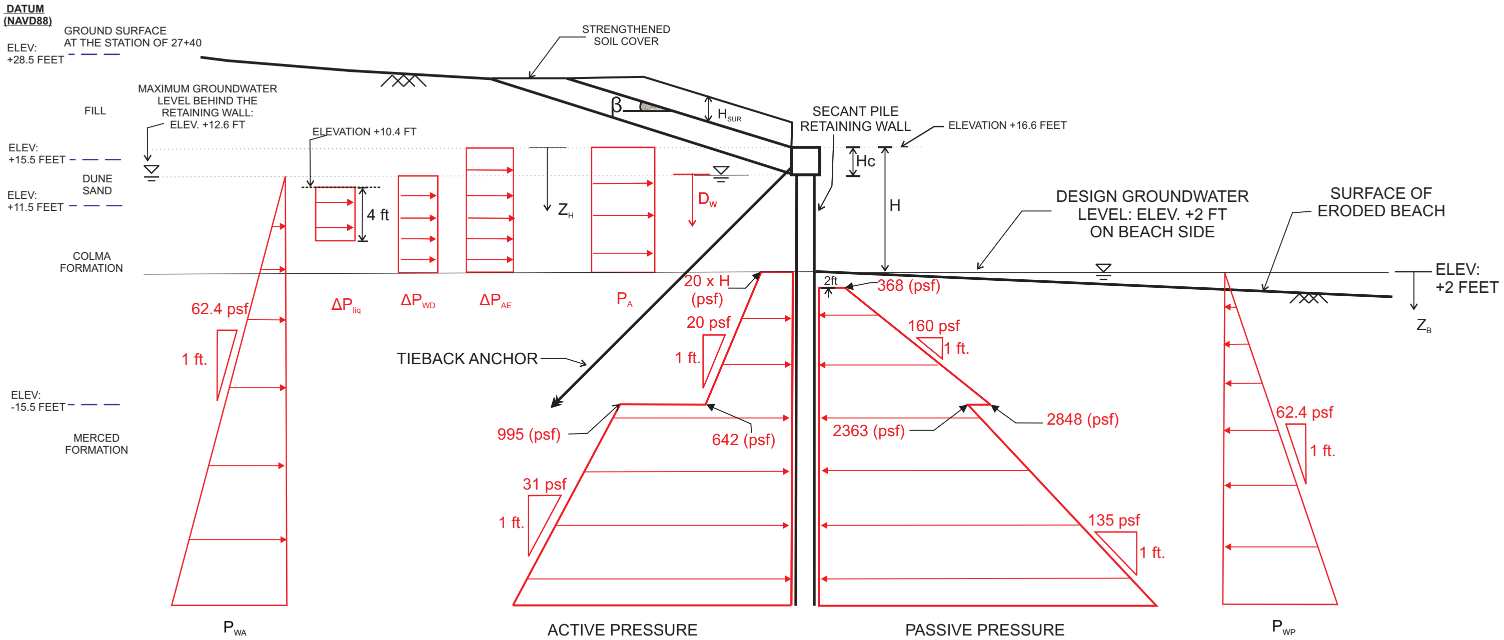
ΔP_{liq} = LIQUEFACTION-INDUCED LATERAL PRESSURE (INCLUDES HYDROSTATIC PRESSURE) = 110 x ($Hc+H_{SUR}$) (PSF)

ΔP_{AE} = SEISMIC EARTH PRESSURE = 38 x ($H+½H_{SUR}$) (PSF)

ΔP_{WD} = HYDRODYNAMIC WATER PRESSURE = 23 x (H-Hc) (PSF)

NOT TO SCALE

LATERAL EARTH PRESSURES FOR SECANT PILE WALL EQR REACH Sta. 22+30 - BEACH ELEVATION +10 FEET SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JULY 2021	PLATE 8D



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P_A = APPARENT EARTH PRESSURE = 35 x H_{SUR}+ 18 x Hc + 17 x H (PSF)

P_{WA} and P_{WP} = HYDROSTATIC PRESSURE (PSF)

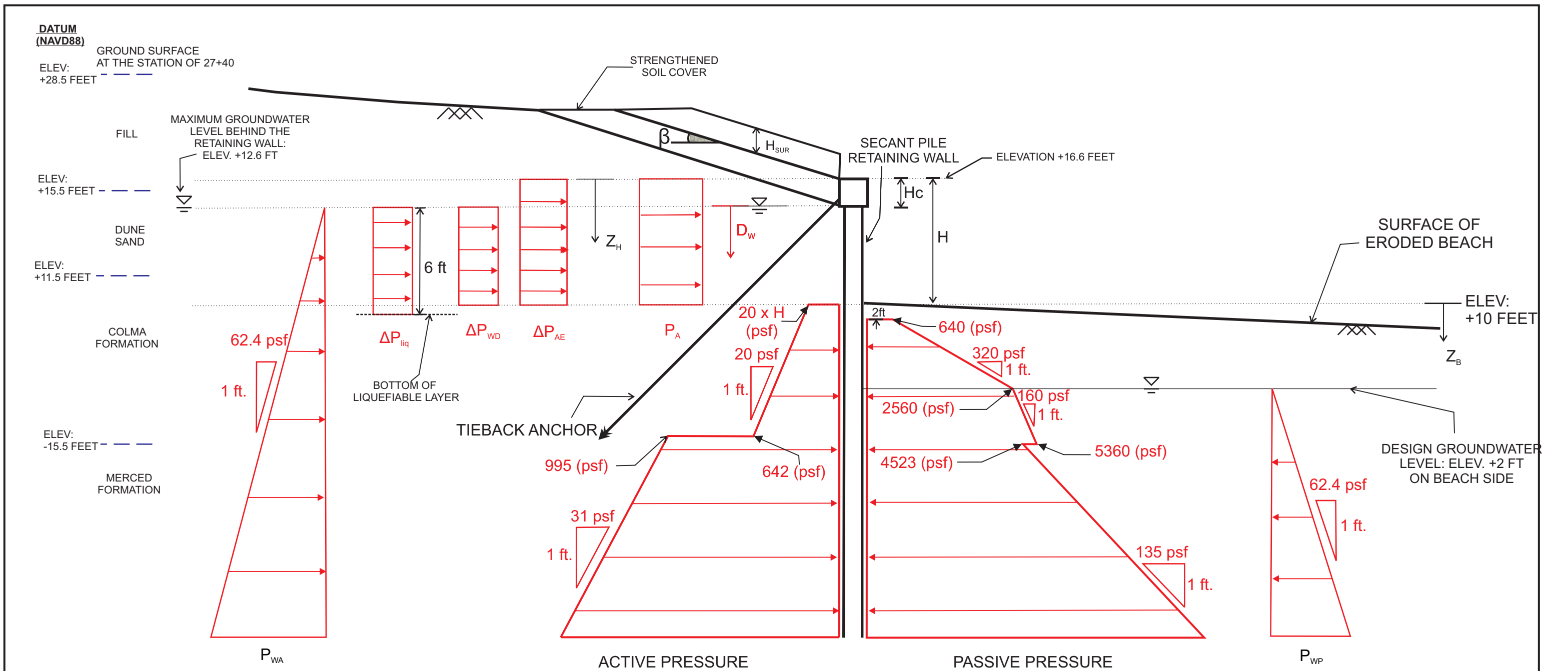
ΔP_{liq} = LIQUEFACTION-INDUCED LATERAL PRESSURE (INCLUDES HYDROSTATIC PRESSURE) = 110 x (Hc+H_{SUR}) (PSF)

ΔP_{AE} = SEISMIC EARTH PRESSURE = 10 x (H+½H_{SUR}) (PSF)

ΔP_{WD} = HYDRODYNAMIC WATER PRESSURE = 6 x (H-Hc) (PSF)

NOT TO SCALE

LATERAL EARTH PRESSURES FOR SECANT PILE WALL RUBBLE REACH Sta. 27+40 - BEACH ELEVATION +2 FEET SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JULY 2021	PLATE 8E



NOTES:

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P_{WA} and P_{WP} = HYDROSTATIC PRESSURE (PSF)

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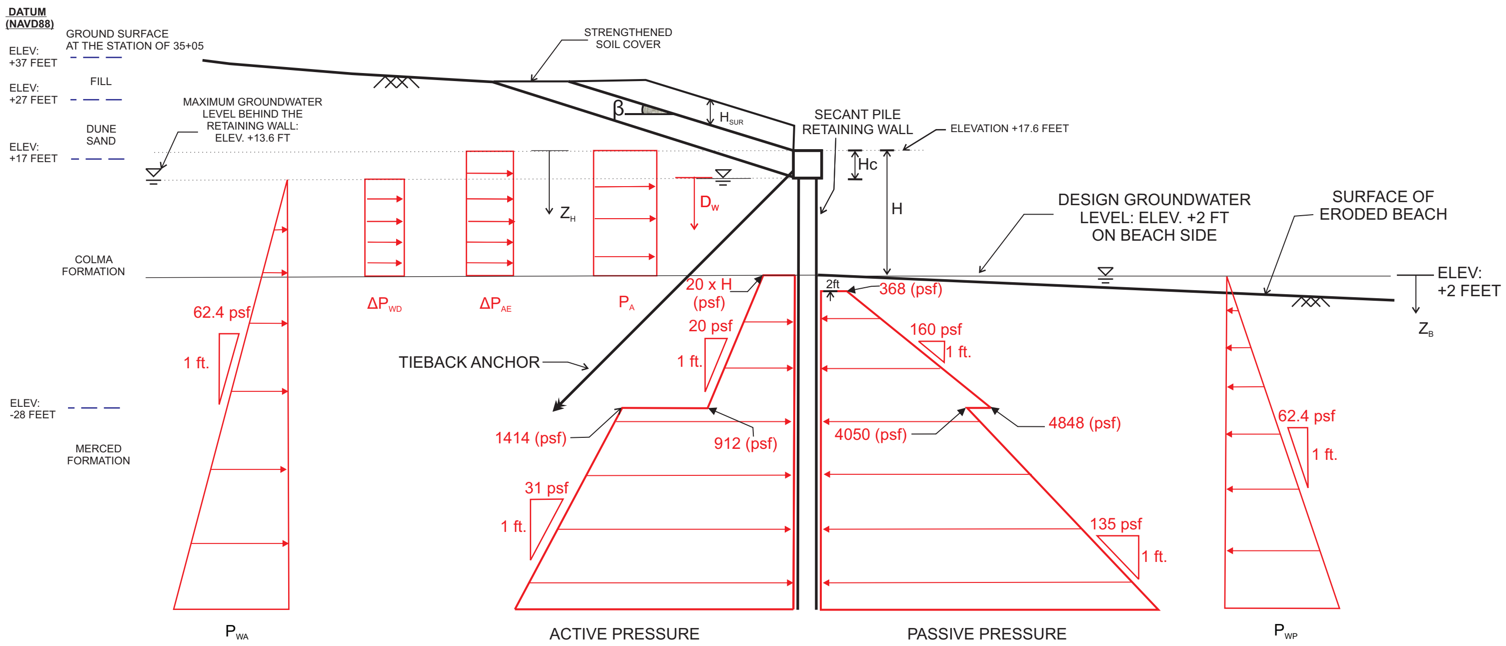
ΔP_{AE} = SEISMIC EARTH PRESSURE = 38 x (H+½H_{SUR}) (PSF)

ΔP_{WD} = HYDRODYNAMIC WATER PRESSURE = 23 x (H-Hc) (PSF)

NOT TO SCALE

LATERAL EARTH PRESSURES FOR SECANT PILE WALL RUBBLE REACH Sta. 27+40 - BEACH ELEVATION +10 FEET SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JULY 2021	PLATE 8F



NOTES:

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Hc= HEIGHT OF PILE CAP (FEET)

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Z_H = DEPTH BELOW TOP OF WALL (FEET)

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P_A = APPARENT EARTH PRESSURE = 35 x H_{SUR} + 18 x Hc + 17 x H (PSF)

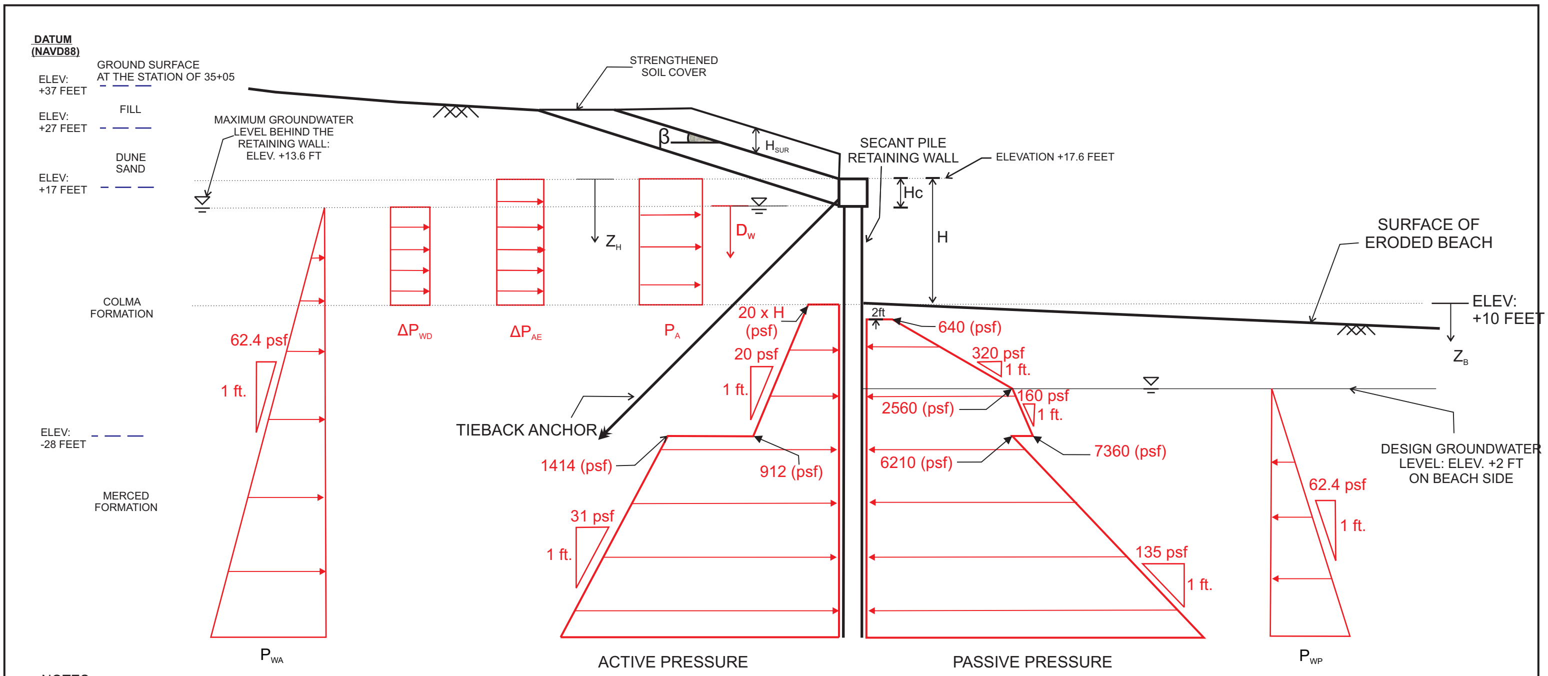
P_{WA} and P_{WP} = HYDROSTATIC PRESSURE (PSF)

ΔP_{AE} = SEISMIC EARTH PRESSURE = 10 x (H+½H_{SUR}) (PSF)

ΔP_{WD} = HYDRODYNAMIC WATER PRESSURE = 6 x (H-Hc) (PSF)

NOT TO SCALE

LATERAL EARTH PRESSURES FOR SECANT PILE WALL BLUFF REACH Sta. 35+05 - BEACH ELEVATION +2 FEET SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JULY 2021	PLATE 8G



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Hc= HEIGHT OF PILE CAP (FEET)

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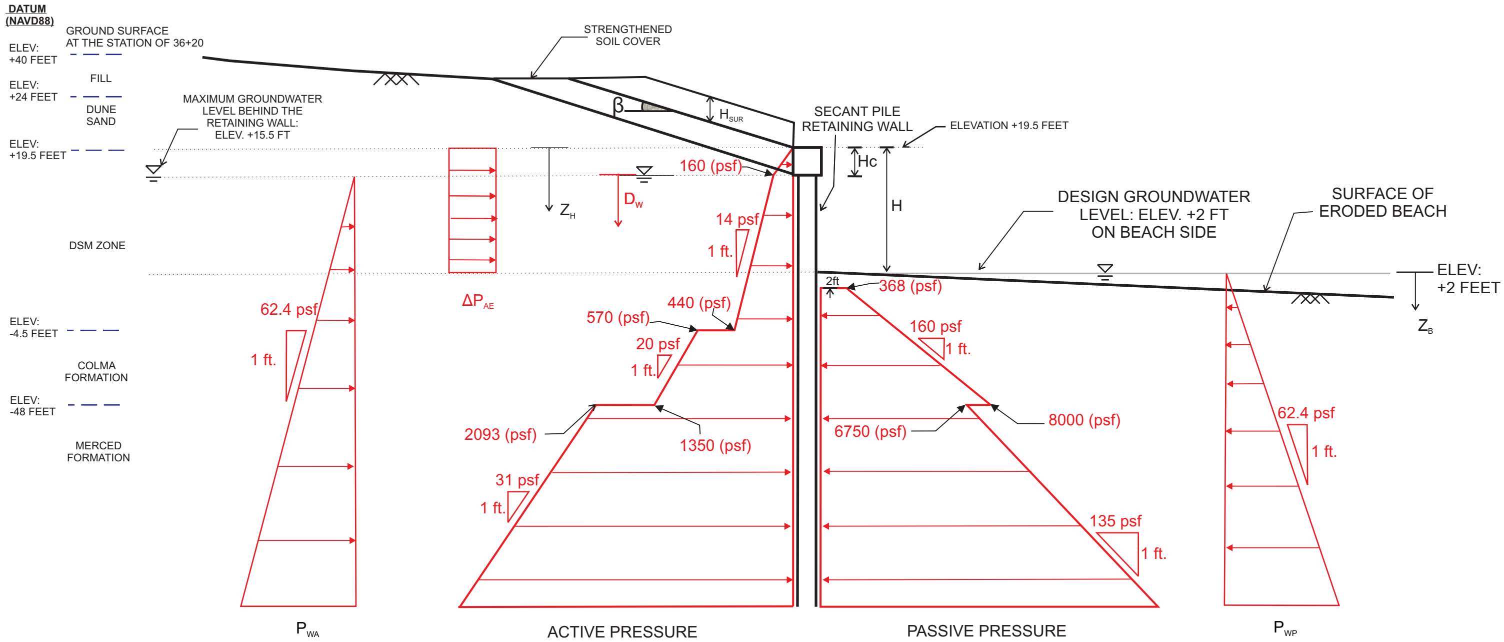
ΔP_{AE} = SEISMIC EARTH PRESSURE = 38 x (H+½H_{SUR}) (PSF)

ΔP_{WD} = HYDRODYNAMIC WATER PRESSURE = 23 x (H-Hc) (PSF)

NOT TO SCALE

LATERAL EARTH PRESSURES FOR SECANT PILE WALL BLUFF REACH Sta. 35+05 - BEACH ELEVATION +10 FEET SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JULY 2021	PLATE 8H



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Z_H = DEPTH BELOW TOP OF WALL (FEET)

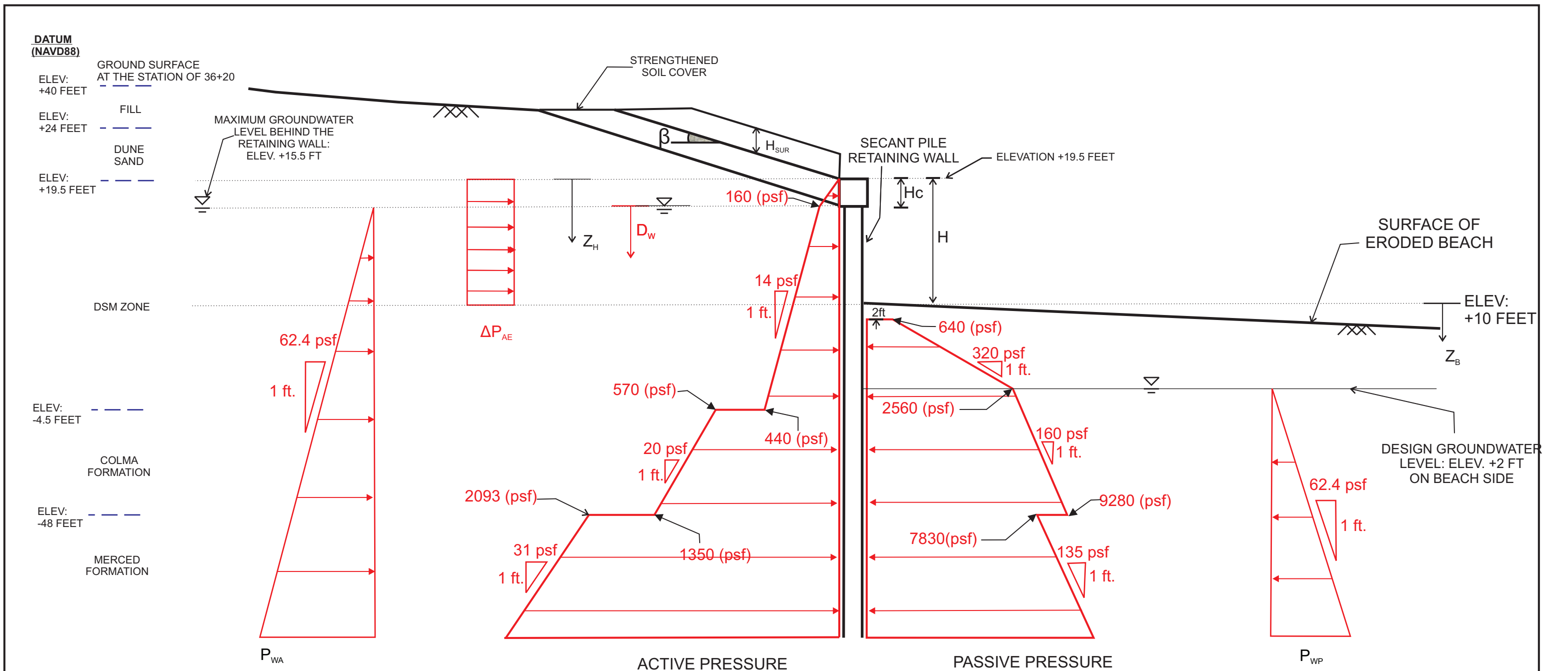
H_{SUR} = HEIGHT OF SAND LAYER SURCHARGE (FEET)

P_{WA} and P_{WP} = HYDROSTATIC PRESSURE (PSF)

ΔP_{AE} = SEISMIC EARTH PRESSURE = 10 x ($H+½H_{SUR}$) (PSF)

NOT TO SCALE

LATERAL EARTH PRESSURES FOR SECANT PILE WALL NEAR SWOO OVERLAPPING DSM Sta. 36+20 - BEACH ELEVATION +2 FEET SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JULY 2021	PLATE 8I



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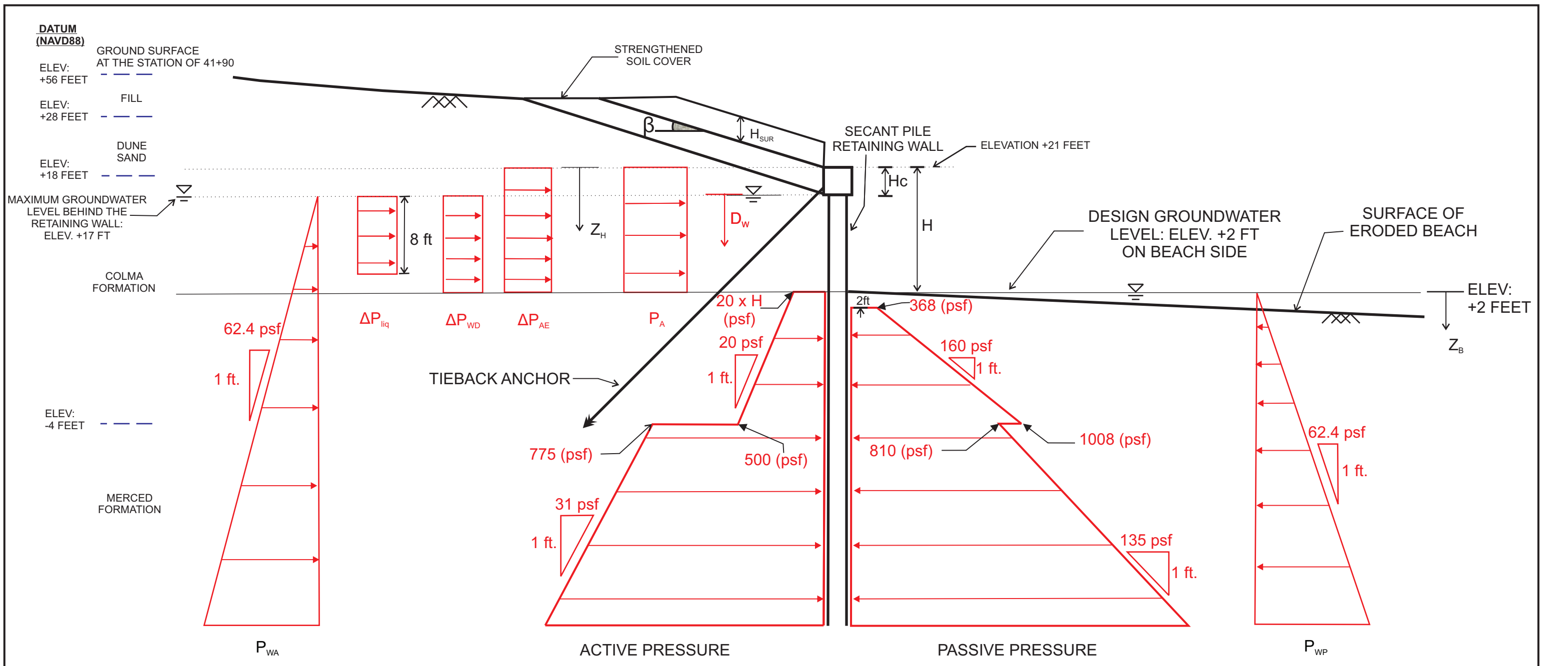
H_{SUR} = HEIGHT OF SAND LAYER SURCHARGE (FEET)

P_{WA} and P_{WP} = HYDROSTATIC PRESSURE (PSF)

ΔP_{AE} = SEISMIC EARTH PRESSURE = 38 x ($H+\frac{1}{2}H_{SUR}$) (PSF)

NOT TO SCALE

LATERAL EARTH PRESSURES FOR SECANT PILE WALL NEAR SWOO OVERLAPPING DSM Sta. 36+20 - BEACH ELEVATION +10 FEET SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JULY 2021	PLATE 8J



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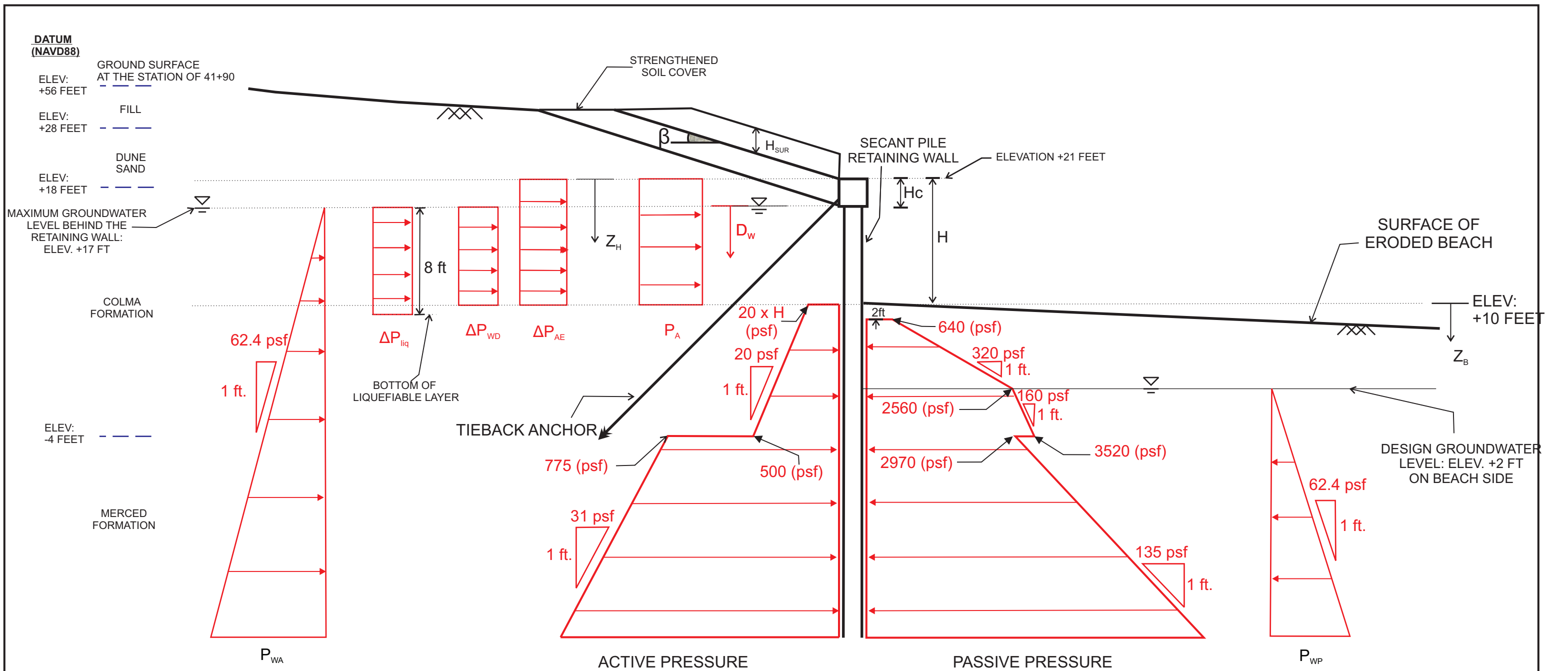
ΔP_{WD} = HYDRODYNAMIC WATER PRESSURE = 6 x (H-Hc) (PSF)

NOT TO SCALE

LATERAL EARTH PRESSURES FOR SECANT PILE WALL SOUTH REACH Sta. 41+90 - BEACH ELEVATION +2 FEET SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA	AGS
JOB NO. AGS-18-003 / DATE: JULY 2021	PLATE 8K



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ΔP_{AE} = SEISMIC EARTH PRESSURE = 38 x (H+½H_{SUR}) (PSF)

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NOT TO SCALE

LATERAL EARTH PRESSURES FOR SECANT PILE WALL SOUTH REACH Sta. 41+90 - BEACH ELEVATION +10 FEET SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JULY 2021	PLATE 8L

APPENDIX A

LIQUEFACTION ANALYSIS

APPENDIX A LIQUEFACTION ANALYSIS

A.1. GENERAL

Soil liquefaction is a phenomenon in which saturated (submerged) cohesionless soils experience a temporary loss of strength due to the build-up of excess pore water pressure during cyclic seismic loadings. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movements. Soils most susceptible to liquefaction are loose, clean, saturated, and uniformly graded, fine-grained sands, which lie within about 50 feet of the ground surface. Saturated loose silty and clayey sands may also liquefy during strong ground shaking.

This appendix presents the results of our liquefaction potential evaluation for the proposed improvement discussed in the main text. The liquefaction potential evaluation was based on the requirements of Special Publication 117A (CGS, 2008), using blow counts from Standard Penetration Test (SPT) samplers, and corrected blow counts from other samplers, in the boring logs from the site vicinity. The blow counts shown on these logs were corrected for various factors, as discussed below, and used in the liquefaction analyses.

The design earthquakes evaluated were a Maximum Moment Magnitude event of Mw 8.05 on the San Andreas Fault. The San Andreas Fault is located about 2.6 km southwest of the site. Our evaluations were made using the liquefaction evaluation procedure developed by National Center for Earthquake Engineering Research (NCEER), 1996; Youd and Idriss 2001, and Idriss and Boulanger (2008) based on liquefaction observation in previous earthquakes.

A comprehensive collection of site conditions at various locations where some evidence of liquefaction was known to have or to have not taken place was collected by Seed, et al. (1984). These data on sandy soils with a fines content less than 5 percent under magnitude 7.5 earthquake conditions was presented as relationships between field values of average cyclic stress ratio, T_{av}/σ'_o (where: T_{av} = average horizontal shear stress induced by an earthquake; and σ'_o = initial effective overburden pressure on the soil element), and the SPT blow counts corrected for certain effects. For an earthquake of magnitude 8.05, the cyclic shear stress ratio necessary to cause liquefaction in Seed's curve was corrected to account for the earthquake magnitude or duration effect (Idriss, 1996).

For the first step in estimating liquefaction potential, the measured SPT blow counts should be corrected for various factors using the method proposed by NCEER (1997). The raw SPT blow count, N, is corrected to obtain the modified penetration resistance, $N_{1(60)}$. The modified penetration resistance is computed as follows:

$N_{1(60)} = N \times C_m \times C_z \times C_h \times C_s \times C_n$ ....(A.1)

where:

N: raw SPT or Modified California blow count (blows/ft);

C_m: a factor to correct for the size of the sampler.

C_m = 1.0 for the SPT sampler; C_m = 0.61 for the Modified California sampler;

C_z: a factor that depends on the length of the drive rods; the following C_z factors may be used for various depths:

Depth	C_z
> 30 ft	1.0
13 to 20 ft	0.95
10 to 13 ft	0.85
< 10 ft	0.75

C_h: a factor that accounts for the hammer efficiency used in the field to normalize the actual hammer efficiency to the cathead-and-rope system efficiency of 60%.

$C_h = E_h/60\%$, where E_h is the hammer efficiency; 80% for this investigation.

C_s: a factor that depends on the sampling tube; for a split-spoon sampler without liner (ID = 1.5" and OD = 2.0"), the following C_s factors may be used:

Raw Blow Count, N	C_S
< 10	1.0
> 10	1.2

C_n: a factor that depends on the effective overburden pressure at the depth when the penetration test was conducted.

As presented by NCEER (1996), another correction factor, $\delta(N_1)_{60}$, should be added to $(N_1)_{60}$ to account for fine contents as follows:

$N_{1(60)cs} = N_{1(60)} + \delta\, N_{1(60)}$ ....(A.2)

$\delta\, N_{1(60)} = \exp\left(1.63 + \frac{9.7}{FC+0.01} - \left(\frac{15.7}{FC+0.01}\right)^2\right)$ ....(A.3)

Where FC is the fines content expressed as a decimal. (So 45% fines is 0.45)

The average cyclic stress ratio (CSR), τ_{av}/σ'_o, at a specific depth can be estimated from dynamic site response analyses. It also can be estimated with reasonable accuracy from the following equation as discussed by Seed and Idriss (1982).

$\tau_{av}/\sigma'_o = 0.65 \times a_{max}/g \times \sigma_o/\sigma'_o \times r_d$ ....(A.4) where:

a_{max}: maximum acceleration at the ground surface;

σ_o: total overburden pressure;

σ_o': effective overburden pressure; and

r_d: a stress reduction factor.

Based on the magnitude of the design earthquake, and the peak ground acceleration generated by that earthquake, the cyclic stress ratio was calculated using Equation E.3. The cyclic stress ratio was then corrected to account for an earthquake magnitude other than 7.5. Cumulative liquefaction and seismically-induced settlements are attached for Borings B-1 to B-8

Equation B.5 is used to estimate corrected cyclic resistance ratio (CRR).

$$CRR_R = CRR_M \times MSF \times K_s \quad \text{(A.5)}$$

The Magnitude Scaling Factor (MSF) is used to adjust the CRR to a common value of M_w=7.5, because the CRR depends on the number of loading cycles, which correlates with M_w (Seed et al. 1975b). MSF is calculated using equation B.6.

$$MSF = 6.9 \exp\left(\frac{-\mathrm{MCE}}{4} - 0.058\right) \leq 1.8 \quad \text{(A.6)}$$

The overburden correction factor (K_s) was introduced by Seed (1983) to adjust the CRR. Overburden correction factor can be estimated using equation B.7.

$$K_s = 1 - \left(\frac{1}{18.9 - 2.55\sqrt[2]{(N_1)_{60\,cs} \geq 37}}\right) \times \ln\left(\frac{\left(\frac{\sigma_0'}{21.09}\right)}{101}\right) \geq 1.1 \quad \text{(A.7)}$$

The derived correlation between CRR and penetration resistances is expressed via following equations developed by Idriss and Boulanger (2008).

For $(N_1)_{60\,cs} < 37.5$,

$$CRR_M = \exp\left(\left(\frac{(N_1)_{60\,cs}}{14.1}\right) + \left(\frac{(N_1)_{60\,cs}}{126}\right)^2 - \left(\frac{(N_1)_{60\,cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60\,cs}}{25.4}\right)^4 - 2.8\right) \quad \text{(A.8)}$$

For $((N_1)_{60\,cs} \geq 37.5$,

$$CRR_M = 2 \quad \text{(A.9)}$$

Finally, CRR can be estimated using equation B.10.

$$CRR = MIN(2, CRR_R) \quad \text{(A.10)}$$

The factor of safety against liquefaction can be estimated using equation B.11.

$$FS_{liq} = CRR/CSR \text{..........} (A.11)$$

A.2. SEISMICALLY-INDUCED SETTLEMENT

For coarse-grained soils such as sand and gravel with various amount of silt and clay, AGS used a liquefaction evaluation approach developed over the years by Seed and his co-authors.

For fine-grained soils such as silt and clay, there are currently two screening procedures. Both approaches are based on modified Chinese Criteria for liquefaction evaluation of fine-grained soils. The first approach was developed by Bray and Sancio (2006), and another approach was developed by Boulanger and Idriss (2006). The Bray and Sancio (2006) criteria state that a soil is:

a) Susceptible to liquefaction if w_c/LL > 0.85, PI < 12, and LL<37

b) Moderately susceptible to liquefaction if 0.8 < w_c/LL< 0.9 and 12< PI <18

c) Not susceptible to liquefaction if w_c/LL< 0.8 and PI >18

where w_c is water content, LL is Liquid Limit, and PI is Plasticity Index. The criteria presented by Boulanger and Idriss (2006) state that a soil is

a) sand-like if PI < 7
b) clay-like if PI > 7

where sand-like soils are susceptible to liquefaction, and clay-like soils should be evaluated using Boulanger and Idriss (2004) method based on the cyclic triaxial shear testing. The method of analyses is based on the relationship proposed by Tokimatsu and Seed (1987), and Ishihara and Yoshimine (1992), Zhang et al. (2004) with scale effect modification.

A.3. SEISMICALLY-INDUCED LATERAL DEFORMATION

Seismically-induced lateral deformation (lateral spreading) is another phenomenon which can occur during a seismic event. The potential for lateral deformation of the soil is evaluated using empirical relationships developed by Zhang et al. (2004). A “Lateral Deformation Index” (LDI) is calculated from estimated shear strains in each liquefiable layer based on the soil properties and thickness of the liquefiable layer, the magnitude of the earthquake from the site, and the intensity of the ground shaking. Once the LDI is calculated, estimated horizontal ground movement is calculated from the LDI and the boundary conditions (ground slope, slope of liquefiable layer, or distance to and height of a free face).

The continuity of potentially liquefiable soil layers is a key consideration in evaluating the potential for seismically-induced lateral deformation. It should be noted that for a significant areal lateral deformation to occur, a continuous layer of potentially liquefiable soil extending for a distance on the order of several hundred feet is required.

A.4. RESULTS

AGS' liquefaction evaluation was based on the following:

- PGA_M for two hazard levels (72-year and 975-year return period); 0.25g and 0.96g respectively, and
- Groundwater at Elevation +16 feet (for the North, EQR and Rubble Reaches), Elevation +18 feet (for the Bluff Reach) and Elevation +19 feet (for the South Reach).

Based on the results of AGS' evaluation and the estimated thicknesses of the liquefiable soils, the estimated seismically-induced settlement of the project area would be about 0 to 6 inches. Based on the site conditions, seismically-induced lateral spreading appears to not be a hazard at this site. Results of our analyses are presented on the following pages. A summary of our results is presented in Table 8 in Section 3.5.3 of this report.

AGS-18-003 South Ocean Beach Coastal Erosion and Wastewat

Site Data

S	Ground Slope / Liquefiable Layer Slope	0.1%
L	Horizontal Distance from Free Face (ft)	15
H	Free Face Height (ft)	30
Gw	Depth to groundwater (ft)	24
Mw	Maximum Moment Magnitude	8.1
PGA	Peak Ground Acceleration	1.02

Boring Data

Depth of boring (ft)	71.5
Depth to groundwater (ft)	15.0
Borehole diameter (in)	6

Settlement

ft	in	
0.39	4.68	
0.00	0.00	(above YBM)
0.39	4.68	(in upper 50 feet)
0.39	4.68	(in upper 70 feet)

Lateral Displacement

LDI	Sloping		Free Face	
	ft	in	ft	in
1.862	0.4	4.5	19.5	233.4

Sample Depth is to top of sample layer.

Fines content estimated based on soil classification where no test data available.

Soil Type based on Unified Soils Classification System (USCS), ASTM D2487.

Equations and procedures used to calculate liquefaction potential is based on Soil Liquefaction During Earthquakes by Idriss and Boulanger published by EERI MNO-12 2008

Blue cells are input data, orange cells are calculated data.

Seismically-Induced Settlement & Lateral Deformation Calculation - AGS-2010 R1-B3

SAMPLE DEPTH (feet)	SAMPLE TYPE	UNCORRECTED BLOW COUNT (SPT-N) (blows per foot)	SOIL TYPE	PI Clay/Silt	ESTIMATED FINES CONTENT (%)	GRAVEL? Y=yes N=no	SAMPLER LINER? Y=yes N=no	UNIT WEIGHT OF SOIL LAYER (pcf)	ENERGY RATIO, ER (%)	PORE PRESSURE (psf)	EFFECTIVE OVER-BURDEN (psf)	EFFECTIVE OVER-BURDEN (MPa)	CORRECTION FACTORS: C_b Borehole Size Factor	Cn Depth Reduction Factor	Cm MC Factor	CR Drill Rod Length Factor	Ch Hammer Efficiency Factor	Cs Unlined SPT Factor	PENETRATION RESISTANCE N60	MODIFIED PENETRATION RESISTANCE (N1)60
2.0	spt	44	SP-SM		5	n	n	130	75	0	260	12.45	1.15	1.70	1.00	0.75	1.25	1.30	47.44	80.6
5.0	spt	40	SM		13	n	n	110	75	0	590	28.26	1.15	1.43	1.00	0.75	1.25	1.30	43.13	61.6
6.5	spt	44	SM		13	n	n	100	75	0	740	35.44	1.15	1.32	1.00	0.80	1.25	1.30	50.60	66.7
10.0	spt	34	SM		13	n	n	115	75	0	1142.5	54.72	1.15	1.20	1.00	0.80	1.25	1.30	39.10	47.1
15.0	spt	24	SP-SM		7	n	n	115	75	0	1717.5	82.26	1.15	1.08	1.00	0.85	1.25	1.30	29.33	31.6
20.0	spt	24	SW-GC		7	n	n	115	75	0	2292.5	109.79	1.15	0.97	1.00	0.95	1.25	1.30	32.78	31.8
25.0	spt	19	SP-SM		10	n	n	115	75	62.4	2805.1	134.34	1.15	0.89	1.00	0.95	1.25	1.23	25.95	23.2
30.0	spt	27	SP-SM		6	n	n	115	75	374.4	3068.1	146.94	1.15	0.89	1.00	0.95	1.25	1.30	36.87	32.7
31.5	spt	18	SP-SM		6	n	n	115	75	468	3147	150.72	1.15	0.85	1.00	1.00	1.25	1.22	25.88	22.1
40.0	spt	23	SC-SM		20	n	n	115	75	998.4	3594.1	172.13	1.15	0.83	1.00	1.00	1.25	1.28	33.06	27.5
50.0	spt	38	SC-SM		20	n	n	115	75	1622.4	4120.1	197.32	1.15	0.84	1.00	1.00	1.25	1.30	54.63	45.8
60.0	spt	65	SM		14	n	n	110	75	2246.4	4596.1	220.12	1.15	0.81	1.00	1.00	1.25	1.30	93.44	76.1
70.0	spt	63	SM		14	n	n	105	75	2870.4	5022.1	240.52	1.15	0.80	1.00	1.00	1.25	1.30	90.56	72.1

SAMPLE DEPTH (feet)	δN for Fines Content	MOD PEN RES ADJUSTED TO CLEAN SAND (N1)60CS	STRESS REDUCTION COEFFICIENT (rd)	CYCLIC STRESS RATIO (CSR)	CYCLIC RESISTANCE RATIO (CRR) Sand	CYCLIC RESISTANCE RATIO (CRR) Clay	CRR OVERBURDEN CORRECTION (Ks)	CRR STATIC SHEAR CORRECTION (Ka)	CORRECTED CRR (CCR_R) Sand	CORRECTED CRR (CCR_R) Clay	MAGNITUDE SCALING FACTOR (MSF)	FACTOR OF SAFETY (FS)Clay	FACTOR OF SAFETY (FS)Sand	LIQUEFACTION POTENTIAL	FS
2.0	0.0	80.65	1.00	0.66	1.901	10.86	1.10	1.00	2.000	11.94	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00
5.0	2.5	64.84	1.00	0.66	1.901	4.76	1.10	1.00	2.000	5.23	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00
6.5	2.5	69.91	1.00	0.66	1.901	3.81	1.10	1.00	2.000	4.20	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00
10.0	2.5	50.36	0.99	0.66	1.901	2.43	1.10	1.00	2.000	2.68	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00
15.0	0.1	31.80	0.98	0.65	0.566	1.29	1.05	1.00	0.625	1.35	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00
20.0	0.1	32.02	0.97	0.64	0.549	0.97	0.98	1.00	0.646	0.95	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00
25.0	1.1	24.61	0.96	0.65	0.232	0.62	0.96	1.00	0.281	0.60	0.86	-	0.358	YES, LIQ.	0.36
30.0	0.0	32.77	0.94	0.70	0.577	0.81	0.92	1.00	0.730	0.74	0.86	-	0.823	YES, LIQ.	0.82
31.5	0.0	22.14	0.94	0.71	0.192	0.53	0.94	1.00	0.235	0.50	0.86	-	0.268	YES, LIQ.	0.27
40.0	4.5	33.26	0.91	0.77	0.600	0.59	0.88	1.00	0.794	0.52	0.86	-	0.777	YES, LIQ.	0.78
50.0	4.5	51.62	0.88	0.81	1.392	0.68	0.81	1.00	2.000	0.55	0.86	-	1.712	NO, NOT LIQ.	1.71
60.0	2.9	79.90	0.85	0.83	1.336	0.63	0.77	1.00	2.000	0.49	0.86	-	1.601	NO, NOT LIQ.	1.60
70.0	2.9	75.85	0.81	0.85	1.291	0.58	0.75	1.00	2.000	0.43	0.86	-	1.527	NO, NOT LIQ.	1.53

Liquefaction Settlement / Lateral Displacement Index and Settlement

SAMPLE DEPTH (feet)	LIQUEFACTION POTENTIAL Incl. chk against PI	Soil Layer Thickness (ft)	Dynamic Vol. Strain (%): Tokimatsu	Dynamic Vol. Strain (%): Ishihara	Liquefaction Settlement (ft): Tokimatsu	Liquefaction Settlement (ft): Ishihara	Limiting Shear Strain gmin	Parameter Fa	Maximum Shear Strain gmax	DH (ft)	DLD_{ii} (ft)	Vertical Reconsol. Strain ev	Liquefaction Settlement (ft) Ishihara
2.00	NO, NOT LIQ.	5.0	0.05	1.30	0.00	0.00	0.00	-4.256	0.00	5.00	0.000	0.000	0.00
5.00	NO, NOT LIQ.	1.5	0.05	1.30	0.00	0.00	0.00	-2.841	0.00	1.50	0.000	0.000	0.00
6.50	NO, NOT LIQ.	3.5	0.05	1.30	0.00	0.00	0.00	-3.287	0.00	3.50	0.000	0.000	0.00
10.00	NO, NOT LIQ.	5.0	0.05	1.30	0.00	0.00	0.00	-1.618	0.00	5.00	0.000	0.000	0.00
15.00	NO, NOT LIQ.	5.0	0.55	1.30	0.00	0.00	0.04	-0.211	0.00	5.00	0.000	0.000	0.00
20.00	NO, NOT LIQ.	5.0	0.55	1.30	0.00	0.00	0.03	-0.226	0.00	5.00	0.000	0.000	0.00
25.00	YES, LIQ.	5.0	1.20	2.15	0.06	0.11	0.09	0.256	0.09	5.00	0.465	0.019	0.10
30.00	YES, LIQ.	1.5	0.55	0.74	0.01	0.01	0.03	-0.278	0.03	1.50	0.047	0.006	0.01
31.50	YES, LIQ.	8.5	1.20	2.15	0.10	0.18	0.12	0.400	0.12	8.50	1.059	0.021	0.18
40.00	YES, LIQ.	10.0	0.15	0.89	0.02	0.09	0.03	-0.312	0.03	10.00	0.289	0.005	0.05
50.00	NO, NOT LIQ.	10.0	0.05	0.09	0.00	0.00	0.00	-1.721	0.00	10.00	0.001	0.000	0.00
60.00	NO, NOT LIQ.	10.0	0.05	0.09	0.00	0.00	0.00	-4.187	0.00	10.00	0.000	0.000	0.00
70.00	NO, NOT LIQ.	1.5	0.05	0.15	0.00	0.00	0.00	-3.820	0.00	1.50	0.000	0.000	0.00

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Site Data

S	Ground Slope / Liquefiable Layer Slope	0.1%
L	Horizontal Distance from Free Face (ft)	25
H	Free Face Height (ft)	15
Gw	Depth to groundwater (ft)	19.9
Mw	Maximum Moment Magnitude	8.1
PGA	Peak Ground Acceleration	1.02

Boring Data

Depth of boring (ft)	31.5
Depth to groundwater (ft)	19.9
Borehole diameter (in)	6

Settlement

ft	in	
0.14	1.66	
0.00	0.00	(above YBM)
0.14	1.66	(in upper 50 feet)
0.14	1.66	(in upper 70 feet)

Lateral Displacement

LDI	Sloping		Free Face	
	ft	in	ft	in
1.374	0.3	3.3	5.5	65.8

Sample Depth is to top of sample layer.
Fines content estimated based on soil classification where no test data available.
Soil Type based on Unified Soils Classification System (USCS), ASTM D2487.
Equations and procedures used to calculate liquefaction potential is based on Soil Liquefaction During Earthquakes by Idriss and Boulanger published by EERI MNO-12 2008
Blue cells are input data, orange cells are calculated data.

Seismically-Induced Settlement & Lateral Deformation Calculation - AGS-2010 R1-C2

SAMPLE DEPTH (feet)	SAMPLE TYPE	UNCORRECTED BLOW COUNT (SPT-N) (blows per foot)	SOIL TYPE	PI Clay/Silt	ESTIMATED FINES CONTENT (%)	GRAVEL? Y=yes N=no	SAMPLER LINER? Y=yes N=no	UNIT WEIGHT OF SOIL LAYER (pcf)	ENERGY RATIO, ER (%)	PORE PRESSURE (psf)	EFFECTIVE OVER-BURDEN (psf)	EFFECTIVE OVER-BURDEN (MPa)
2.0	spt	29	SP-SM		5	n	n	130	70	0	260	12.45
5.0	spt	28	SP-SM		5	n	n	115	70	0	605	28.98
7.5	spt	15	SP-SM		5	n	n	115	70	0	892.5	42.74
10.0	spt	12	SM		33	n	n	115	70	0	1180	56.51
15.0	spt	10	SM		12	n	n	115	70	0	1755	84.05
20.0	spt	12	SP-SM		8	n	n	115	70	6.24	2323.76	111.29
25.0	spt	37	SP-SM		5	n	n	115	70	318.24	2586.76	123.89
30.0	spt	32	SP-SM		5	n	n	115	70	630.24	2849.76	136.48

SAMPLE DEPTH (feet)	CORRECTION FACTORS: C_b Borehole Size Factor	Cn Depth Reduction Factor	Cm MC Factor	CR Drill Rod Length Factor	Ch Hammer Efficiency Factor	Cs Unlined SPT Factor	PENETRATION RESISTANCE N60	MODIFIED PENETRATION RESISTANCE (N1)60	δN for Fines Content	MOD PEN RES ADJUSTED TO CLEAN SAND (N1)60CS	STRESS REDUCTION COEFFICIENT (rd)	CYCLIC STRESS RATIO (CSR)	CYCLIC RESISTANCE RATIO (CRR) Sand	CYCLIC RESISTANCE RATIO (CRR) Clay	CRR OVERBURDEN CORRECTION (Ks)	CRR STATIC SHEAR CORRECTION (Ka)	CORRECTED CRR (CCR_R) Sand	CORRECTED CRR (CCR_R) Clay	MAGNITUDE SCALING FACTOR (MSF)	FACTOR OF SAFETY (FS)Clay	FACTOR OF SAFETY (FS)Sand	LIQUEFACTION POTENTIAL	FS
2.0	1.15	1.70	1.00	0.75	1.17	1.30	29.18	49.6	0.0	49.61	1.00	0.66	1.901	10.28	1.10	1.00	2.000	11.31	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00
5.0	1.15	1.60	1.00	0.75	1.17	1.30	28.18	45.1	0.0	45.08	1.00	0.66	1.901	4.27	1.10	1.00	2.000	4.69	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00
7.5	1.15	1.51	1.00	0.80	1.17	1.24	16.10	24.2	0.0	24.24	0.99	0.66	0.260	1.55	1.10	1.00	0.273	1.70	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00
10.0	1.15	1.34	1.00	0.80	1.17	1.17	12.88	17.3	5.5	23.71	0.99	0.66	0.247	0.92	1.09	1.00	0.262	1.00	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00
15.0	1.15	1.10	1.00	0.85	1.17	1.13	11.40	12.6	2.1	14.89	0.98	0.65	0.137	0.51	1.02	1.00	0.155	0.52	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00
20.0	1.15	0.95	1.00	0.95	1.17	1.15	15.30	14.6	0.4	15.02	0.97	0.64	0.134	0.47	0.99	1.00	0.156	0.46	0.86	-	0.208	YES, LIQ.	0.21
25.0	1.15	0.95	1.00	0.95	1.17	1.30	47.16	44.7	0.0	44.69	0.96	0.71	1.629	1.08	0.94	1.00	2.000	1.02	0.86	-	2.000	NO, NOT LIQ.	2.00
30.0	1.15	0.92	1.00	0.95	1.17	1.30	40.79	37.3	0.0	37.34	0.94	0.76	1.507	0.97	0.91	1.00	1.907	0.89	0.86	-	1.974	NO, NOT LIQ.	1.97
					0.00																		

Liquefaction Settlement

SAMPLE DEPTH (feet)	LIQUEFACTION POTENTIAL Incl. chk against PI	Soil Layer Thickness (ft)	Dynamic Vol. Strain (%) Tokimatsu	Dynamic Vol. Strain (%) Ishihara	Liquefaction Settlement (ft) Tokimatsu	Liquefaction Settlement (ft) Ishihara
2.00	NO, NOT LIQ.	5.0	0.05	1.30	0.00	0.00
5.00	NO, NOT LIQ.	2.5	0.05	1.30	0.00	0.00
7.50	NO, NOT LIQ.	2.5	1.20	2.15	0.00	0.00
10.00	NO, NOT LIQ.	5.0	1.20	2.15	0.00	0.00
15.00	NO, NOT LIQ.	5.0	2.20	2.76	0.00	0.00
20.00	YES, LIQ.	5.0	1.90	2.76	0.10	0.14
25.00	NO, NOT LIQ.	5.0	0.05	0.00	0.00	0.00
30.00	NO, NOT LIQ.	1.5	0.05	0.09	0.00	0.00

Lateral Displacement Index and Settlement

Limiting Shear Strain gmin	Parameter Fa	Maximum Shear Strain gmax	DH (ft)	DLD_{ii} (ft)	Vertical Reconsol. Strain ev	Liquefaction Settlement (ft) Ishihara
0.00	-1.557	0.00	5.00	0.000	0.000	0.00
0.00	-1.196	0.00	2.50	0.000	0.000	0.00
0.10	0.278	0.00	2.50	0.000	0.000	0.00
0.10	0.310	0.00	5.00	0.000	0.000	0.00
0.28	0.759	0.00	5.00	0.000	0.000	0.00
0.27	0.754	0.27	5.00	1.373	0.029	0.14
0.00	-1.165	0.00	5.00	0.000	0.000	0.00
0.01	-0.606	0.00	1.50	0.001	0.000	0.00

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Site Data

S	Ground Slope / Liquefiable Layer Slope	0.1%
L	Horizontal Distance from Free Face (ft)	80
H	Free Face Height (ft)	30
Gw	Depth to groundwater (ft)	15
Mw	Maximum Moment Magnitude	8.1
PGA	Peak Ground Acceleration	1.02

Boring Data

Depth of boring (ft)	66.5
Depth to groundwater (ft)	15.0
Borehole diameter (in)	6

Settlement		
ft	in	
0.00	0.00	
0.00	0.00	(above YBM)
0.00	0.00	(in upper 50 feet)
0.00	0.00	(in upper 70 feet)

Lateral Displacement				
LDI	Sloping		Free Face	
	ft	in	ft	in
0.009	0.0	0.0	0.0	0.3

Sample Depth is to top of sample layer.
Fines content estimated based on soil classification where no test data available.
Soil Type based on Unified Soils Classification System (USCS), ASTM D2487.
Equations and procedures used to calculate liquefaction potential is based on Soil Liquefaction During Earthquakes by Idriss and Boulanger published by EERI MNO-12 2008
Blue cells are input data, orange cells are calculated data.

Seismically-Induced Settlement & Lateral Deformation Calculation - AGS-1989 B-5

SAMPLE DEPTH (feet)	SAMPLE TYPE	UNCORRECTED BLOW COUNT (SPT-N) (blows per foot)	SOIL TYPE	PI Clay/Silt	ESTIMATED FINES CONTENT (%)	GRAVEL? Y=yes N=no	SAMPLER LINER? Y=yes N=no	UNIT WEIGHT OF SOIL LAYER (pcf)	ENERGY RATIO, ER (%)	PORE PRESSURE (psf)	EFFECTIVE OVER-BURDEN (psf)	EFFECTIVE OVER-BURDEN (MPa)	CORRECTION FACTORS: C_b Borehole Size Factor	Cn Depth Reduction Factor	Cm MC Factor	CR Drill Rod Length Factor	Ch Hammer Efficiency Factor	Cs Unlined SPT Factor
0.1	spt	19	SM		12	y	n	130	75	0	13	0.62	1.15	1.70	1.00	0.75	1.25	1.30
5.0	spt	29	SP			n	n	110	75	0	552	26.44	1.15	1.61	1.00	0.75	1.25	1.30
10.0	spt	33	SP			n	n	100	75	0	1052	50.38	1.15	1.24	1.00	0.80	1.25	1.30
15.0	spt	17	SM		12	n	n	115	75	0	1627	77.92	1.15	1.12	1.00	0.85	1.25	1.23
20.0	spt	29	SM		27	n	n	115	75	312	1890	90.52	1.15	1.03	1.00	0.95	1.25	1.30
25.0	spt	32	SM		27	n	n	115	75	624	2153	103.11	1.15	0.99	1.00	0.95	1.25	1.30
30.0	spt	29	SP-SM		10	n	n	115	75	936	2416	115.71	1.15	0.96	1.00	0.95	1.25	1.30
35.0	spt	67	SP-SM		10	n	n	115	75	1248	2679	128.30	1.15	0.94	1.00	1.00	1.25	1.30
40.0	spt	73	SP-SM		10	n	n	115	75	1560	2942	140.90	1.15	0.92	1.00	1.00	1.25	1.30
45.0	spt	92	SP-SM		10	n	n	115	75	1872	3205	153.50	1.15	0.90	1.00	1.00	1.25	1.30
50.0	spt	67	SP-SM		10	n	n	115	75	2184	3468	166.09	1.15	0.88	1.00	1.00	1.25	1.30
55.0	spt	51	SM		20	n	n	110	75	2496	3706	177.49	1.15	0.86	1.00	1.00	1.25	1.30
60.0	spt	32	CL	20	80	n	n	105	75	2808	3919	187.69	1.15	0.85	1.00	1.00	1.25	1.30
65.0	spt	68	SM		12	n	n	110	75	3120	4157	199.09	1.15	0.84	1.00	1.00	1.25	1.30

SAMPLE DEPTH (feet)	PENETRATION RESISTANCE N60	MODIFIED PENETRATION RESISTANCE (N1)60	dN for Fines Content	MOD PEN RES ADJUSTED TO CLEAN SAND (N1)60CS	STRESS REDUCTION COEFFICIENT (rd)	CYCLIC STRESS RATIO (CSR)	CYCLIC RESISTANCE RATIO (CRR) Sand	CYCLIC RESISTANCE RATIO (CRR) Clay	CRR OVERBURDEN CORRECTION (Ks)	CRR STATIC SHEAR CORRECTION (Ka)	CORRECTED CRR (CCR_R) Sand	CORRECTED CRR (CCR_R) Clay	MAGNITUDE SCALING FACTOR (MSF)	FACTOR OF SAFETY (FS)Clay	FACTOR OF SAFETY (FS)Sand	LIQUEFACTION POTENTIAL
0.1	20.48	34.8	2.1	37.52	1.01	0.67	1.901	134.73	1.10	1.00	2.000	148.20	0.86	-	ABOVE G.W.	ABOVE G.W.
5.0	31.27	50.3	0.0	50.29	1.00	0.66	1.901	4.84	1.10	1.00	2.000	5.30	0.86	-	ABOVE G.W.	ABOVE G.W.
10.0	37.95	47.1	0.0	47.11	0.99	0.66	1.901	2.64	1.10	1.00	2.000	2.90	0.86	-	ABOVE G.W.	ABOVE G.W.
15.0	20.77	23.2	2.1	25.60	0.98	0.65	0.280	0.96	1.05	1.00	0.310	1.01	0.86	-	ABOVE G.W.	ABOVE G.W.
20.0	39.60	40.9	5.2	47.71	0.97	0.75	1.789	1.41	1.04	1.00	2.000	1.46	0.86	-	2.000	NO, NOT LIQ.
25.0	43.70	43.5	5.2	50.23	0.96	0.82	1.723	1.29	1.00	1.00	2.000	1.28	0.86	-	2.000	NO, NOT LIQ.
30.0	39.60	38.0	1.1	39.61	0.94	0.87	1.664	1.11	0.96	1.00	2.000	1.07	0.86	-	1.919	NO, NOT LIQ.
35.0	96.31	90.4	1.1	91.93	0.93	0.90	1.611	1.09	0.93	1.00	2.000	1.01	0.86	-	1.767	NO, NOT LIQ.
40.0	104.94	96.1	1.1	97.63	0.91	0.93	1.564	1.00	0.90	1.00	2.000	0.90	0.86	-	1.689	NO, NOT LIQ.
45.0	132.25	118.5	1.1	119.95	0.90	0.94	1.520	0.94	0.88	1.00	2.000	0.83	0.86	-	1.614	NO, NOT LIQ.
50.0	96.31	84.5	1.1	85.99	0.88	0.95	1.480	0.84	0.86	1.00	2.000	0.72	0.86	-	1.557	NO, NOT LIQ.
55.0	73.31	63.2	4.5	69.03	0.86	0.96	1.446	0.77	0.84	1.00	2.000	0.64	0.86	-	1.510	NO, NOT LIQ.
60.0	46.00	39.1	5.5	46.29	0.85	0.96	NA	0.71	0.82	1.00	NA	0.58	0.86	0.60	NA	YES, LIQ.
65.0	97.75	81.8	2.1	84.46	0.83	0.96	1.387	0.70	0.80	1.00	2.000	0.56	0.86	-	1.443	NO, NOT LIQ.

FS	Liquefaction Settlement: SAMPLE DEPTH (feet)	LIQUEFACTION POTENTIAL Incl. chk against PI	Soil Layer Thickness (ft)	Dynamic Vol. Strain (%): Tokimatsu	Dynamic Vol. Strain (%): Ishihara	Liquefaction Settlement (ft): Tokimatsu	Liquefaction Settlement (ft): Ishihara	Lateral Displacement Index and Settlement: Limiting Shear Strain gmin	Parameter Fa	Maximum Shear Strain gmax	DH (ft)	DLD_{ii} (ft)	Vertical Reconsol. Strain ev	Liquefaction Settlement (ft) Ishihara
0.00	0.10	NO, NOT LIQ.	3.0	0.05	1.30	0.00	0.00	0.01	-0.619	0.00	5.00	0.000	0.000	0.00
0.00	5.00	NO, NOT LIQ.	7.0	0.05	1.30	0.00	0.00	0.00	-1.612	0.00	5.00	0.000	0.000	0.00
0.00	10.00	NO, NOT LIQ.	5.0	0.05	1.30	0.00	0.00	0.00	-1.356	0.00	5.00	0.000	0.000	0.00
0.00	15.00	NO, NOT LIQ.	5.0	1.10	1.72	0.00	0.00	0.08	0.183	0.00	5.00	0.000	0.000	0.00
2.00	20.00	NO, NOT LIQ.	5.0	0.05	0.00	0.00	0.00	0.00	-1.404	0.00	5.00	0.000	0.000	0.00
2.00	25.00	NO, NOT LIQ.	5.0	0.05	0.00	0.00	0.00	0.00	-1.608	0.00	5.00	0.000	0.000	0.00
1.92	30.00	NO, NOT LIQ.	5.0	0.05	0.09	0.00	0.00	0.01	-0.767	0.00	5.00	0.009	0.000	0.00
1.79	35.00	NO, NOT LIQ.	5.0	0.05	0.09	0.00	0.00	0.00	-5.303	0.00	5.00	0.000	0.000	0.00
1.69	40.00	NO, NOT LIQ.	5.0	0.05	0.09	0.00	0.00	0.00	-5.842	0.00	5.00	0.000	0.000	0.00
1.61	45.00	NO, NOT LIQ.	5.0	0.05	0.09	0.00	0.00	0.00	-8.005	0.00	5.00	0.000	0.000	0.00
1.56	50.00	NO, NOT LIQ.	5.0	0.05	0.15	0.00	0.00	0.00	-4.748	0.00	5.00	0.000	0.000	0.00
1.51	55.00	NO, NOT LIQ.	5.0	0.05	0.15	0.00	0.00	0.00	-3.209	0.00	5.00	0.000	0.000	0.00
0.60	60.00	NO, NOT LIQ.	4.0	0.05	1.09	0.00	0.00	0.00	-1.291	0.00	5.00	0.000	0.000	0.00
1.44	65.00	NO, NOT LIQ.	2.5	0.05	0.15	0.00	0.00	0.00	-4.607	0.00	1.50	0.000	0.000	0.00

AGS-18-003 South Ocean Beach Coastal Erosion and Wastewat

Site Data

S	Ground Slope / Liquefiable Layer Slope	0.1%
L	Horizontal Distance from Free Face (ft)	250
H	Free Face Height (ft)	20
Gw	Depth to groundwater (ft)	14.5
Mw	Maximum Moment Magnitude	8.1
PGA	Peak Ground Acceleration	1.02

Boring Data

Depth of boring (ft)	81
Depth to groundwater (ft)	14.5
Borehole diameter (in)	6

Settlement

ft	in	
0.16	1.97	
0.00	0.00	(above YBM)
0.14	1.66	(in upper 50 feet)
0.16	1.97	(in upper 70 feet)

Lateral Displacement

LDI	Sloping		Free Face	
	ft	in	ft	in
1.523	0.3	3.7	1.2	14.5

Sample Depth is to top of sample layer.

Fines content estimated based on soil classification where no test data available.

Soil Type based on Unified Soils Classification System (USCS), ASTM D2487.

Equations and procedures used to calculate liquefaction potential is based on Soil Liquefaction During Earthquakes by Idriss and Boulanger published by EERI MNO-12 2008

Blue cells are input data, orange cells are calculated data.

Seismically-Induced Settlement & Lateral Deformation Calculation - AGS-1989 B-6

SAMPLE DEPTH (feet)	SAMPLE TYPE	UNCORRECTED BLOW COUNT (SPT-N) (blows per foot)	SOIL TYPE	PI Clay/Silt	ESTIMATED FINES CONTENT (%)	GRAVEL? Y=yes N=no	SAMPLER LINER? Y=yes N=no	UNIT WEIGHT OF SOIL LAYER (pcf)	ENERGY RATIO, ER (%)	PORE PRESSURE (psf)	EFFECTIVE OVER-BURDEN (psf)	EFFECTIVE OVER-BURDEN (MPa)	CORRECTION FACTORS: C_b Borehole Size Factor	Cn Depth Reduction Factor	Cm MC Factor	CR Drill Rod Length Factor	Ch Hammer Efficiency Factor	Cs Unlined SPT Factor
5.0	spt	3	SP			y	n	130	75	0	650	31.13	1.15	1.70	1.00	0.75	1.25	1.10
10.0	spt	5	SP			n	n	110	75	0	1200	57.47	1.15	1.40	1.00	0.80	1.25	1.10
15.0	spt	11	SP			n	n	100	75	31.2	1668.8	79.92	1.15	1.12	1.00	0.85	1.25	1.15
20.0	spt	28	SP			n	n	115	75	343.2	1931.8	92.52	1.15	1.03	1.00	0.95	1.25	1.30
25.0	spt	57	SP			n	n	115	75	655.2	2194.8	105.11	1.15	0.99	1.00	0.95	1.25	1.30
30.0	spt	47	SP			n	n	115	75	967.2	2457.8	117.71	1.15	0.96	1.00	0.95	1.25	1.30
35.0	spt	54	SP			n	n	115	75	1279.2	2720.8	130.31	1.15	0.94	1.00	1.00	1.25	1.30
40.0	spt	30	SP			n	n	115	75	1591.2	2983.8	142.90	1.15	0.91	1.00	1.00	1.25	1.30
45.0	spt	61	SP			n	n	115	75	1903.2	3246.8	155.50	1.15	0.89	1.00	1.00	1.25	1.30
50.0	spt	100	SM		12	n	n	115	75	2215.2	3509.8	168.09	1.15	0.87	1.00	1.00	1.25	1.30
55.0	spt	23	SM		51	n	n	115	75	2527.2	3772.8	180.69	1.15	0.82	1.00	1.00	1.25	1.27
60.0	spt	84	SM		12	n	n	110	75	2839.2	4010.8	192.09	1.15	0.84	1.00	1.00	1.25	1.30
70.0	spt	100	SP-SM		5	n	n	105	75	3463.2	4436.8	212.49	1.15	0.82	1.00	1.00	1.25	1.30
80.0	spt	62	SP-SM		5	n	n	110	75	4087.2	4912.8	235.29	1.15	0.80	1.00	1.00	1.25	1.30

SAMPLE DEPTH (feet)	PENETRATION RESISTANCE N60	MODIFIED PENETRATION RESISTANCE (N1)60	δN for Fines Content	MOD PEN RES ADJUSTED TO CLEAN SAND (N1)60CS	STRESS REDUCTION COEFFICIENT (rd)	CYCLIC STRESS RATIO (CSR)	CYCLIC RESISTANCE RATIO (CRR) Sand	CYCLIC RESISTANCE RATIO (CRR) Clay	CRR OVERBURDEN CORRECTION (Ks)	CRR STATIC SHEAR CORRECTION (Ka)	CORRECTED CRR (CCR_R) Sand	CORRECTED CRR (CCR_R) Clay	MAGNITUDE SCALING FACTOR (MSF)	FACTOR OF SAFETY (FS)Clay	FACTOR OF SAFETY (FS)Sand	LIQUEFACTION POTENTIAL	FS
5.0	3.23	5.5	0.0	5.50	1.00	0.66	0.084	0.40	1.09	1.00	0.089	0.44	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00
10.0	5.75	8.1	0.0	8.06	0.99	0.66	0.095	0.36	1.05	1.00	0.105	0.38	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00
15.0	13.44	15.1	0.0	15.12	0.98	0.66	0.139	0.59	1.03	1.00	0.157	0.61	0.86	-	0.211	YES, LIQ.	0.21
20.0	38.24	39.3	0.0	39.29	0.97	0.76	1.778	1.34	1.03	1.00	2.000	1.37	0.86	-	2.000	NO, NOT LIQ.	2.00
25.0	77.84	77.0	0.0	77.03	0.96	0.82	1.713	1.31	0.99	1.00	2.000	1.30	0.86	-	2.000	NO, NOT LIQ.	2.00
30.0	64.18	61.7	0.0	61.65	0.94	0.87	1.655	1.15	0.96	1.00	2.000	1.10	0.86	-	1.901	NO, NOT LIQ.	1.90
35.0	77.63	72.6	0.0	72.59	0.93	0.90	1.604	1.05	0.93	1.00	2.000	0.98	0.86	-	1.773	NO, NOT LIQ.	1.77
40.0	43.13	39.1	0.0	39.14	0.91	0.93	1.557	0.93	0.90	1.00	2.000	0.83	0.86	-	1.678	NO, NOT LIQ.	1.68
45.0	87.69	78.3	0.0	78.28	0.90	0.94	1.513	0.89	0.88	1.00	2.000	0.78	0.86	-	1.605	NO, NOT LIQ.	1.61
50.0	143.75	125.7	2.1	128.41	0.88	0.95	1.474	0.87	0.85	1.00	2.000	0.74	0.86	-	1.549	NO, NOT LIQ.	1.55
55.0	33.06	27.1	5.6	34.23	0.86	0.96	0.703	0.56	0.86	1.00	0.949	0.48	0.86	-	0.736	YES, LIQ.	0.74
60.0	120.75	102.0	2.1	104.65	0.85	0.96	1.406	0.74	0.81	1.00	2.000	0.61	0.86	-	1.468	NO, NOT LIQ.	1.47
70.0	143.75	118.2	0.0	118.20	0.81	0.96	1.354	0.69	0.78	1.00	2.000	0.54	0.86	-	1.414	NO, NOT LIQ.	1.41
80.0	89.13	71.3	0.0	71.35	0.78	0.95	1.302	0.59	0.75	1.00	2.000	0.44	0.86	-	1.378	NO, NOT LIQ.	1.38

Liquefaction Settlement / Lateral Displacement Index and Settlement

SAMPLE DEPTH (feet)	LIQUEFACTION POTENTIAL Incl. chk against PI	Soil Layer Thickness (ft)	Dynamic Vol. Strain (%) Tokimatsu	Dynamic Vol. Strain (%) Ishihara	Liquefaction Settlement (ft) Tokimatsu	Liquefaction Settlement (ft) Ishihara	Limiting Shear Strain gmin	Parameter Fa	Maximum Shear Strain gmax	DH (ft)	DLD_{ii} (ft)	Vertical Reconsol. Strain ev	Liquefaction Settlement (ft) Ishihara
5.00	NO, NOT LIQ.	10.0	4.00	5.50	0.00	0.00	0.50	0.948	0.00	10.00	0.000	0.000	0.00
10.00	NO, NOT LIQ.	5.0	3.00	4.50	0.00	0.00	0.50	0.943	0.00	5.00	0.000	0.000	0.00
15.00	YES, LIQ.	5.0	1.90	2.76	0.10	0.14	0.27	0.750	0.27	5.00	1.358	0.029	0.14
20.00	NO, NOT LIQ.	5.0	0.05	0.00	0.00	0.00	0.01	-0.751	0.00	5.00	0.000	0.000	0.00
25.00	NO, NOT LIQ.	5.0	0.05	0.00	0.00	0.00	0.00	-3.926	0.00	5.00	0.000	0.000	0.00
30.00	NO, NOT LIQ.	5.0	0.05	0.09	0.00	0.00	0.00	-2.565	0.00	5.00	0.000	0.000	0.00
35.00	NO, NOT LIQ.	5.0	0.05	0.09	0.00	0.00	0.00	-3.526	0.00	5.00	0.000	0.000	0.00
40.00	NO, NOT LIQ.	5.0	0.05	0.09	0.00	0.00	0.01	-0.739	0.01	5.00	0.041	0.001	0.01
45.00	NO, NOT LIQ.	5.0	0.05	0.09	0.00	0.00	0.00	-4.039	0.00	5.00	0.000	0.000	0.00
50.00	NO, NOT LIQ.	58.0	0.05	0.15	0.00	0.00	0.00	-8.843	0.00	5.00	0.000	0.000	0.00
55.00	YES, LIQ.	3.0	0.15	0.89	0.00	0.03	0.02	-0.381	0.02	5.00	0.124	0.004	0.02
60.00	NO, NOT LIQ.	7.0	0.05	0.15	0.00	0.00	0.00	-6.514	0.00	10.00	0.000	0.000	0.00
70.00	NO, NOT LIQ.	15.0	0.05	0.15	0.00	0.00	0.00	-7.833	0.00	10.00	0.000	0.000	0.00
80.00	NO, NOT LIQ.	1.0	0.05	0.18	0.00	0.00	0.00	-3.415	0.00	1.00	0.000	0.000	0.00

AGS-18-003 South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection

Site Data

S	Ground Slope / Liquefiable Layer Slope	0.1%
L	Horizontal Distance from Free Face (ft)	3500
H	Free Face Height (ft)	25
Gw	Depth to groundwater (ft)	4
Mw	Maximum Moment Magnitude	8.1
PGA	Peak Ground Acceleration	1.02

Boring Data

Depth of boring (ft)	106.2
Depth to groundwater (ft)	4
Borehole diameter (in)	4.5

Settlement

ft	in	
0.33	3.97	
0.00	0.00	(above YBM)
0.00	0.00	(in upper 50 feet)
0.33	3.97	(in upper 70 feet)

Lateral Displacement

LDI	Sloping		Free Face	
	ft	in	ft	in
0.000	0.0	0.0	0.0	0.0

Sample Depth is to top of sample layer.

Fines content estimated based on soil classification where no test data available.

Soil Type based on Unified Soils Classification System (USCS), ASTM D2487.

Equations and procedures used to calculate liquefaction potential is based on Soil Liquefaction During Earthquakes by Idriss and Boulanger published by EERI MNO-12 2008

Blue cells are input data, orange cells are calculated data.

Table B-1 - Seismically-Induced Settlement & Lateral Deformation Calculation - B-1

SAMPLE DEPTH (feet)	SAMPLE TYPE	UNCORRECTED BLOW COUNT (SPT-N) (blows per foot)	SOIL TYPE	PI Clay/Silt	ESTIMATED FINES CONTENT (%)	GRAVEL? Y=yes N=no	SAMPLER LINER? Y=yes N=no	UNIT WEIGHT OF SOIL LAYER (pcf)	ENERGY RATIO, ER (%)	PORE PRESSURE (psf)	EFFECTIVE OVER-BURDEN (psf)	EFFECTIVE OVER-BURDEN (MPa)
5.0	mc	10	CL	9	65	y	y	120	75	62.4	537.6	25.75
20.0	mc	1	ch	60	99	n	y	96	75	998.4	1041.6	49.89
30.0	mc	1	ch	60	99	n	y	95	75	1622.4	1367.6	65.50
40.0	mc	2	ch	60	99	n	y	94	75	2246.4	1683.6	80.63
50.0	mc	111	sm	2	41	n	y	137	75	2870.4	2427.6	116.26
60.0	spt	12	ml	2	65	n	n	124	75	3494.4	3043.6	145.77
64.0	spt	12	ml	3	85	n	n	119	75	3744	3270	156.61
70.0	spt	14	ch	50	90	n	n	100	75	4118.4	3495.6	167.41
80.0	spt	6	ch	50	90	n	n	100	75	4742.4	3871.6	185.42
90.0	spt	3	ch	50	90	n	n	100	75	5366.4	4247.6	203.43
106	spt	150	sp	0	20	n	n	140	75	6364.8	5489.2	262.89

SAMPLE DEPTH (feet)	CORRECTION FACTORS						PENETRATION RESISTANCE N60	MODIFIED PENETRATION RESISTANCE (N1)60	δN for Fines Content	MOD PEN RES ADJUSTED TO CLEAN SAND (N1)60CS	STRESS REDUCTION COEFFICIENT (rd)	CYCLIC STRESS RATIO (CSR)	CYCLIC RESISTANCE RATIO (CRR) Sand	CYCLIC RESISTANCE RATIO (CRR) Clay	CRR OVERBURDEN CORRECTION (Ks)
	C_b Borehole Size Factor	Cn Depth Reduction Factor	Cm MC Factor	CR Drill Rod Length Factor	Ch Hammer Efficiency Factor	Cs Unlined SPT Factor									
5.0	1.00	1.70	0.61	0.75	1.25	1.10	5.72	9.7	5.6	15.87	1.00	0.74	NA	0.98	1.10
20.0	1.00	1.66	0.61	0.95	1.25	1.10	0.72	1.2	5.5	7.24	0.97	1.26	NA	0.10	1.06
30.0	1.00	1.37	0.61	0.95	1.25	1.10	0.72	1.0	5.5	7.03	0.94	1.37	NA	0.07	1.04
40.0	1.00	1.17	0.61	1.00	1.25	1.10	1.53	1.8	5.5	7.82	0.91	1.41	NA	0.08	1.02
50.0	1.00	0.96	0.61	1.00	1.25	1.30	84.64	81.6	5.6	88.82	0.88	1.27	1.662	1.20	0.96
60.0	1.00	0.84	1.00	1.00	1.25	1.13	15.00	12.5	5.6	18.84	0.85	1.20	NA	0.36	0.95
64.0	1.00	0.81	1.00	1.00	1.25	1.12	15.00	12.1	5.5	18.32	0.83	1.18	NA	0.33	0.95
70.0	1.00	0.79	1.00	1.00	1.25	1.14	17.50	13.9	5.5	20.13	0.81	1.17	NA	0.37	0.93
80.0	1.00	0.71	1.00	1.00	1.25	1.10	7.50	5.3	5.5	11.36	0.78	1.15	NA	0.13	0.94
90.0	1.00	0.64	1.00	1.00	1.25	1.10	3.75	2.4	5.5	8.47	0.75	1.12	NA	0.06	0.94
106	1.00	0.78	1.00	1.00	1.25	1.30	187.50	145.8	4.5	151.60	0.71	1.01	1.246	0.59	0.72

SAMPLE DEPTH (feet)	CRR STATIC SHEAR CORRECTION (Ka)	CORRECTED CRR (CCR_R) Sand	CORRECTED CRR (CCR_R) Clay	MAGNITUDE SCALING FACTOR (MSF)	FACTOR OF SAFETY (FS)Clay	FACTOR OF SAFETY (FS)Sand	LIQUEFACTION POTENTIAL	**FS**
5.0	1.00	NA	1.08	0.86	1.46	NA	NO, NOT LIQ.	**1.46**
20.0	1.00	NA	0.10	0.86	0.08	NA	YES, LIQ.	**0.08**
30.0	1.00	NA	0.08	0.86	0.06	NA	YES, LIQ.	**0.06**
40.0	1.00	NA	0.08	0.86	0.06	NA	YES, LIQ.	**0.06**
50.0	1.00	2.000	1.16	0.86	-	1.305	NO, NOT LIQ.	**1.31**
60.0	1.00	NA	0.34	0.86	-	NA	YES, LIQ.	**0.00**
64.0	1.00	NA	0.31	0.86	-	NA	YES, LIQ.	**0.00**
70.0	1.00	NA	0.34	0.86	0.29	NA	YES, LIQ.	**0.29**
80.0	1.00	NA	0.13	0.86	0.11	NA	YES, LIQ.	**0.11**
90.0	1.00	NA	0.06	0.86	0.05	NA	YES, LIQ.	**0.05**
106	1.00	2.000	0.43	0.86	-	1.229	NO, NOT LIQ.	**1.23**

Liquefaction Settlement

SAMPLE DEPTH (feet)	LIQUEFACTION POTENTIAL Incl. chk against PI	Soil Layer Thickness (ft)	Dynamic Vol. Strain (%)		Liquefaction Settlement (ft)	
			Tokimatsu	Ishihara	Tokimatsu	Ishihara
5.00	NO, NOT LIQ.	8.5	1.90	0.27	0.00	0.00
20.00	NO, NOT LIQ.	12.5	3.00	4.50	0.00	0.00
30.00	NO, NOT LIQ.	10.0	3.00	4.50	0.00	0.00
40.00	NO, NOT LIQ.	10.0	3.00	4.50	0.00	0.00
50.00	NO, NOT LIQ.	10.0	0.05	0.18	0.00	0.00
60.00	YES, LIQ	7.0	1.70	2.76	0.12	0.19
64.00	YES, LIQ	5.0	1.70	2.76	0.09	0.14
70.00	NO, NOT LIQ.	8.0	1.40	2.15	0.00	0.00
80.00	NO, NOT LIQ.	10.0	3.00	3.50	0.00	0.00
90.00	NO, NOT LIQ.	13.0	3.00	4.50	0.00	0.00
106.00	NO, NOT LIQ.	8.2	0.05	0.26	0.00	0.00

Lateral Displacement Index and Settlement

SAMPLE DEPTH (feet)	Limiting Shear Strain gmin	Parameter Fa	Maximum Shear Strain gmax	DH (ft)	DLD_i (ft)	Vertical Reconsol. Strain ev	Liquefaction Settlement (ft) Ishihara
5.0	0.25	0.718	0.00	12.50	0.000	0.000	0.00
20.0	0.50	0.947	0.00	12.50	0.000	0.000	0.00
30.0	0.50	0.948	0.00	10.00	0.000	0.000	0.00
40.0	0.50	0.945	0.00	10.00	0.000	0.000	0.00
50.0	0.00	-5.012	0.00	10.00	0.000	0.000	0.00
60.0	0.18	0.578	0.00	7.00	0.000	0.000	0.00
64.0	0.19	0.604	0.00	5.00	0.000	0.000	0.00
70.0	0.16	0.511	0.00	8.00	0.000	0.000	0.00
80.0	0.41	0.881	0.00	10.00	0.000	0.000	0.00
90.0	0.50	0.939	0.00	13.00	0.000	0.000	0.00
106	0.00	-11.180	0.00	8.20	0.000	0.000	0.00

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Site Data

S	Ground Slope / Liquefiable Layer Slope	0.1%
L	Horizontal Distance from Free Face (ft)	3500
H	Free Face Height (ft)	25
Gw	Depth to groundwater (ft)	4
Mw	Maximum Moment Magnitude	8.1
PGA	Peak Ground Acceleration	1.02

Boring Data

Depth of boring (ft)	108.3
Depth to groundwater (ft)	4
Borehole diameter (in)	4.5

Settlement

ft	in	
0.30	3.58	
0.15	1.82	(above YBM)
0.15	1.82	(in upper 50 feet)
0.30	3.58	(in upper 70 feet)

Lateral Displacement

LDI	Sloping		Free Face	
	ft	in	ft	in
2.374	0.5	5.7	0.3	3.3

Sample Depth is to top of sample layer.

Fines content estimated based on soil classification where no test data available.

Soil Type based on Unified Soils Classification System (USCS), ASTM D2487.

Equations and procedures used to calculate liquefaction potential is based on Soil Liquefaction During Earthquakes by Idriss and Boulanger published by EERI MNO-12 2008

Blue cells are input data, orange cells are calculated data.

Table B-2 - Seismically-Induced Settlement & Lateral Deformation Calculation - B-2

SAMPLE DEPTH (feet)	SAMPLE TYPE	UNCORRECTED BLOW COUNT (SPT-N) (blows per foot)	SOIL TYPE	PI Clay/Silt	ESTIMATED FINES CONTENT (%)	GRAVEL? Y=yes N=no	SAMPLER LINER? Y=yes N=no	UNIT WEIGHT OF SOIL LAYER (pcf)	ENERGY RATIO, ER (%)	PORE PRESSURE (psf)	EFFECTIVE OVER-BURDEN (psf)	EFFECTIVE OVER-BURDEN (MPa)	CORRECTION FACTORS: C_b Borehole Size Factor	Cn Depth Reduction Factor	Cm MC Factor	CR Drill Rod Length Factor	Ch Hammer Efficiency Factor	Cs Unlined SPT Factor	PENETRATION RESISTANCE N60
5.0	mc	10	sc	9	45	y	y	120	75	62.4	537.6	25.75	1.00	1.70	0.61	0.75	1.25	1.10	5.72
20.0	mc	1	ch	60	99	n	y	96	75	998.4	1041.6	49.89	1.00	1.66	0.61	0.95	1.25	1.10	0.72
35.0	mc	6	ch	60	99	n	y	95	75	1934.4	1530.6	73.30	1.00	1.22	0.61	1.00	1.25	1.10	4.58
48.0	mc	69	sm	20	30	n	y	138	75	2745.6	2513.4	120.37	1.00	0.95	0.61	1.00	1.25	1.30	52.61
55.0	spt	10	ch	20	90	n	n	100	75	3182.4	2776.6	132.98	1.00	0.87	1.00	1.00	1.25	1.11	12.50
61.0	mc	35	ml	13	64	n	n	124	75	3556.8	3146.2	150.68	1.00	0.86	0.61	1.00	1.25	1.23	26.69
70.0	mc	26	ch	50	90	n	n	100	75	4118.4	3484.6	166.89	1.00	0.80	0.61	1.00	1.25	1.16	19.83
80.0	mc	13	ch	50	90	n	n	100	75	4742.4	3860.6	184.89	1.00	0.72	0.61	1.00	1.25	1.10	9.91
90.0	spt	8	ch	50	90	n	n	100	75	5366.4	4236.6	202.90	1.00	0.69	1.00	1.00	1.25	1.10	10.00
100.0	spt	25	ch	50	90	n	n	100	75	5990.4	4612.6	220.91	1.00	0.76	1.00	1.00	1.25	1.24	31.25
107.0	spt	110	sp	0	20	n	n	140	75	6427.2	5155.8	246.93	1.00	0.79	1.00	1.00	1.25	1.30	137.50

SAMPLE DEPTH (feet)	MODIFIED PENETRATION RESISTANCE (N1)60	δN for Fines Content	MOD PEN RES ADJUSTED TO CLEAN SAND (N1)60CS	STRESS REDUCTION COEFFICIENT (rd)	CYCLIC STRESS RATIO (CSR)	CYCLIC RESISTANCE RATIO (CRR) Sand	CYCLIC RESISTANCE RATIO (CRR) Clay	CRR OVERBURDEN CORRECTION (Ks)	CRR STATIC SHEAR CORRECTION (Ka)	CORRECTED CRR (CCR_R) Sand	CORRECTED CRR (CCR_R) Clay	MAGNITUDE SCALING FACTOR (MSF)	FACTOR OF SAFETY (FS)Clay	FACTOR OF SAFETY (FS)Sand	LIQUEFACTION POTENTIAL	FS
5.0	9.7	5.6	15.89	1.00	0.74	0.156	0.98	1.10	1.00	0.164	1.08	0.86	-	0.211	YES, LIQ.	0.21
20.0	1.2	5.5	7.24	0.97	1.26	NA	0.10	1.06	1.00	NA	0.10	0.86	0.08	NA	YES, LIQ.	0.08
35.0	5.6	5.5	11.62	0.93	1.39	NA	0.21	1.03	1.00	NA	0.21	0.86	0.15	NA	YES, LIQ.	0.15
48.0	50.2	5.4	57.21	0.89	1.23	1.644	1.12	0.95	1.00	2.000	1.07	0.86	-	1.337	NO, NOT LIQ.	1.34
55.0	10.9	5.5	16.97	0.86	1.23	NA	0.32	0.97	1.00	NA	0.31	0.86	0.25	NA	YES, LIQ.	0.25
61.0	22.9	5.6	29.73	0.84	1.19	NA	0.63	0.92	1.00	NA	0.58	0.86	0.48	NA	YES, LIQ.	0.48
70.0	15.9	5.5	22.27	0.81	1.17	NA	0.42	0.93	1.00	NA	0.39	0.86	0.33	NA	YES, LIQ.	0.33
80.0	7.1	5.5	13.21	0.78	1.15	NA	0.18	0.94	1.00	NA	0.17	0.86	0.14	NA	YES, LIQ.	0.14
90.0	6.9	5.5	12.92	0.75	1.12	NA	0.16	0.93	1.00	NA	0.15	0.86	0.13	NA	YES, LIQ.	0.13
100.0	23.7	5.5	30.50	0.72	1.10	NA	0.50	0.84	1.00	NA	0.42	0.86	0.38	NA	YES, LIQ.	0.38
107.0	108.7	4.5	114.50	0.71	1.05	1.278	0.60	0.74	1.00	2.000	0.44	0.86	-	1.215	NO, NOT LIQ.	1.21

Liquefaction Settlement: SAMPLE DEPTH (feet)	LIQUEFACTION POTENTIAL Incl. chk against PI	Soil Layer Thickness (ft)	Dynamic Vol. Strain (%): Tokimatsu	Dynamic Vol. Strain (%): Ishihara	Liquefaction Settlement (ft): Tokimatsu	Liquefaction Settlement (ft): Ishihara	Lateral Displacement Index and Settlement: Limiting Shear Strain gmin	Parameter Fa	Maximum Shear Strain gmax	DH (ft)	DLD_s (ft)	Vertical Reconsol. Strain ev	Liquefaction Settlement (ft) Ishihara
5.00	YES, LIQ.	5.5	1.90	2.76	0.10	0.15	0.25	0.717	0.25	9.50	2.374	0.028	0.26
20.00	NO, NOT LIQ.	18.0	3.00	4.50	0.00	0.00	0.50	0.947	0.00	18.00	0.000	0.000	0.00
35.00	NO, NOT LIQ.	18.5	3.00	3.50	0.00	0.00	0.40	0.874	0.00	18.50	0.000	0.000	0.00
48.00	NO, NOT LIQ.	7.0	0.05	0.18	0.00	0.00	0.00	-2.186	0.00	7.00	0.000	0.000	0.00
55.00	NO, NOT LIQ.	7.5	1.90	2.76	0.00	0.00	0.22	0.668	0.00	7.50	0.000	0.000	0.00
61.00	YES, LIQ	8.5	1.00	1.72	0.09	0.15	0.05	-0.071	0.00	8.50	0.000	0.000	0.00
70.00	NO, NOT LIQ.	6.0	1.20	2.15	0.00	0.00	0.12	0.393	0.00	6.00	0.000	0.000	0.00
80.00	NO, NOT LIQ.	10.0	2.20	3.50	0.00	0.00	0.33	0.823	0.00	10.00	0.000	0.000	0.00
90.00	NO, NOT LIQ.	10.0	2.20	3.50	0.00	0.00	0.34	0.833	0.00	10.00	0.000	0.000	0.00
100.00	NO, NOT LIQ.	9.0	0.55	1.30	0.00	0.00	0.04	-0.122	0.00	9.00	0.000	0.000	0.00
107.00	NO, NOT LIQ.	4.3	0.05	0.26	0.00	0.00	0.00	-7.470	0.00	4.30	0.000	0.000	0.00

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AGS

Site Data

S	Ground Slope / Liquefiable Layer Slope	0.1%
L	Horizontal Distance from Free Face (ft)	3500
H	Free Face Height (ft)	25
Gw	Depth to groundwater (ft)	4
Mw	Maximum Moment Magnitude	8.1
PGA	Peak Ground Acceleration	1.02

Boring Data

Depth of boring (ft)	108.3
Depth to groundwater (ft)	4
Borehole diameter (in)	4.5

Settlement

ft	in	
0.92	11.09	
0.38	4.59	(above YBM)
0.38	4.59	(in upper 50 feet)
0.78	9.41	(in upper 70 feet)

Lateral Displacement

LDI	Sloping		Free Face	
	ft	in	ft	in
6.000	1.2	14.5	0.7	8.3

Sample Depth is to top of sample layer.

Fines content estimated based on soil classification where no test data available.

Soil Type based on Unified Soils Classification System (USCS), ASTM D2487.

Equations and procedures used to calculate liquefaction potential is based on Soil Liquefaction During Earthquakes by Idriss and Boulanger published by EERI MNO-12 2008

Blue cells are input data, orange cells are calculated data.

Table B-3 - Seismically-Induced Settlement & Lateral Deformation Calculation - B-3

SAMPLE DEPTH (feet)	SAMPLE TYPE	UNCORRECTED BLOW COUNT (SPT-N) (blows per foot)	SOIL TYPE	PI Clay/Silt	ESTIMATED FINES CONTENT (%)	GRAVEL? Y=yes N=no	SAMPLER LINER? Y=yes N=no	UNIT WEIGHT OF SOIL LAYER (pcf)	ENERGY RATIO, ER (%)	PORE PRESSURE (psf)	EFFECTIVE OVER-BURDEN (psf)	EFFECTIVE OVER-BURDEN (MPa)	CORRECTION FACTORS: C_b Borehole Size Factor	CORRECTION FACTORS: Cn Depth Reduction Factor	CORRECTION FACTORS: Cm MC Factor	CORRECTION FACTORS: CR Drill Rod Length Factor	CORRECTION FACTORS: Ch Hammer Efficiency Factor	CORRECTION FACTORS: Cs Unlined SPT Factor
5.0	mc	10	gm	0	7	y	y	120	75	62.4	537.6	25.75	1.00	1.70	0.61	0.75	1.25	1.10
20.0	mc	1	ch	60	99	n	y	96	75	998.4	1041.6	49.89	1.00	1.66	0.61	0.95	1.25	1.10
40.0	spt	1	ch	60	99	n	n	95	75	2246.4	1693.6	81.11	1.00	1.17	1.00	1.00	1.25	1.10
50.0	spt	12	cl	11	55	n	n	136	75	2870.4	2429.6	116.36	1.00	0.93	1.00	1.00	1.25	1.14
55.0	spt	4	cl	17	98	n	n	124	75	3182.4	2737.6	131.11	1.00	0.85	1.00	1.00	1.25	1.10
60.0	spt	26	ml	0	69	n	n	123	75	3494.4	3040.6	145.62	1.00	0.88	1.00	1.00	1.25	1.29
65.0	spt	11	ch	50	90	n	n	100	75	3806.4	3228.6	154.63	1.00	0.81	1.00	1.00	1.25	1.11
75.0	spt	4	ch	50	90	n	n	100	75	4430.4	3604.6	172.63	1.00	0.72	1.00	1.00	1.25	1.10
85.0	spt	8	ch	50	90	n	n	100	75	5054.4	3980.6	190.64	1.00	0.71	1.00	1.00	1.25	1.10
95.0	spt	12	ch	50	90	n	n	100	75	5678.4	4356.6	208.65	1.00	0.70	1.00	1.00	1.25	1.11
100.0	spt	92	cl	18	50	n	n	140	75	5990.4	4744.6	227.23	1.00	0.81	1.00	1.00	1.25	1.30
105.0	spt	142	ml	12	50	n	n	140	75	6302.4	5132.6	245.81	1.00	0.79	1.00	1.00	1.25	1.30

SAMPLE DEPTH (feet)	PENETRATION RESISTANCE N60	MODIFIED PENETRATION RESISTANCE (N1)60	δN for Fines Content	MOD PEN RES ADJUSTED TO CLEAN SAND (N1)60CS	STRESS REDUCTION COEFFICIENT (rd)	CYCLIC STRESS RATIO (CSR)	CYCLIC RESISTANCE RATIO (CRR) Sand	CYCLIC RESISTANCE RATIO (CRR) Clay	CRR OVERBURDEN CORRECTION (Ks)	CRR STATIC SHEAR CORRECTION (Ka)	CORRECTED CRR (CCR_R) Sand	CORRECTED CRR (CCR_R) Clay	MAGNITUDE SCALING FACTOR (MSF)	FACTOR OF SAFETY (FS)Clay	FACTOR OF SAFETY (FS)Sand	LIQUEFACTION POTENTIAL	FS
5.0	5.72	9.7	0.1	9.87	1.00	0.74	0.111	0.98	1.10	1.00	0.117	1.08	0.86	-	0.151	YES, LIQ.	0.15
20.0	0.72	1.2	5.5	7.24	0.97	1.26	NA	0.10	1.06	1.00	NA	0.10	0.86	0.08	NA	YES, LIQ.	0.08
40.0	1.25	1.5	5.5	7.50	0.91	1.41	NA	0.07	1.02	1.00	NA	0.07	0.86	0.05	NA	YES, LIQ.	0.05
50.0	15.00	14.0	5.6	20.40	0.88	1.27	NA	0.45	0.98	1.00	NA	0.44	0.86	0.35	NA	YES, LIQ.	0.35
55.0	5.00	4.3	5.5	10.30	0.86	1.24	NA	0.13	0.98	1.00	NA	0.12	0.86	0.10	NA	YES, LIQ.	0.10
60.0	32.50	28.6	5.6	35.81	0.85	1.20	NA	0.79	0.90	1.00	NA	0.71	0.86	-	NA	YES, LIQ.	0.00
65.0	13.75	11.1	5.5	17.24	0.83	1.20	NA	0.31	0.95	1.00	NA	0.29	0.86	0.24	NA	YES, LIQ.	0.24
75.0	5.00	3.6	5.5	9.67	0.79	1.17	NA	0.10	0.95	1.00	NA	0.09	0.86	0.08	NA	YES, LIQ.	0.08
85.0	10.00	7.1	5.5	13.16	0.76	1.15	NA	0.17	0.94	1.00	NA	0.16	0.86	0.14	NA	YES, LIQ.	0.14
95.0	15.00	10.5	5.5	16.63	0.73	1.12	NA	0.25	0.92	1.00	NA	0.23	0.86	0.20	NA	YES, LIQ.	0.20
100.0	115.00	92.9	5.6	100.20	0.72	1.08	NA	0.64	0.76	1.00	NA	0.49	0.86	0.45	NA	YES, LIQ.	0.45
105.0	177.50	140.5	5.6	147.76	0.71	1.05	NA	0.63	0.74	1.00	NA	0.46	0.86	0.44	NA	YES, LIQ.	0.44

Liquefaction Settlement: SAMPLE DEPTH (feet)	LIQUEFACTION POTENTIAL Incl. chk against PI	Soil Layer Thickness (ft)	Dynamic Vol. Strain (%): Tokimatsu	Dynamic Vol. Strain (%): Ishihara	Liquefaction Settlement (ft): Tokimatsu	Liquefaction Settlement (ft): Ishihara	Lateral Displacement Index and Settlement: Limiting Shear Strain gmin	Parameter Fa	Maximum Shear Strain gmax	DH (ft)	DLD_i (ft)	Vertical Reconsol. Strain ev	Liquefaction Settlement (ft) Ishihara
5.00	YES, LIQ.	8.5	3.00	4.50	0.26	0.38	0.48	0.917	0.48	12.50	6.000	0.038	0.47
20.00	NO, NOT LIQ.	17.5	3.00	4.50	0.00	0.00	0.50	0.947	0.00	17.50	0.000	0.000	0.00
40.00	NO, NOT LIQ.	15.0	3.00	4.50	0.00	0.00	0.50	0.947	0.00	15.00	0.000	0.000	0.00
50.00	YES, LIQ	7.5	1.40	2.15	0.11	0.16	0.15	0.497	0.00	7.50	0.000	0.000	0.00
55.00	YES, LIQ	5.0	3.00	3.50	0.15	0.18	0.46	0.907	0.00	5.00	0.000	0.000	0.00
60.00	YES, LIQ	5.0	0.05	1.30	0.00	0.07	0.02	-0.494	0.00	5.00	0.000	0.000	0.00
65.00	NO, NOT LIQ.	7.5	1.70	2.76	0.00	0.00	0.22	0.656	0.00	7.50	0.000	0.000	0.00
75.00	NO, NOT LIQ.	10.0	3.00	4.50	0.00	0.00	0.49	0.921	0.00	10.00	0.000	0.000	0.00
85.00	NO, NOT LIQ.	10.0	2.20	3.50	0.00	0.00	0.34	0.824	0.00	10.00	0.000	0.000	0.00
95.00	NO, NOT LIQ.	7.5	1.90	2.76	0.00	0.00	0.23	0.684	0.00	7.50	0.000	0.000	0.00
100.00	YES, LIQ	5.0	0.05	1.30	0.00	0.07	0.00	-6.088	0.00	5.00	0.000	0.000	0.00
105.00	YES, LIQ	5.8	0.05	1.30	0.00	0.08	0.00	-10.790	0.00	5.80	0.000	0.000	0.00

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Site Data

S	Ground Slope / Liquefiable Layer Slope	0.1%
L	Horizontal Distance from Free Face (ft)	80
H	Free Face Height (ft)	20
Gw	Depth to groundwater (ft)	13.9
Mw	Maximum Moment Magnitude	8.1
PGA	Peak Ground Acceleration	1.02

Boring Data

Depth of boring (ft)	102
Depth to groundwater (ft)	14.0
Borehole diameter (in)	6

Settlement

ft	in	
0.09	1.03	
0.00	0.00	(above YBM)
0.09	1.03	(in upper 50 feet)
0.09	1.03	(in upper 70 feet)

Lateral Displacement

LDI	Sloping		Free Face	
	ft	in	ft	in
0.416	0.1	1.0	0.8	9.9

Sample Depth is to top of sample layer.
Fines content estimated based on soil classification where no test data available.
Soil Type based on Unified Soils Classification System (USCS), ASTM D2487.
Equations and procedures used to calculate liquefaction potential is based on Soil Liquefaction During Earthquakes by Idriss and Boulanger published by EERI MNO-12 2008
Blue cells are input data, orange cells are calculated data.

Table B-4 - Seismically-Induced Settlement & Lateral Deformation Calculation - B-4

SAMPLE DEPTH (feet)	SAMPLE TYPE	UNCORRECTED BLOW COUNT (SPT-N) (blows per foot)	SOIL TYPE	PI Clay/Silt	ESTIMATED FINES CONTENT (%)	GRAVEL? Y=yes N=no	SAMPLER LINER? Y=yes N=no	UNIT WEIGHT OF SOIL LAYER (pcf)	ENERGY RATIO, ER (%)	PORE PRESSURE (psf)	EFFECTIVE OVER-BURDEN (psf)	EFFECTIVE OVER-BURDEN (MPa)	CORRECTION FACTORS: C_b Borehole Size Factor	Cn Depth Reduction Factor	Cm MC Factor	CR Drill Rod Length Factor	Ch Hammer Efficiency Factor	Cs Unlined SPT Factor	PENETRATION RESISTANCE N60
2.5	spt	14	SP			y	n	130	85	0	325	15.57	1.15	1.70	1.00	0.75	1.42	1.29	17.11
5.0	spt	9	SP		2	y	n	120	85	0	625	29.93	1.15	1.70	1.00	0.75	1.42	1.19	11.00
10.0	spt	2	SP		1	n	n	120	85	0	1225	58.67	1.15	1.43	1.00	0.80	1.42	1.10	2.61
13.0	spt	5	SP			n	n	115	85	0	1570	75.19	1.15	1.19	1.00	0.85	1.42	1.10	6.92
15.0	spt	17	SP			n	n	115	85	68.64	1731.36	82.92	1.15	1.08	1.00	0.85	1.42	1.26	23.54
20.0	spt	32	SP		5	n	n	115	85	380.64	1994.36	95.52	1.15	1.01	1.00	0.95	1.42	1.30	49.53
25.0	spt	33	SP-SM		3	n	n	115	85	692.64	2257.36	108.11	1.15	0.98	1.00	0.95	1.42	1.30	51.07
30.0	spt	46	SP-SM		4	n	n	115	85	1004.64	2520.36	120.71	1.15	0.95	1.00	0.95	1.42	1.30	71.19
32.0	spt	46	SP-SM		5	n	n	115	85	1129.44	2625.56	125.75	1.15	0.94	1.00	1.00	1.42	1.30	74.94
35.0	spt	53	SP-SM		5	n	n	115	85	1316.64	2783.36	133.30	1.15	0.93	1.00	1.00	1.42	1.30	86.35
40.0	spt	80	SP-SM		5	y	n	115	85	1628.64	3046.36	145.90	1.15	0.91	1.00	1.00	1.42	1.30	130.33
45.0	spt	61	SP-SM		5	n	n	115	85	1940.64	3309.36	158.49	1.15	0.89	1.00	1.00	1.42	1.30	99.38
50.0	spt	72	SP-SM		8	n	n	115	85	2252.64	3572.36	171.09	1.15	0.87	1.00	1.00	1.42	1.30	117.30
60.0	spt	86	SP-SM		5	n	n	115	85	2876.64	4098.36	196.28	1.15	0.84	1.00	1.00	1.42	1.30	140.11
62.0	spt	50	SP-SM		5	n	n	115	85	3001.44	4203.56	201.32	1.15	0.83	1.00	1.00	1.42	1.30	81.46
70.0	spt	112	SP-SM		6	n	n	115	85	3500.64	4624.36	221.47	1.15	0.81	1.00	1.00	1.42	1.30	182.47
80.0	spt	27	SM		51	n	n	115	85	4124.64	5150.36	246.66	1.15	0.78	1.00	1.00	1.42	1.30	43.99
100.0	spt	15	CH	20	90	n	n	115	85	5372.64	6202.36	297.05	1.15	0.65	1.00	1.00	1.42	1.16	24.44

SAMPLE DEPTH (feet)	MODIFIED PENETRATION RESISTANCE (N1)60	Δ(N1) for Fines Content	MOD PEN RES ADJUSTED TO CLEAN SAND (N1)60CS	STRESS REDUCTION COEFFICIENT (rd)	CYCLIC STRESS RATIO (CSR)	CYCLIC RESISTANCE RATIO (CRR) Sand	CYCLIC RESISTANCE RATIO (CRR) Clay	CRR OVERBURDEN CORRECTION (Kσ)	CRR STATIC SHEAR CORRECTION (Kα)	CORRECTED CRR (CCRs) Sand	CORRECTED CRR (CCRc) Clay	MAGNITUDE SCALING FACTOR (MSF)	FACTOR OF SAFETY (FS)Clay	FACTOR OF SAFETY (FS)Sand	LIQUEFACTION POTENTIAL	FS
2.5	29.1	0.0	29.08	1.00	0.66	0.412	3.85	1.10	1.00	0.433	4.35	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00
5.0	18.7	0.0	18.69	1.00	0.66	0.182	1.26	1.10	1.00	0.191	1.39	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00
10.0	3.7	0.0	3.73	0.99	0.66	0.071	0.14	1.04	1.00	0.079	0.15	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00
13.0	8.2	0.0	8.22	0.98	0.65	0.094	0.29	1.03	1.00	0.106	0.26	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00
15.0	25.5	0.0	25.53	0.98	0.68	0.271	0.91	1.03	1.00	0.303	0.94	0.86	-	0.401	YES, LIQ.	0.40
20.0	50.3	0.0	50.26	0.97	0.76	1.762	1.38	1.02	1.00	2.000	1.42	0.86	-	2.000	NO, NOT LIQ.	2.00
25.0	50.2	0.0	50.17	0.96	0.83	1.699	1.23	0.98	1.00	2.000	1.21	0.86	-	2.000	NO, NOT LIQ.	2.00
30.0	67.9	0.0	67.93	0.94	0.87	1.643	1.12	0.95	1.00	2.000	1.07	0.86	-	1.879	NO, NOT LIQ.	1.88
32.0	70.7	0.0	70.75	0.94	0.89	1.622	1.09	0.94	1.00	2.000	1.01	0.86	-	1.826	NO, NOT LIQ.	1.83
35.0	80.3	0.0	80.27	0.93	0.91	1.592	1.03	0.92	1.00	2.000	0.95	0.86	-	1.757	NO, NOT LIQ.	1.76
40.0	118.3	0.0	118.31	0.91	0.93	1.546	0.97	0.89	1.00	2.000	0.87	0.86	-	1.665	NO, NOT LIQ.	1.66
45.0	89.3	0.0	89.27	0.90	0.94	1.504	0.87	0.87	1.00	2.000	0.76	0.86	-	1.595	NO, NOT LIQ.	1.59
50.0	102.1	0.4	102.59	0.89	0.95	1.465	0.82	0.85	1.00	2.000	0.70	0.86	-	1.540	NO, NOT LIQ.	1.54
60.0	117.6	0.0	117.64	0.85	0.95	1.395	0.73	0.81	1.00	2.000	0.59	0.86	-	1.462	NO, NOT LIQ.	1.46
62.0	67.9	0.0	67.94	0.84	0.95	1.382	0.68	0.80	1.00	2.000	0.54	0.86	-	1.450	NO, NOT LIQ.	1.45
70.0	148.4	0.0	148.45	0.81	0.95	1.333	0.67	0.77	1.00	2.000	0.52	0.86	-	1.411	NO, NOT LIQ.	1.41
80.0	34.4	5.6	41.72	0.78	0.93	1.278	0.46	0.74	1.00	2.000	0.36	0.86	-	1.376	NO, NOT LIQ.	1.38
100.0	15.8	5.5	22.18	0.72	0.88	NA	0.22	0.64	1.00	NA	0.19	0.86	0.21	NA	YES, LIQ.	0.21

Liquefaction Settlement / Lateral Displacement Index and Settlement

SAMPLE DEPTH (feet)	LIQUEFACTION POTENTIAL	Soil Layer Thickness (ft)	Dynamic Vol. Strain (%) Tokimatsu	Dynamic Vol. Strain (%) Ishihara	Liquefaction Settlement (ft) Tokimatsu	Liquefaction Settlement (ft) Ishihara	Limiting Shear Strain γmin	Parameter Fα	Maximum Shear Strain γmax	DH (ft)	DLD (ft)		Vertical Reconsol. Strain εv	Liquefaction Settlement (ft) Ishihara
2.50	NO, NOT LIQ.	5.0	1.00	1.72	0.00	0.00	0.05	-0.028	0.00	5.00	0.000	0.000	0.000	0.00
5.00	NO, NOT LIQ.	5.0	1.70	2.78	0.00	0.00	0.18	0.585	0.00	5.00	0.000	0.000	0.000	0.00
10.00	NO, NOT LIQ.	3.0	7.00	5.50	0.00	0.00	0.50	0.948	0.00	3.00	0.000	0.000	0.000	0.00
13.00	NO, NOT LIQ.	2.0	2.00	4.50	0.00	0.00	0.50	0.942	0.00	2.00	0.000	0.000	0.000	0.00
15.00	YES, LIQ.	5.0	1.10	1.72	0.06	0.09	0.09	0.199	0.08	5.00	0.416	0.416	0.019	0.09
20.00	NO, NOT LIQ.	5.0	0.05	0.00	0.00	0.00	0.00	-1.810	0.00	5.00	0.000	0.416	0.000	0.00
25.00	NO, NOT LIQ.	5.0	0.05	0.00	0.00	0.00	0.00	-1.802	0.00	5.00	0.000	0.000	0.000	0.00
30.00	NO, NOT LIQ.	2.0	0.05	0.09	0.00	0.00	0.00	-3.112	0.00	2.00	0.000	0.000	0.000	0.00
32.00	NO, NOT LIQ.	3.0	0.05	0.09	0.00	0.00	0.00	-3.361	0.00	3.00	0.000	0.000	0.000	0.00
35.00	NO, NOT LIQ.	5.0	0.05	0.09	0.00	0.00	0.00	-4.221	0.00	5.00	0.000	0.000	0.000	0.00
40.00	NO, NOT LIQ.	5.0	0.05	0.09	0.00	0.00	0.00	-7.844	0.00	5.00	0.000	0.000	0.000	0.00
45.00	NO, NOT LIQ.	5.0	0.05	0.15	0.00	0.00	0.00	-4.960	0.00	5.00	0.000	0.000	0.000	0.00
50.00	NO, NOT LIQ.	10.0	0.05	0.15	0.00	0.00	0.00	-6.316	0.00	10.00	0.000	0.000	0.000	0.00
60.00	NO, NOT LIQ.	2.0	0.05	0.15	0.00	0.00	0.00	-7.777	0.00	2.00	0.000	0.000	0.000	0.00
62.00	NO, NOT LIQ.	8.0	0.05	0.15	0.00	0.00	0.00	-3.113	0.00	8.00	0.000	0.000	0.000	0.00
70.00	NO, NOT LIQ.	10.0	0.05	0.15	0.00	0.00	0.00	-10.859	0.00	10.00	0.000	0.000	0.000	0.00
80.00	NO, NOT LIQ.	20.0	0.05	0.18	0.00	0.00	0.01	-0.835	0.01	20.00	0.120	0.000	0.001	0.02
100.00	NO, NOT LIQ.	2.0	1.20	2.15	0.00	0.00	0.12	0.398	0.00	2.00	0.000	0.000	0.000	0.00

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Site Data

S	Ground Slope / Liquefiable Layer Slope	0.1%
L	Horizontal Distance from Free Face (ft)	180
H	Free Face Height (ft)	20
Gw	Depth to groundwater (ft)	14.9
Mw	Maximum Moment Magnitude	8.1
PGA	Peak Ground Acceleration	1.02

Boring Data

Depth of boring (ft)	51.5
Depth to groundwater (ft)	15.0
Borehole diameter (in)	6

Settlement

ft	in	
0.00	0.00	
0.00	0.00	(above YBM)
0.00	0.00	(in upper 50 feet)
0.00	0.00	(in upper 70 feet)

Lateral Displacement

LDI	Sloping		Free Face	
	ft	in	ft	in
0.000	0.0	0.0	0.0	0.0

Sample Depth is to top of sample layer.

Fines content estimated based on soil classification where no test data available.

Soil Type based on Unified Soils Classification System (USCS), ASTM D2487.

Equations and procedures used to calculate liquefaction potential is based on Soil Liquefaction During Earthquakes by Idriss and Boulanger published by EERI MNO-12 2008

Blue cells are input data, orange cells are calculated data.

Table B-5 - Seismically-Induced Settlement & Lateral Deformation Calculation - B-5

SAMPLE DEPTH (feet)	SAMPLE TYPE	UNCORRECTED BLOW COUNT (SPT-N) (blows per foot)	SOIL TYPE	PI Clay/Silt	ESTIMATED FINES CONTENT (%)	GRAVEL? Y=yes N=no	SAMPLER LINER? Y=yes N=no	UNIT WEIGHT OF SOIL LAYER (pcf)	ENERGY RATIO, ER (%)	PORE PRESSURE (psf)	EFFECTIVE OVER-BURDEN (psf)	EFFECTIVE OVER-BURDEN (MPa)	CORRECTION FACTORS: C_b Borehole Size Factor	Cn Depth Reduction Factor	Cm MC Factor	CR Drill Rod Length Factor	Ch Hammer Efficiency Factor	Cs Unlined SPT Factor	PENETRATION RESISTANCE N60	MODIFIED PENETRATION RESISTANCE (N1)60	δN for Fines Content	MOD PEN RES ADJUSTED TO CLEAN SAND (N1)60CS	STRESS REDUCTION COEFFICIENT (rd)	CYCLIC STRESS RATIO (CSR)	CYCLIC RESISTANCE RATIO (CRR) Sand	CYCLIC RESISTANCE RATIO (CRR) Clay	CRR OVERBURDEN CORRECTION (Ks)
0.5	spt	36	SP			y	n	130	85	0	65	3.11	1.15	1.70	1.00	0.75	1.42	1.30	43.99	74.8	0.0	74.78	1.00	0.67	1.901	42.92	1.10
2.0	spt	22	SM		20	n	n	120	85	0	245	11.73	1.15	1.70	1.00	0.75	1.42	1.30	26.88	45.7	4.5	51.52	1.00	0.66	1.901	8.28	1.10
3.5	spt	16	SM		17	n	n	120	85	0	425	20.35	1.15	1.70	1.00	0.75	1.42	1.30	19.55	33.2	3.9	38.24	1.00	0.66	1.901	3.47	1.10
5.0	spt	16	SM		17	n	n	115	85	0	597.5	28.62	1.15	1.70	1.00	0.75	1.42	1.30	19.55	33.2	3.9	38.24	1.00	0.66	1.901	2.47	1.10
6.5	spt	12	SM		12	n	n	115	85	0	770	36.88	1.15	1.62	1.00	0.80	1.42	1.25	15.64	25.4	2.1	27.97	1.00	0.66	0.364	1.41	1.10
8.0	spt	17	SM		12	n	n	115	85	0	942.5	45.14	1.15	1.41	1.00	0.80	1.42	1.30	22.16	31.1	2.1	33.83	0.99	0.66	0.837	1.66	1.10
9.5	spt	20	SP-SM		8	n	n	115	85	0	1115	53.40	1.15	1.28	1.00	0.80	1.42	1.30	26.07	33.5	0.4	33.94	0.99	0.66	0.854	1.65	1.10
11.0	spt	25	SP-SM		8	n	n	115	85	0	1287.5	61.66	1.15	1.18	1.00	0.85	1.42	1.30	34.62	40.8	0.4	41.26	0.99	0.66	1.901	1.79	1.10
12.5	spt	23	SP-SM		8	n	n	115	85	0	1460	69.92	1.15	1.14	1.00	0.85	1.42	1.30	31.85	36.2	0.4	36.71	0.99	0.65	1.549	1.45	1.10
14.0	spt	28	SP-SM		8	n	n	115	85	0	1632.5	78.18	1.15	1.08	1.00	0.85	1.42	1.30	38.77	41.9	0.4	42.41	0.98	0.65	1.864	1.58	1.08
15.5	spt	31	SP-SM		9	n	n	115	85	37.44	1767.56	84.65	1.15	1.05	1.00	0.85	1.42	1.30	42.93	45.1	0.7	46.05	0.98	0.66	1.824	1.57	1.06
17.0	spt	35	SP-SM		9	n	n	115	85	131.04	1846.46	88.43	1.15	1.04	1.00	0.95	1.42	1.30	54.17	56.1	0.7	57.03	0.98	0.69	1.801	1.51	1.04
18.5	spt	31	SP-SM		10	n	n	115	85	224.64	1925.36	92.21	1.15	1.02	1.00	0.95	1.42	1.30	47.98	49.1	1.1	50.64	0.97	0.72	1.780	1.44	1.03
20.0	spt	39	SP-SM		10	n	n	115	85	318.24	2004.26	95.99	1.15	1.01	1.00	0.95	1.42	1.30	60.36	61.2	1.1	62.67	0.97	0.74	1.759	1.40	1.02
25.0	spt	45	SM		12	n	n	115	85	630.24	2267.26	108.59	1.15	0.98	1.00	0.95	1.42	1.30	69.65	68.3	2.1	71.03	0.96	0.81	1.697	1.25	0.98
30.0	spt	47	SM		12	n	n	115	85	942.24	2530.26	121.18	1.15	0.95	1.00	0.95	1.42	1.30	72.74	69.3	2.1	72.03	0.94	0.86	1.641	1.12	0.95
35.0	spt	52	SM		12	n	n	115	85	1254.24	2793.26	133.78	1.15	0.93	1.00	1.00	1.42	1.30	84.72	78.7	2.1	81.37	0.93	0.89	1.590	1.02	0.92
40.0	spt	63	SM		13	n	n	115	85	1566.24	3056.26	146.37	1.15	0.91	1.00	1.00	1.42	1.30	102.64	93.1	2.5	96.35	0.91	0.92	1.544	0.95	0.89
45.0	spt	59	SM		12	n	n	115	85	1878.24	3319.26	158.97	1.15	0.89	1.00	1.00	1.42	1.30	96.12	85.3	2.1	88.00	0.90	0.93	1.502	0.87	0.87
50.0	spt	67	SP-SM		5	n	n	115	85	2190.24	3582.26	171.56	1.15	0.87	1.00	1.00	1.42	1.30	109.15	95.0	0.0	94.95	0.88	0.94	1.463	0.81	0.85

SAMPLE DEPTH (feet)	CRR STATIC SHEAR CORRECTION (Ka)	CORRECTED CRR (CCR_s) Sand	CORRECTED CRR (CCR_c) Clay	MAGNITUDE SCALING FACTOR (MSF)	FACTOR OF SAFETY (FS)Clay	FACTOR OF SAFETY (FS)Sand	LIQUEFACTION POTENTIAL	FS	Liquefaction Settlement: SAMPLE DEPTH (feet)	LIQUEFACTION POTENTIAL Incl. chk against PI	Soil Layer Thickness (ft)	Dynamic Vol. Strain (%): Tokimatsu	Dynamic Vol. Strain (%): Ishihara	Liquefaction Settlement (ft): Tokimatsu	Liquefaction Settlement (ft): Ishihara	Lateral Displacement Index and Settlement: Limiting Shear Strain gmin	Parameter Fa	Maximum Shear Strain gmax	DH (ft)	DLD_{II} (ft)	Vertical Reconsol. Strain ev	Liquefaction Settlement (ft) Ishihara
0.5	1.00	2.000	47.21	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	0.50	NO, NOT LIQ.	2.0	0.05	1.30	0.00	0.00	0.00	-3.722	0.00	2.00	0.000	0.000	0.00
2.0	1.00	2.000	9.11	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	2.00	NO, NOT LIQ.	1.5	0.05	1.30	0.00	0.00	0.00	-1.713	0.00	1.50	0.000	0.000	0.00
3.5	1.00	2.000	3.82	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	3.50	NO, NOT LIQ.	1.5	0.05	1.30	0.00	0.00	0.01	-0.672	0.00	1.50	0.000	0.000	0.00
5.0	1.00	2.000	2.72	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	5.00	NO, NOT LIQ.	1.5	0.05	1.30	0.00	0.00	0.01	-0.672	0.00	1.50	0.000	0.000	0.00
6.5	1.00	0.382	1.55	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	6.50	NO, NOT LIQ.	1.5	1.00	1.72	0.00	0.00	0.06	0.045	0.00	1.50	0.000	0.000	0.00
8.0	1.00	0.881	1.83	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	8.00	NO, NOT LIQ.	1.5	0.15	1.30	0.00	0.00	0.03	-0.353	0.00	1.50	0.000	0.000	0.00
9.5	1.00	0.899	1.82	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	9.50	NO, NOT LIQ.	1.5	0.15	1.30	0.00	0.00	0.03	-0.360	0.00	-14.00	0.000	0.000	0.00
11.0	1.00	2.000	1.97	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	11.00	NO, NOT LIQ.	1.5	0.05	1.30	0.00	0.00	0.01	-0.900	0.00	-17.50	0.000	0.000	0.00
12.5	1.00	1.629	1.60	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	12.50	NO, NOT LIQ.	1.5	0.05	1.30	0.00	0.00	0.02	-0.560	0.00	-18.00	0.000	0.000	0.00
14.0	1.00	2.000	1.71	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	14.00	NO, NOT LIQ.	1.5	0.05	1.30	0.00	0.00	0.01	-0.988	0.00	-19.50	0.000	0.000	0.00
15.5	1.00	2.000	1.65	0.86	-	2.000	NO, NOT LIQ.	2.00	15.50	NO, NOT LIQ.	1.5	0.05	0.00	0.00	0.00	0.00	-1.272	0.00	1.50	0.000	0.000	0.00
17.0	1.00	2.000	1.57	0.86	-	2.000	NO, NOT LIQ.	2.00	17.00	NO, NOT LIQ.	1.5	0.05	0.00	0.00	0.00	0.00	-2.171	0.00	1.50	0.000	0.000	0.00
18.5	1.00	2.000	1.48	0.86	-	2.000	NO, NOT LIQ.	2.00	18.50	NO, NOT LIQ.	1.5	0.05	0.00	0.00	0.00	0.00	-1.641	0.00	1.50	0.000	0.000	0.00
20.0	1.00	2.000	1.42	0.86	-	2.000	NO, NOT LIQ.	2.00	20.00	NO, NOT LIQ.	5.0	0.05	0.00	0.00	0.00	0.00	-2.653	0.00	5.00	0.000	0.000	0.00
25.0	1.00	2.000	1.22	0.86	-	2.000	NO, NOT LIQ.	2.00	25.00	NO, NOT LIQ.	5.0	0.05	0.00	0.00	0.00	0.00	-3.386	0.00	5.00	0.000	0.000	0.00
30.0	1.00	2.000	1.06	0.86	-	1.913	NO, NOT LIQ.	1.91	30.00	NO, NOT LIQ.	5.0	0.05	0.09	0.00	0.00	0.00	-3.476	0.00	5.00	0.000	0.000	0.00
35.0	1.00	2.000	0.94	0.86	-	1.784	NO, NOT LIQ.	1.78	35.00	NO, NOT LIQ.	5.0	0.05	0.09	0.00	0.00	0.00	-4.322	0.00	5.00	0.000	0.000	0.00
40.0	1.00	2.000	0.85	0.86	-	1.688	NO, NOT LIQ.	1.69	40.00	NO, NOT LIQ.	5.0	0.05	0.09	0.00	0.00	0.00	-5.721	0.00	5.00	0.000	0.000	0.00
45.0	1.00	2.000	0.75	0.86	-	1.614	NO, NOT LIQ.	1.61	45.00	NO, NOT LIQ.	5.0	0.05	0.09	0.00	0.00	0.00	-4.935	0.00	5.00	0.000	0.000	0.00
50.0	1.00	2.000	0.69	0.86	-	1.557	NO, NOT LIQ.	1.56	50.00	NO, NOT LIQ.	1.5	0.05	0.15	0.00	0.00	0.00	-5.588	0.00	1.50	0.000	0.000	0.00

AGS-18-003 South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection

Site Data		
S	Ground Slope / Liquefiable Layer Slope	0.1%
L	Horizontal Distance from Free Face (ft)	120
H	Free Face Height (ft)	40
Gw	Depth to groundwater (ft)	20
Mw	Maximum Moment Magnitude	8.1
PGA	Peak Ground Acceleration	0.922

Boring Data	
Depth of boring (ft)	38
Depth to groundwater (ft)	20.0
Borehole diameter (in)	6

Settlement		
ft	in	
0.00	0.00	
0.00	0.00	(above YBM)
0.00	0.00	(in upper 50 feet)
0.00	0.00	(in upper 70 feet)

Lateral Displacement				
LDI	Sloping		Free Face	
	ft	in	ft	in
0.000	0.0	0.0	0.0	0.0

Sample Depth is to top of sample layer.

Fines content estimated based on soil classification where no test data available.

Soil Type based on Unified Soils Classification System (USCS), ASTM D2487.

Equations and procedures used to calculate liquefaction potential is based on Soil Liquefaction During Earthquakes by Idriss and Boulanger published by EERI MNO-12 2008

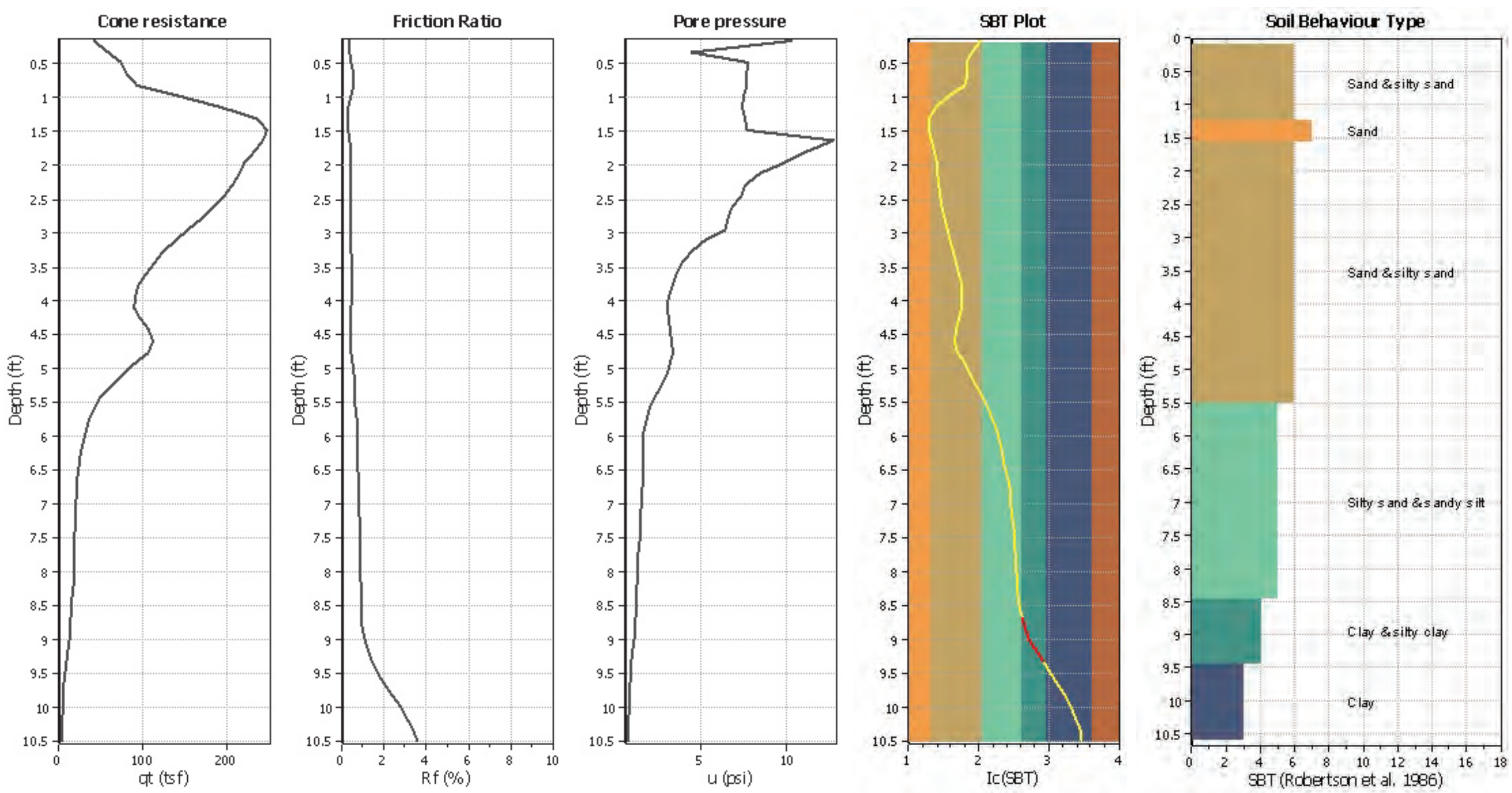
Blue cells are input data, orange cells are calculated data.

Table B-6 - Seismically-Induced Settlement & Lateral Deformation Calculation - B-6

													CORRECTION FACTORS													
SAMPLE DEPTH (feet)	SAMPLE TYPE	UNCORRECTED BLOW COUNT (SPT-N) (blows per foot)	SOIL TYPE	PI Clay/Silt	ESTIMATED FINES CONTENT (%)	GRAVEL? Y=yes N=no	SAMPLER LINER? Y=yes N=no	UNIT WEIGHT OF SOIL LAYER (pcf)	ENERGY RATIO, ER (%)	PORE PRESSURE (psf)	EFFECTIVE OVER-BURDEN (psf)	EFFECTIVE OVER-BURDEN (MPa)	C_b Borehole Size Factor	Cn Depth Reduction Factor	Cm MC Factor	CR Drill Rod Length Factor	Ch Hammer Efficiency Factor	Cs Unlined SPT Factor	PENETRATION RESISTANCE N60	MODIFIED PENETRATION RESISTANCE (N1)60	δN for Fines Content	MOD PEN RES ADJUSTED TO CLEAN SAND (N1)60CS	STRESS REDUCTION COEFFICIENT (rd)	CYCLIC STRESS RATIO (CSR)	CYCLIC RESISTANCE RATIO (CRR) Sand	CYCLIC RESISTANCE RATIO (CRR) Clay
1.0	spt	43	SP			y	n	130	85	0	130	6.23	1.15	1.70	1.00	0.75	1.42	1.30	52.54	89.3	0.0	89.32	1.00	0.60	1.901	21.68
3.0	spt	32	SP-SM		5	n	n	120	85	0	370	17.72	1.15	1.70	1.00	0.75	1.42	1.30	39.10	66.3	0.0	66.34	1.00	0.60	1.901	7.50
5.0	spt	36	SP-SM		5	n	n	120	85	0	610	29.21	1.15	1.41	1.00	0.75	1.42	1.30	43.99	61.8	0.0	61.84	1.00	0.60	1.901	4.57
7.0	spt	36	SP-SM		5	n	n	115	85	0	840	40.23	1.15	1.27	1.00	0.80	1.42	1.30	46.92	59.8	0.0	59.78	1.00	0.60	1.901	3.32
9.0	spt	53	SP-SM		5	n	n	115	85	0	1070	51.25	1.15	1.20	1.00	0.80	1.42	1.30	69.08	82.6	0.0	82.58	0.99	0.59	1.901	2.67
11.0	spt	44	SM		16	n	n	115	85	0	1300	62.26	1.15	1.14	1.00	0.85	1.42	1.30	60.93	69.2	3.6	73.85	0.99	0.59	1.901	2.17
13.0	spt	38	SP		4	n	n	115	85	0	1530	73.28	1.15	1.09	1.00	0.85	1.42	1.30	52.62	57.3	0.0	57.26	0.98	0.59	1.897	1.83
15.0	spt	49	SP-SM		5	n	n	115	85	0	1760	84.29	1.15	1.05	1.00	0.85	1.42	1.30	67.85	71.2	0.0	71.16	0.98	0.59	1.826	1.62
17.0	spt	42	SP-SM		5	n	n	115	85	0	1990	95.31	1.15	1.02	1.00	0.95	1.42	1.30	65.00	66.0	0.0	66.01	0.98	0.58	1.763	1.41
19.0	spt	49	SP-SM		5	n	n	115	85	0	2220	106.32	1.15	0.99	1.00	0.95	1.42	1.30	75.84	74.8	0.0	74.82	0.97	0.58	1.707	1.28
25.0	spt	39	SP-SM		9	n	n	115	85	312	2598	124.43	1.15	0.95	1.00	0.95	1.42	1.30	60.36	57.1	0.7	58.07	0.96	0.64	1.627	1.08
30.0	spt	45	SP-SM		5	n	n	115	85	624	2861	137.02	1.15	0.92	1.00	0.95	1.42	1.30	69.65	64.3	0.0	64.28	0.94	0.69	1.578	0.99
35.0	spt	50	SP-SM		5	n	n	115	85	936	3124	149.62	1.15	0.90	1.00	1.00	1.42	1.30	81.46	73.5	0.0	73.46	0.93	0.72	1.533	0.91

									Liquefaction Settlement			Dynamic Vol. Strain (%)		Liquefaction Settlement (ft)		Lateral Displacement Index and Settlement						
SAMPLE DEPTH (feet)	CRR OVERBURDEN CORRECTION (Ks)	CRR STATIC SHEAR CORRECTION (Ka)	CORRECTED CRR (CCR_R) Sand	CORRECTED CRR (CCR_R) Clay	MAGNITUDE SCALING FACTOR (MSF)	FACTOR OF SAFETY (FS)Clay	FACTOR OF SAFETY (FS)Sand	LIQUEFACTION POTENTIAL	FS	SAMPLE DEPTH (feet)	LIQUEFACTION POTENTIAL Incl. chk against PI	Soil Layer Thickness (ft)	Tokimatsu	Ishihara	Tokimatsu	Ishihara	Limiting Shear Strain gmin	Parameter Fa	Maximum Shear Strain gmax	DH (ft)	DLD_{ii} (ft)	Vertical Reconsol. Strain ev	Liquefaction Settlement (ft) Ishihara
1.0	1.10	1.00	2.000	23.85	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	1.00	NO, NOT LIQ.	3.0	0.05	1.30	0.00	0.00	0.00	-5.058	0.00	2.00	0.000	0.000	0.00
3.0	1.10	1.00	2.000	8.25	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	3.00	NO, NOT LIQ.	2.0	0.05	1.30	0.00	0.00	0.00	-2.972	0.00	1.50	0.000	0.000	0.00
5.0	1.10	1.00	2.000	5.03	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	5.00	NO, NOT LIQ.	2.0	0.05	1.30	0.00	0.00	0.00	-2.582	0.00	1.50	0.000	0.000	0.00
7.0	1.10	1.00	2.000	3.65	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	7.00	NO, NOT LIQ.	2.0	0.05	1.30	0.00	0.00	0.00	-2.405	0.00	1.50	0.000	0.000	0.00
9.0	1.10	1.00	2.000	2.94	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	9.00	NO, NOT LIQ.	2.0	0.05	1.30	0.00	0.00	0.00	-4.433	0.00	1.50	0.000	0.000	0.00
11.0	1.10	1.00	2.000	2.39	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	11.00	NO, NOT LIQ.	2.0	0.05	1.30	0.00	0.00	0.00	-3.639	0.00	1.50	0.000	0.000	0.00
13.0	1.10	1.00	2.000	2.01	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	13.00	NO, NOT LIQ.	2.0	0.05	1.30	0.00	0.00	0.00	-2.190	0.00	-14.00	0.000	0.000	0.00
15.0	1.06	1.00	2.000	1.71	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	15.00	NO, NOT LIQ.	2.0	0.05	1.30	0.00	0.00	0.00	-3.399	0.00	-17.50	0.000	0.000	0.00
17.0	1.02	1.00	2.000	1.44	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	17.00	NO, NOT LIQ.	2.0	0.05	1.30	0.00	0.00	0.00	-2.943	0.00	-18.00	0.000	0.000	0.00
19.0	0.99	1.00	2.000	1.26	0.86	-	ABOVE G.W.	ABOVE G.W.	0.00	19.00	NO, NOT LIQ.	6.0	0.05	1.30	0.00	0.00	0.00	-3.726	0.00	-19.50	0.000	0.000	0.00
25.0	0.94	1.00	2.000	1.02	0.86	-	2.000	NO, NOT LIQ.	2.00	25.00	NO, NOT LIQ.	5.0	0.05	0.00	0.00	0.00	0.00	-2.259	0.00	1.50	0.000	0.000	0.00
30.0	0.91	1.00	2.000	0.90	0.86	-	2.000	NO, NOT LIQ.	2.00	30.00	NO, NOT LIQ.	5.0	0.05	0.00	0.00	0.00	0.00	-2.792	0.00	1.50	0.000	0.000	0.00
35.0	0.89	1.00	2.000	0.81	0.86	-	2.000	NO, NOT LIQ.	2.00	35.00	NO, NOT LIQ.	3.0	0.05	0.00	0.00	0.00	0.00	-3.604	0.00	1.50	0.000	0.000	0.00

CPT basic interpretation plo



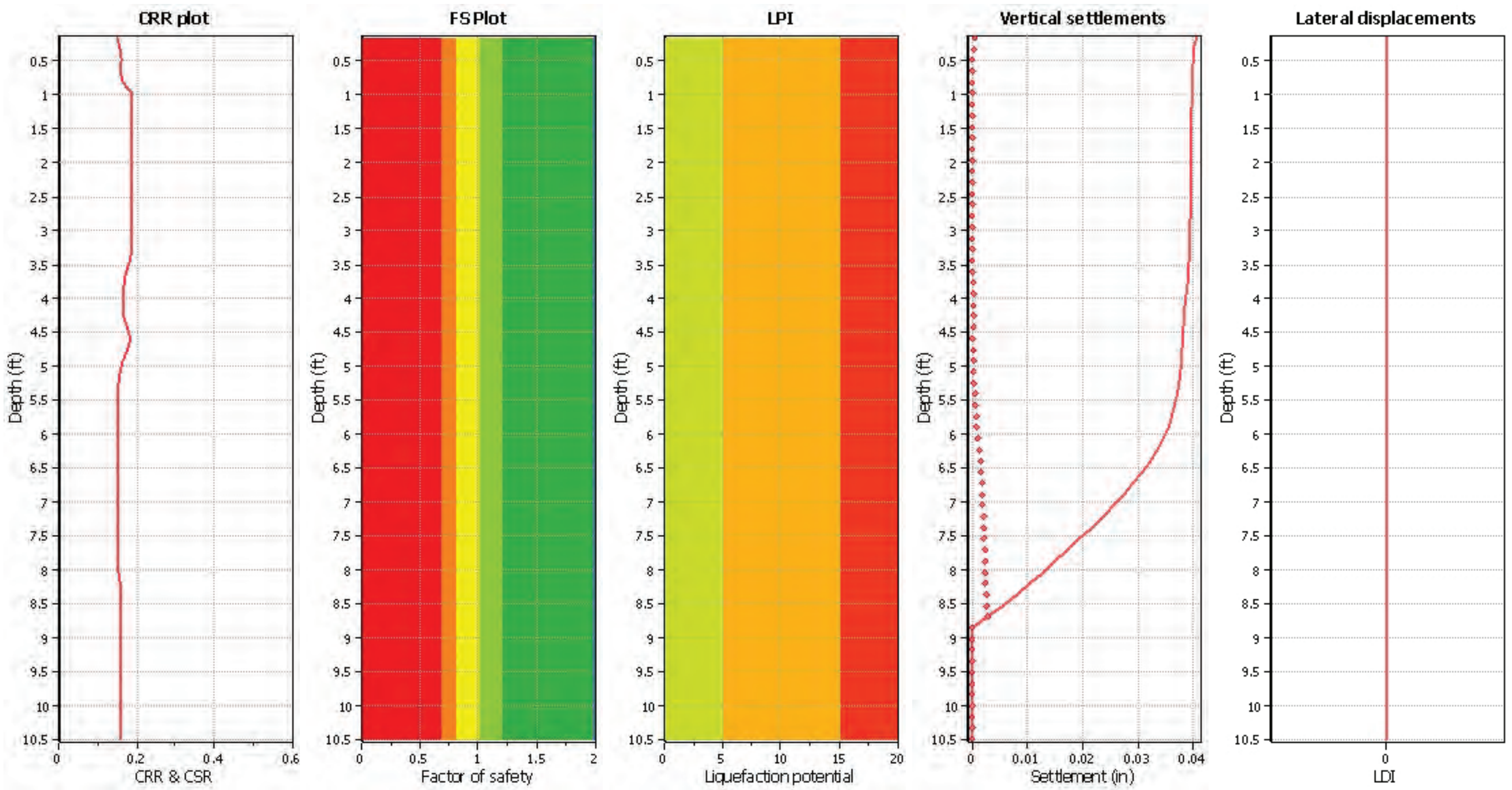
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	15.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to clayey sand
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	15.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Fill height:	N/A	Limit depth:	N/A

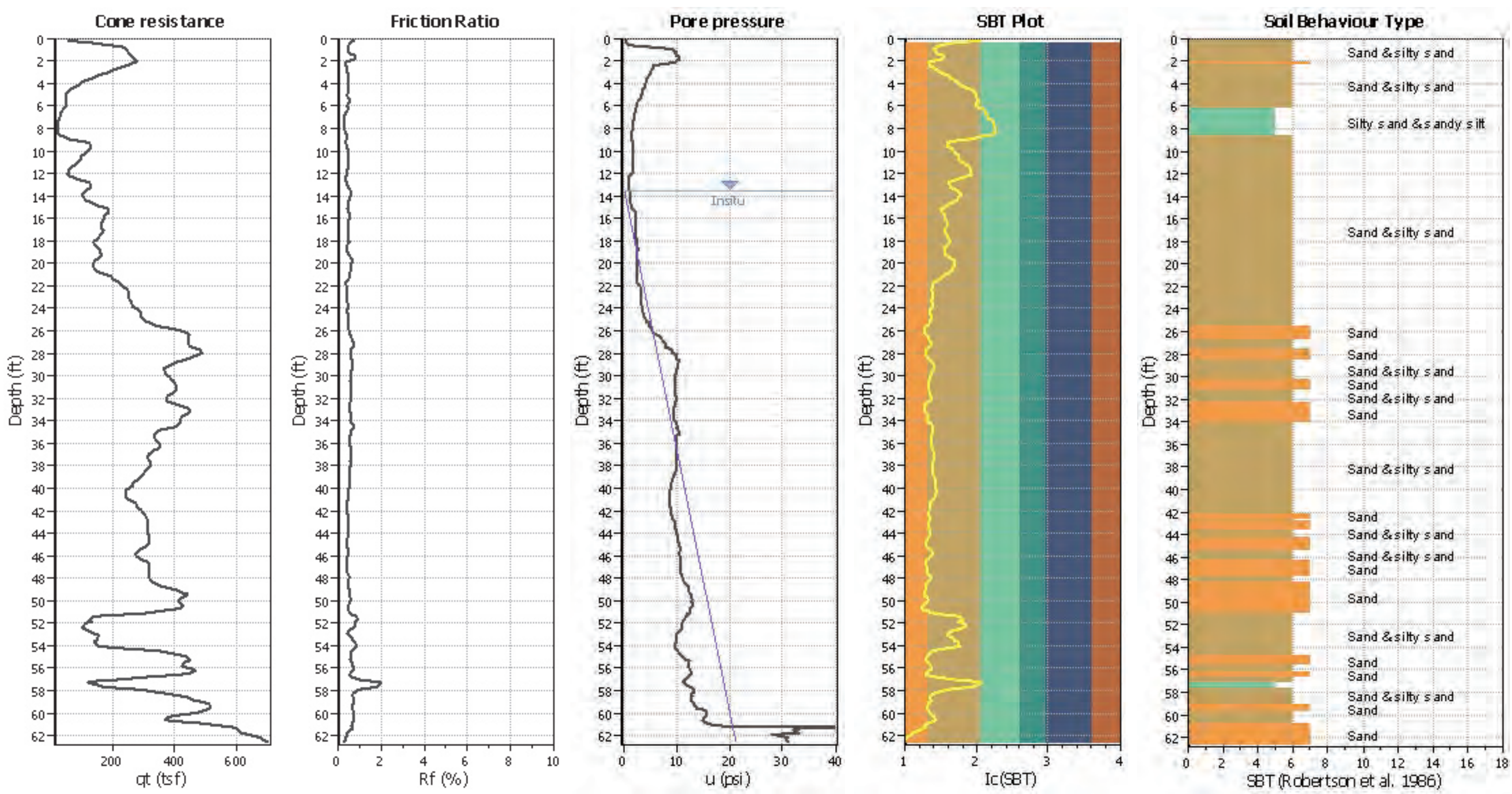
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



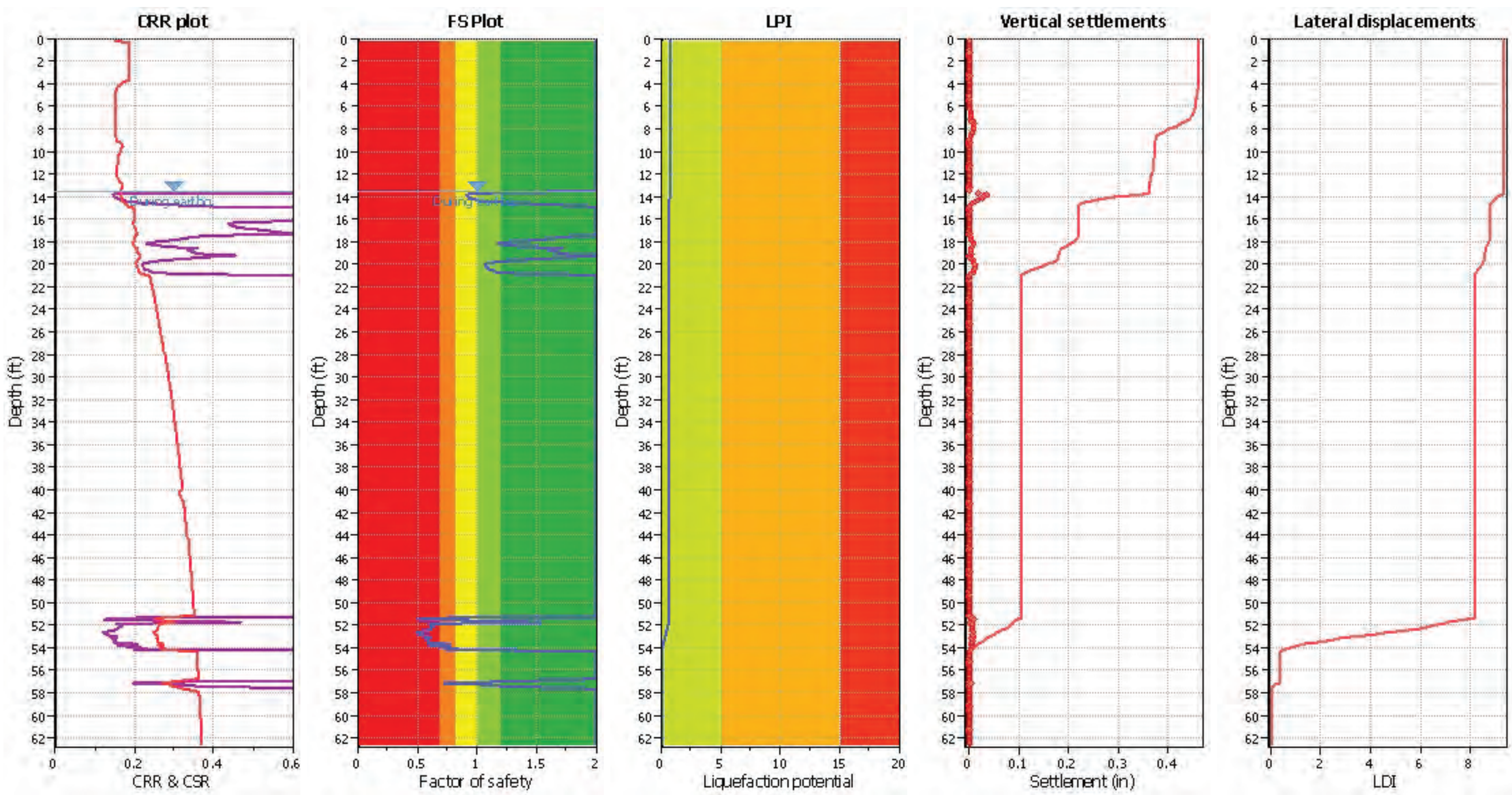
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.50 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.50 ft	Fill height:	N/A	Limit depth:	N/A

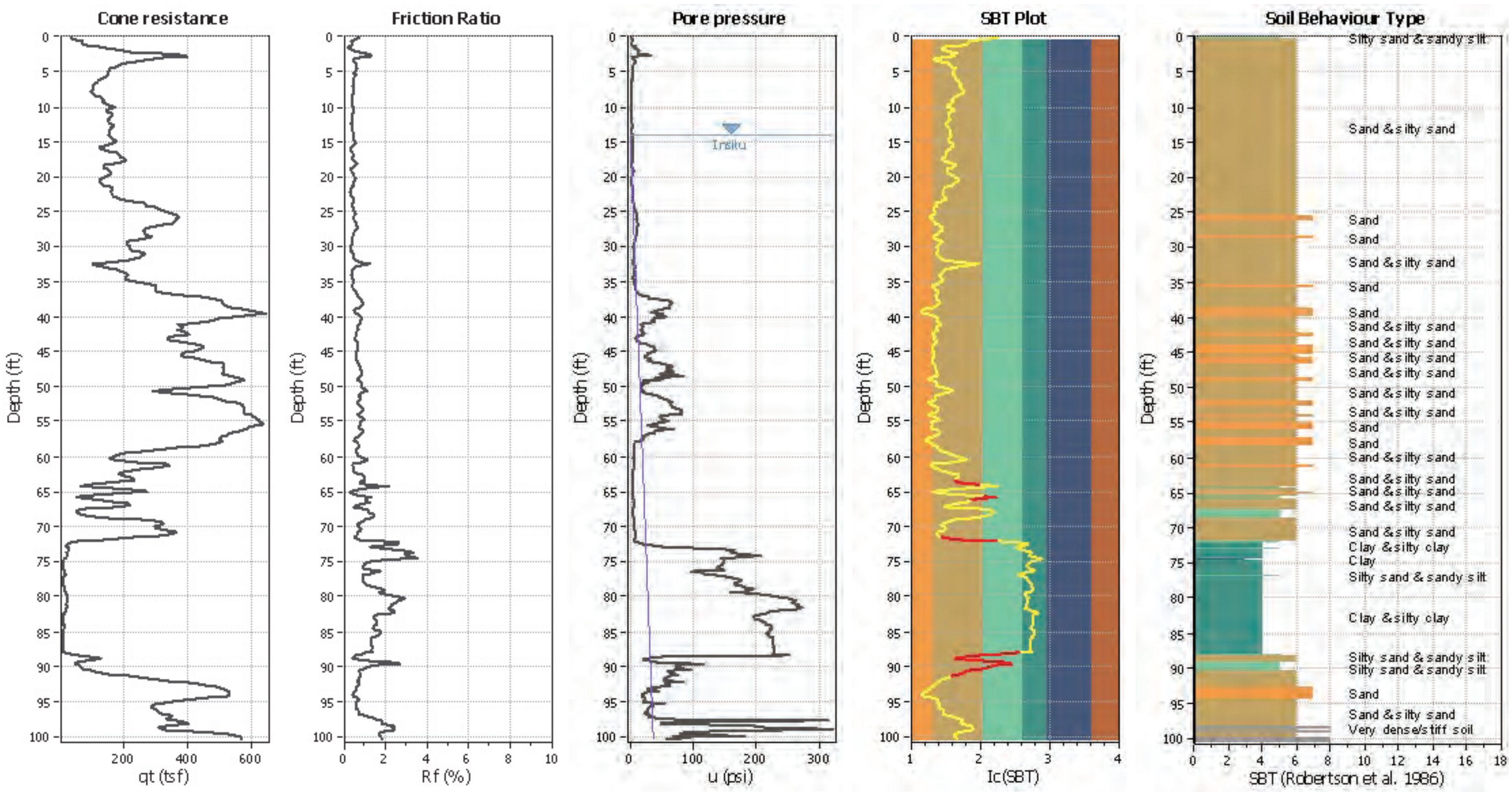
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



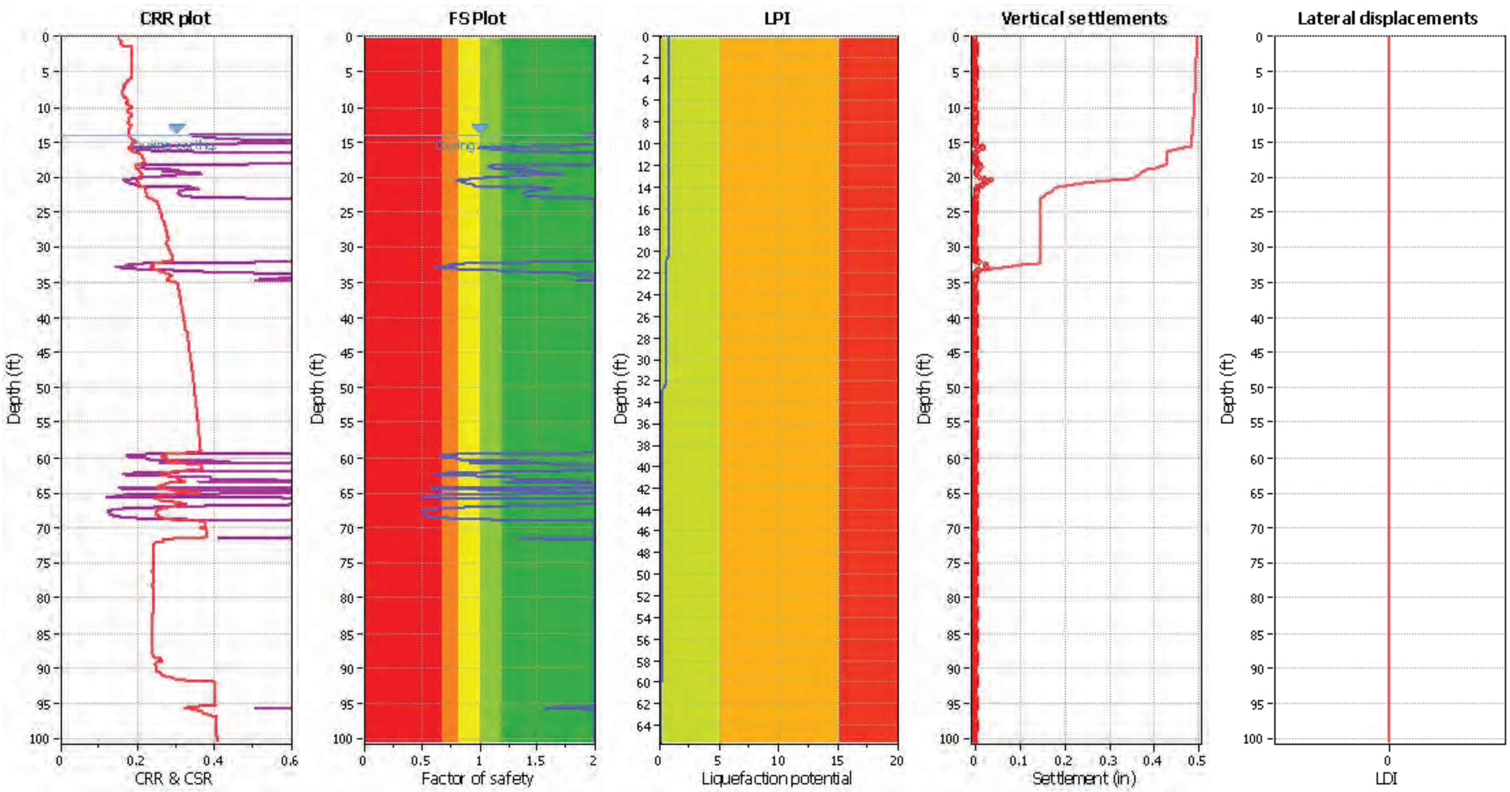
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	14.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	14.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

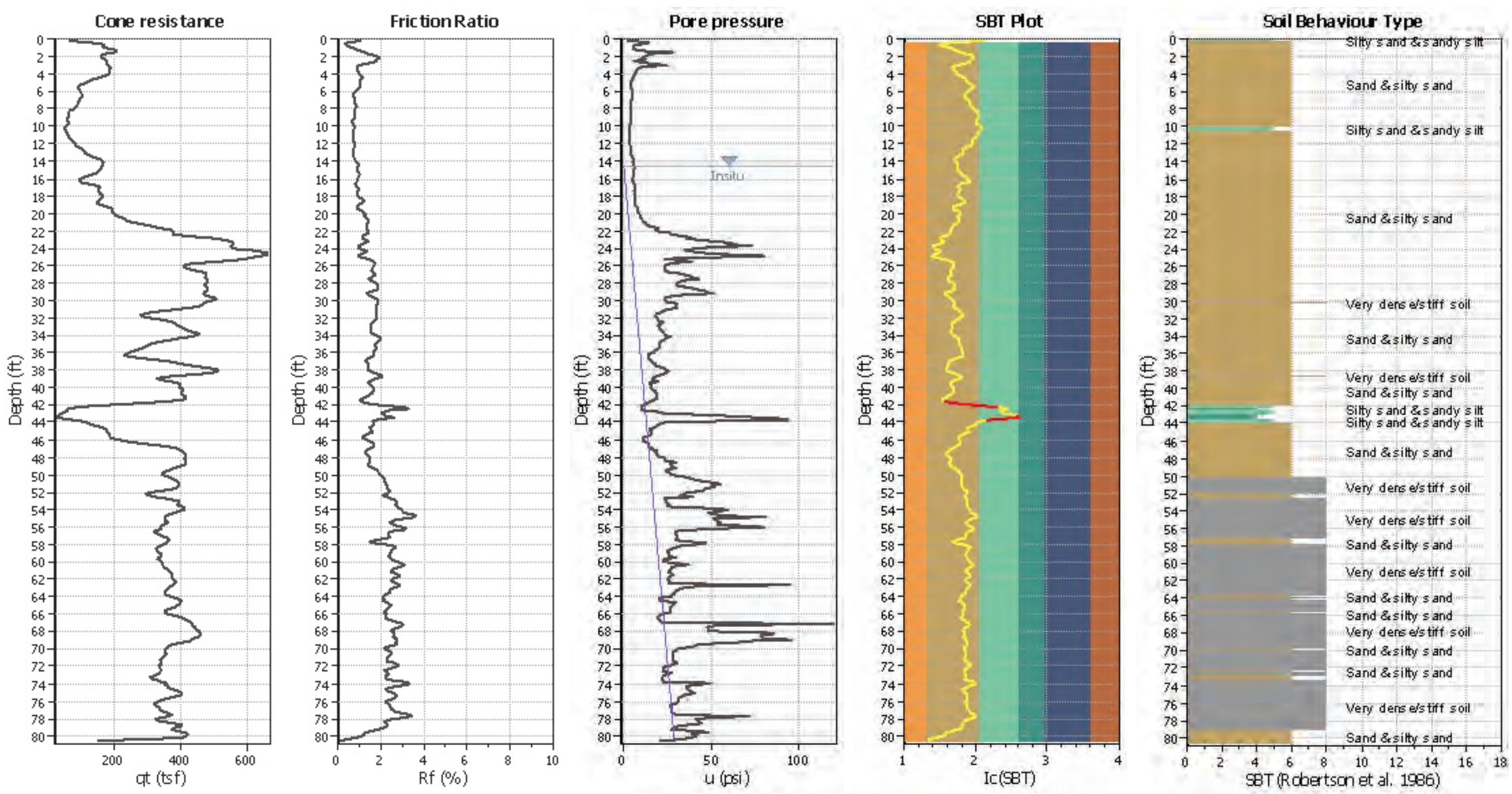
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



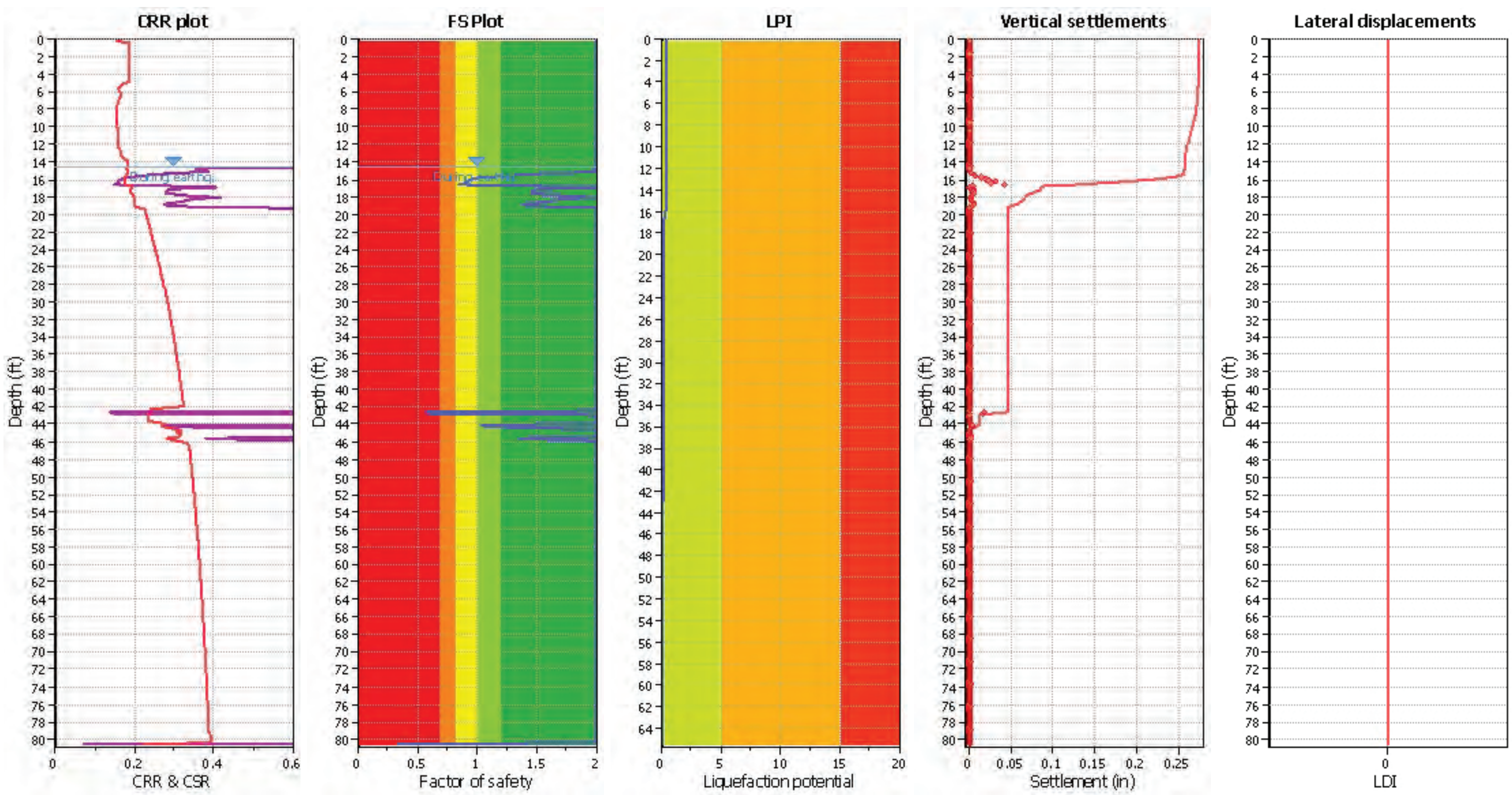
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	14.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.50 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	14.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.50 ft	Fill height:	N/A	Limit depth:	N/A

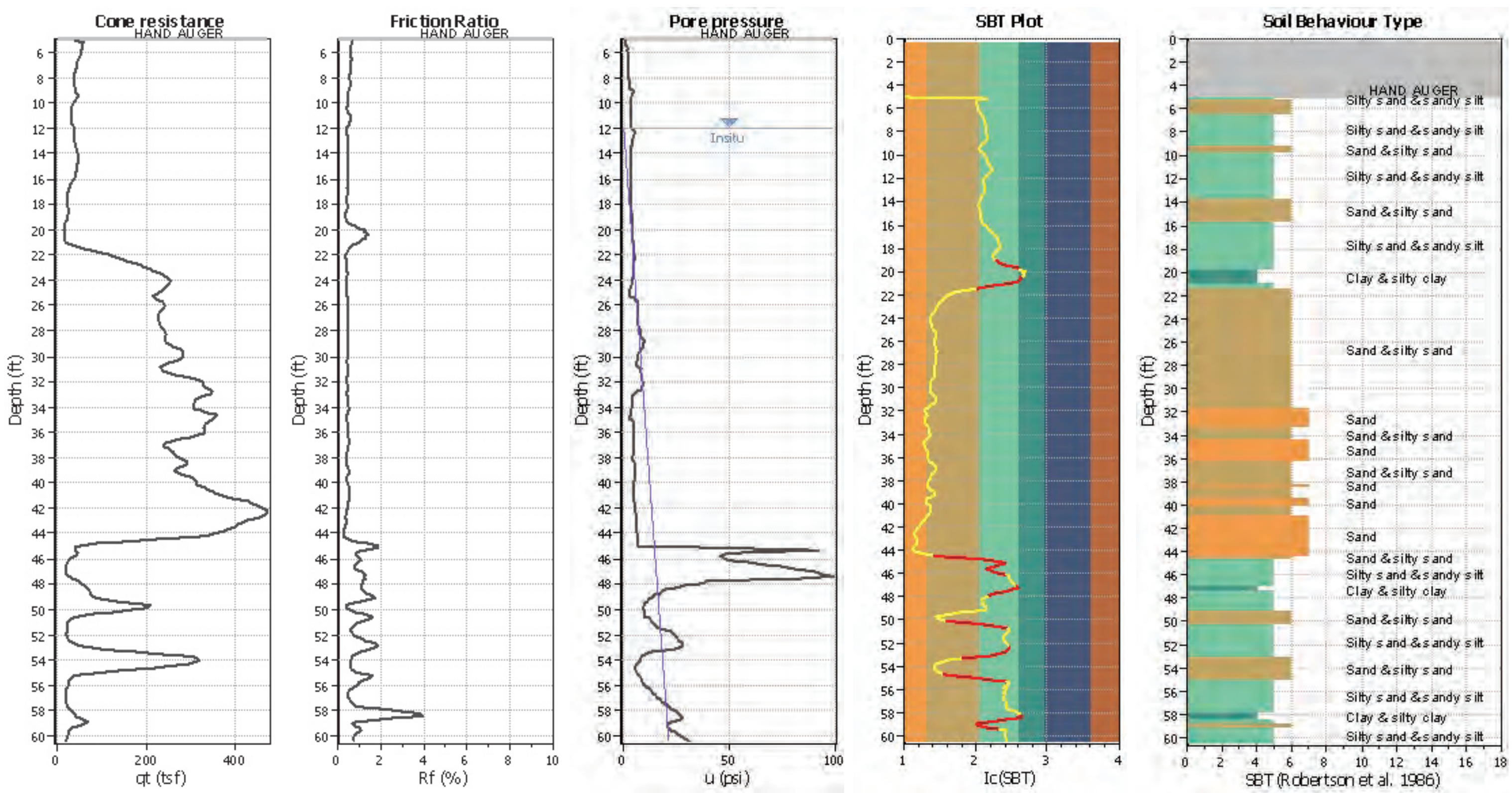
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



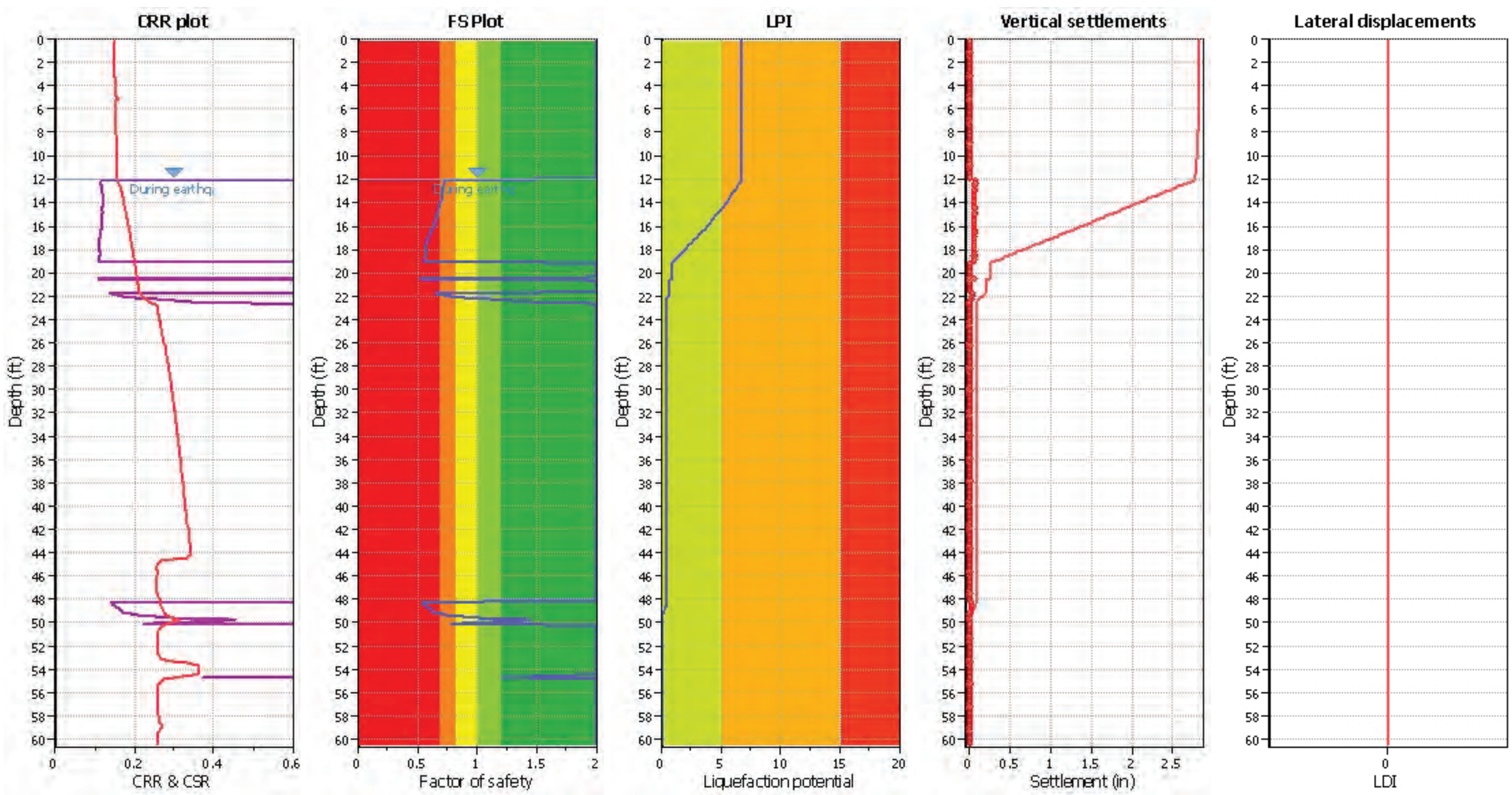
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	12.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	12.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	12.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	12.00 ft	Fill height:	N/A	Limit depth:	N/A

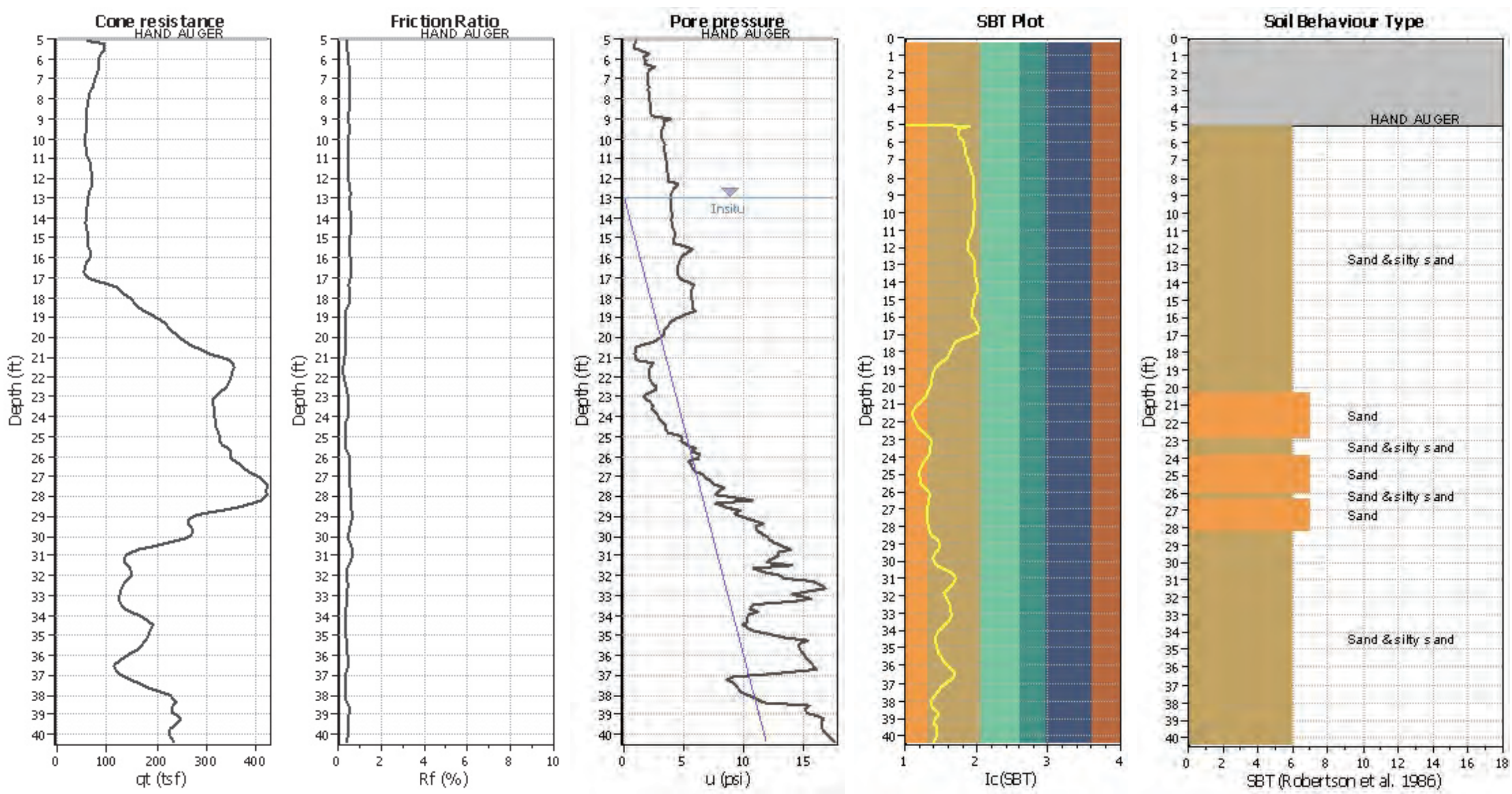
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



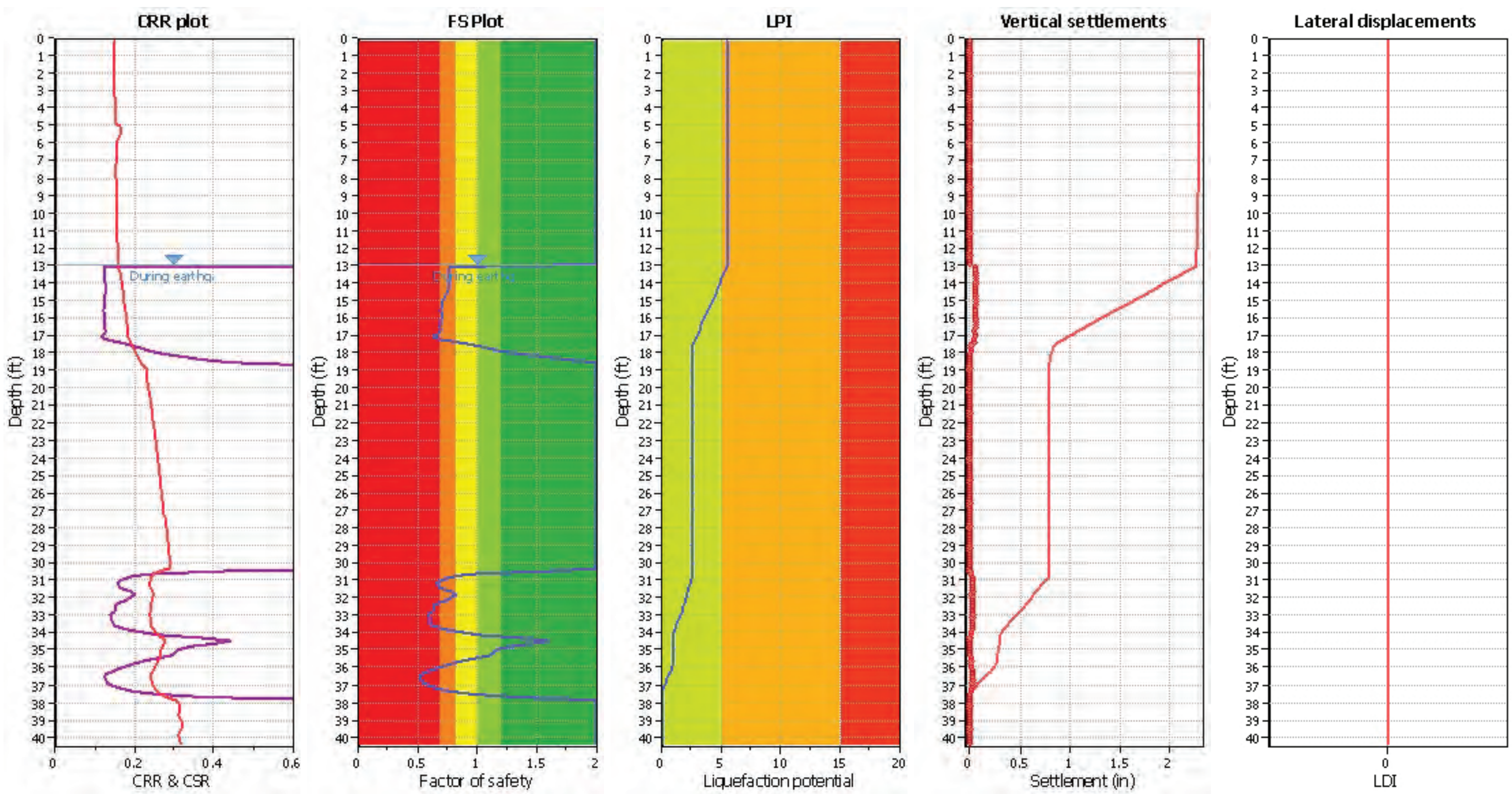
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A

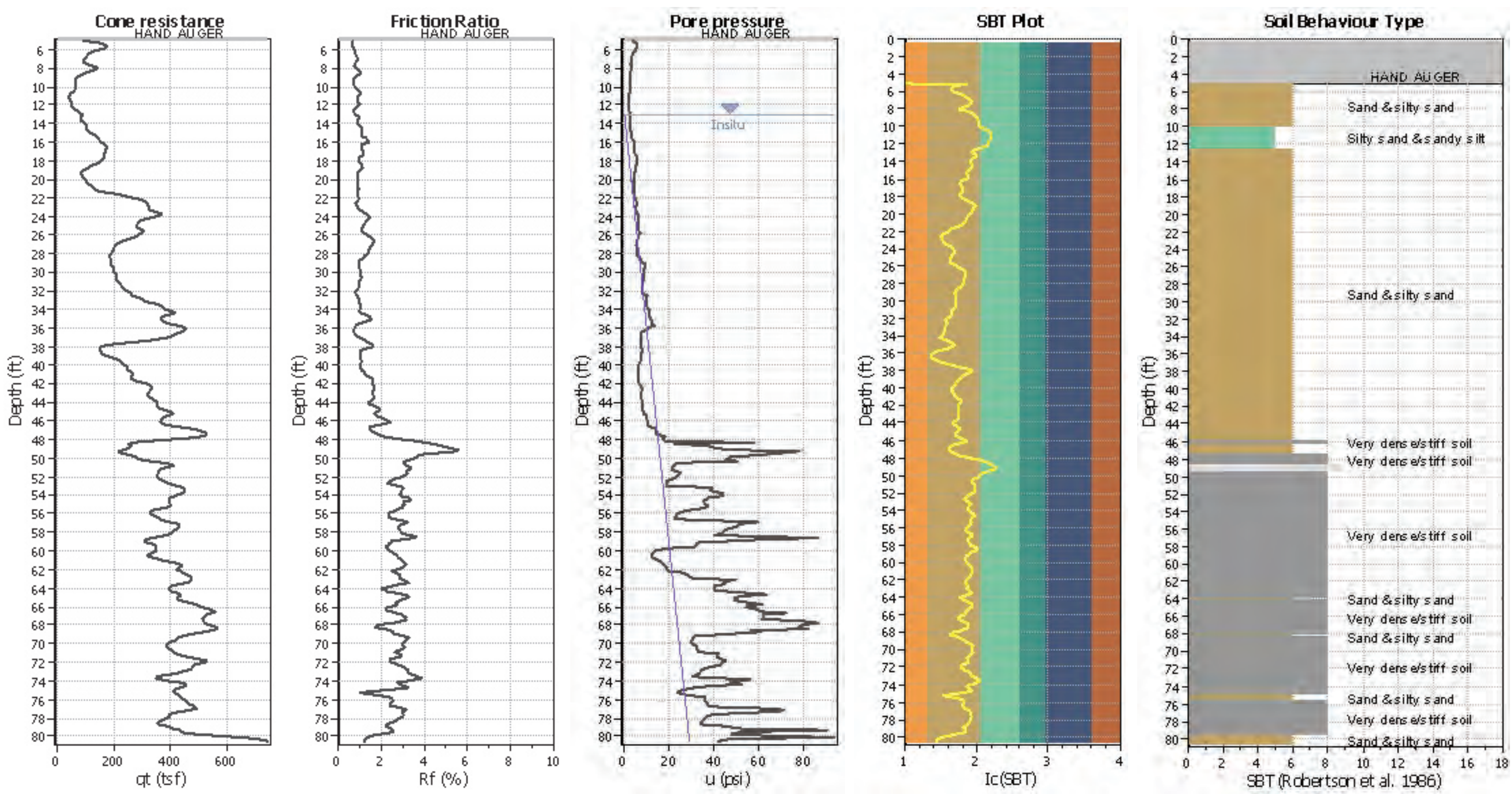
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



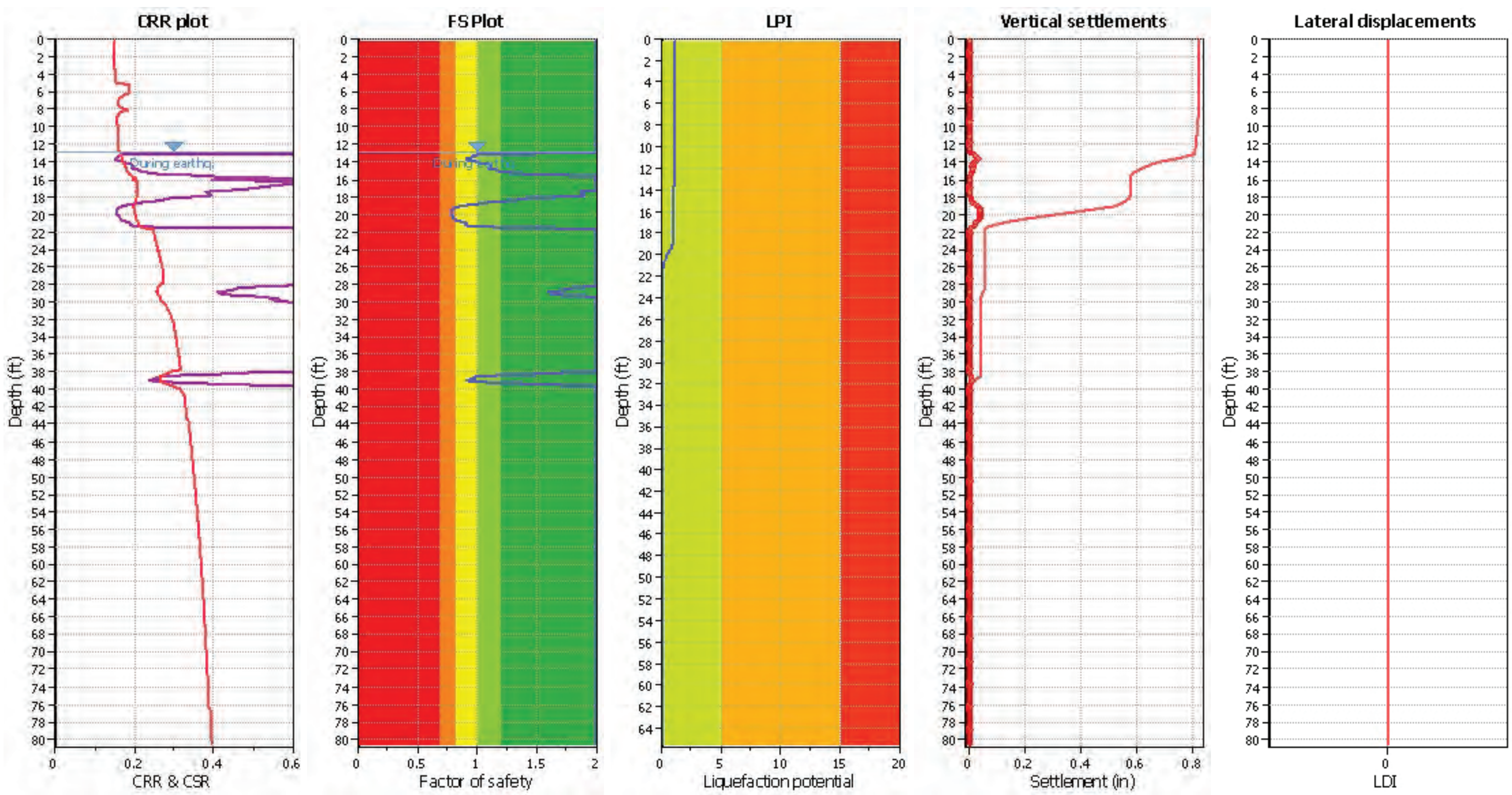
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to clayey sand
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A

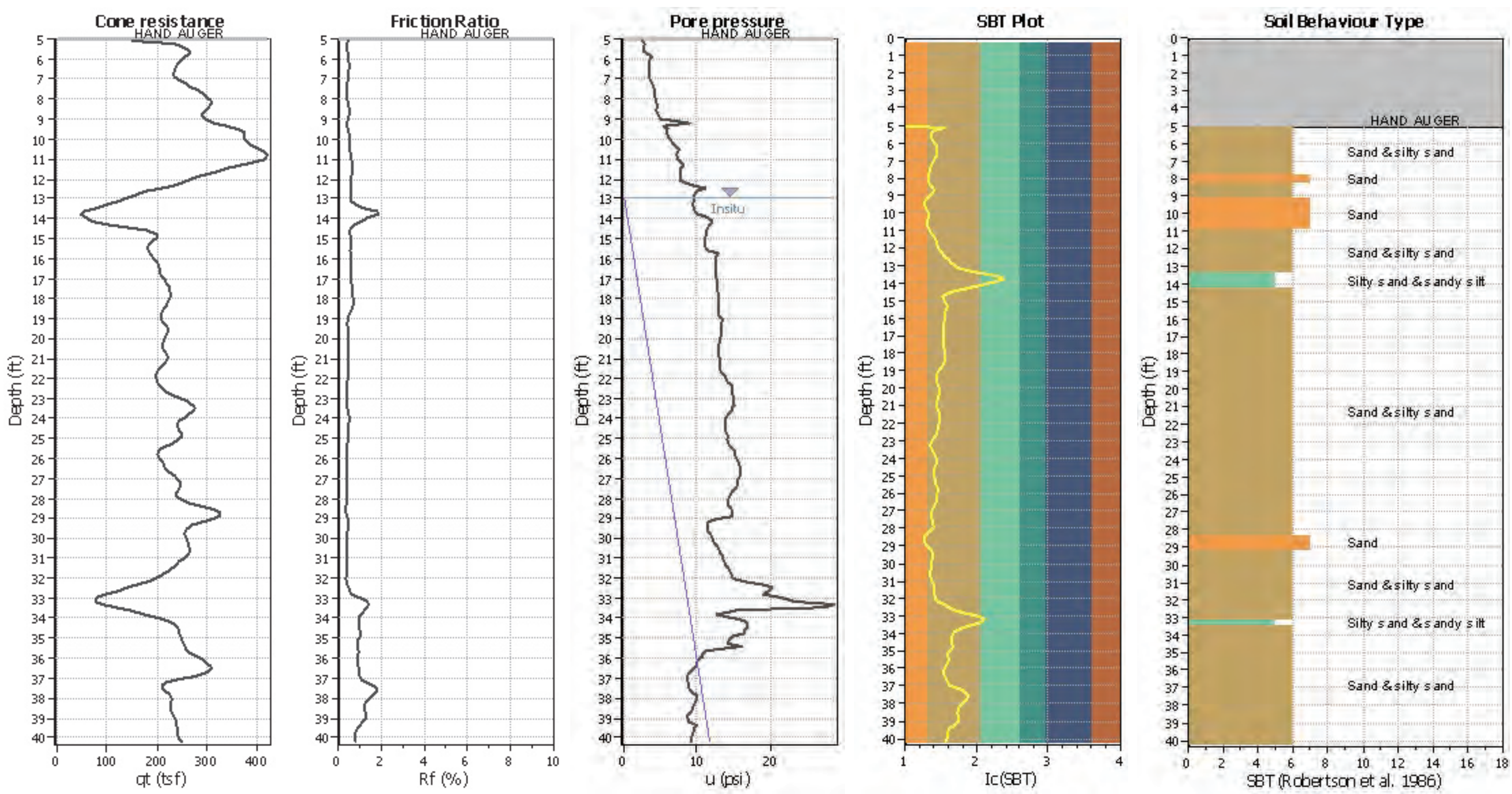
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Exhibit 8
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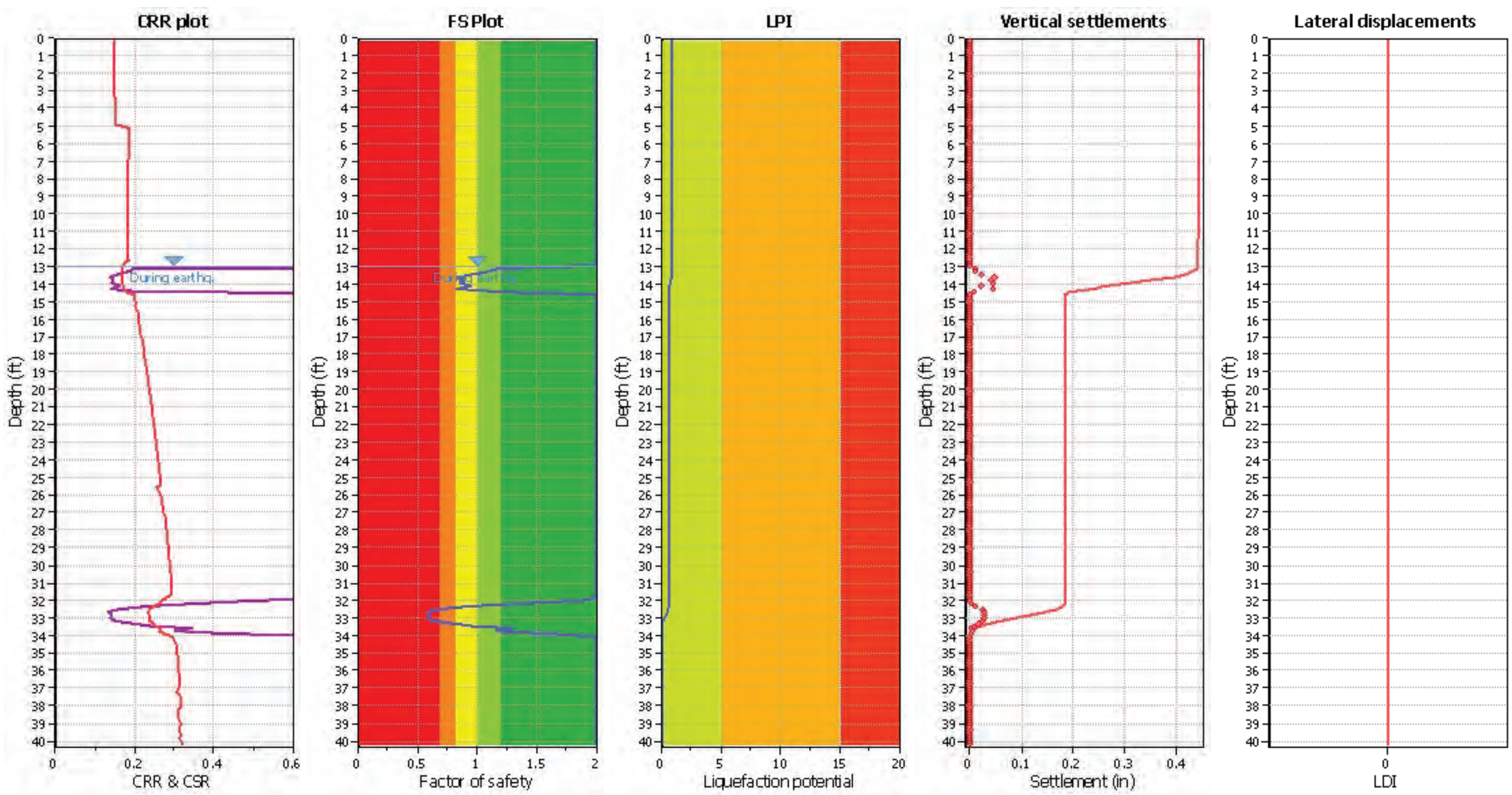
Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A

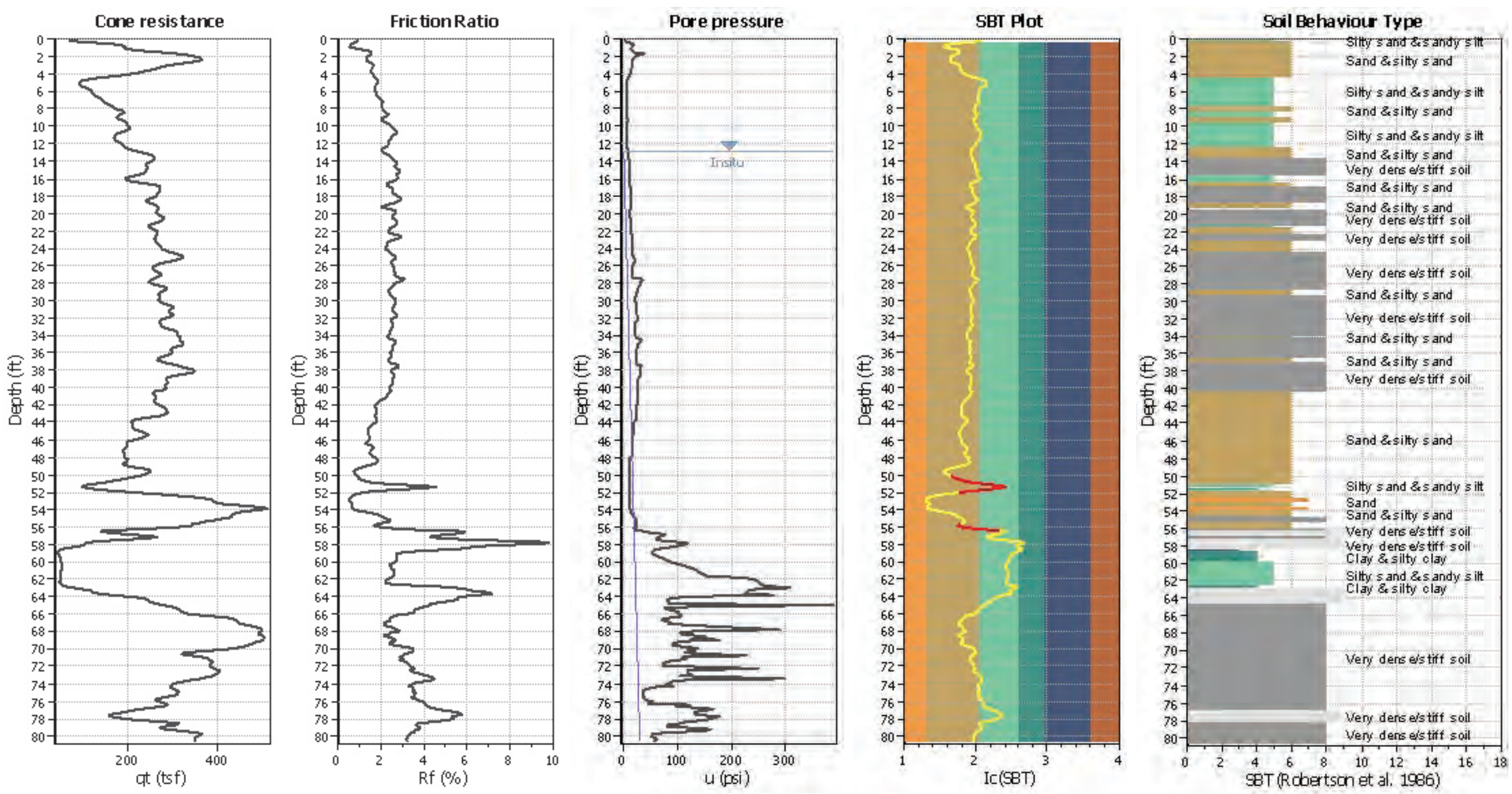
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



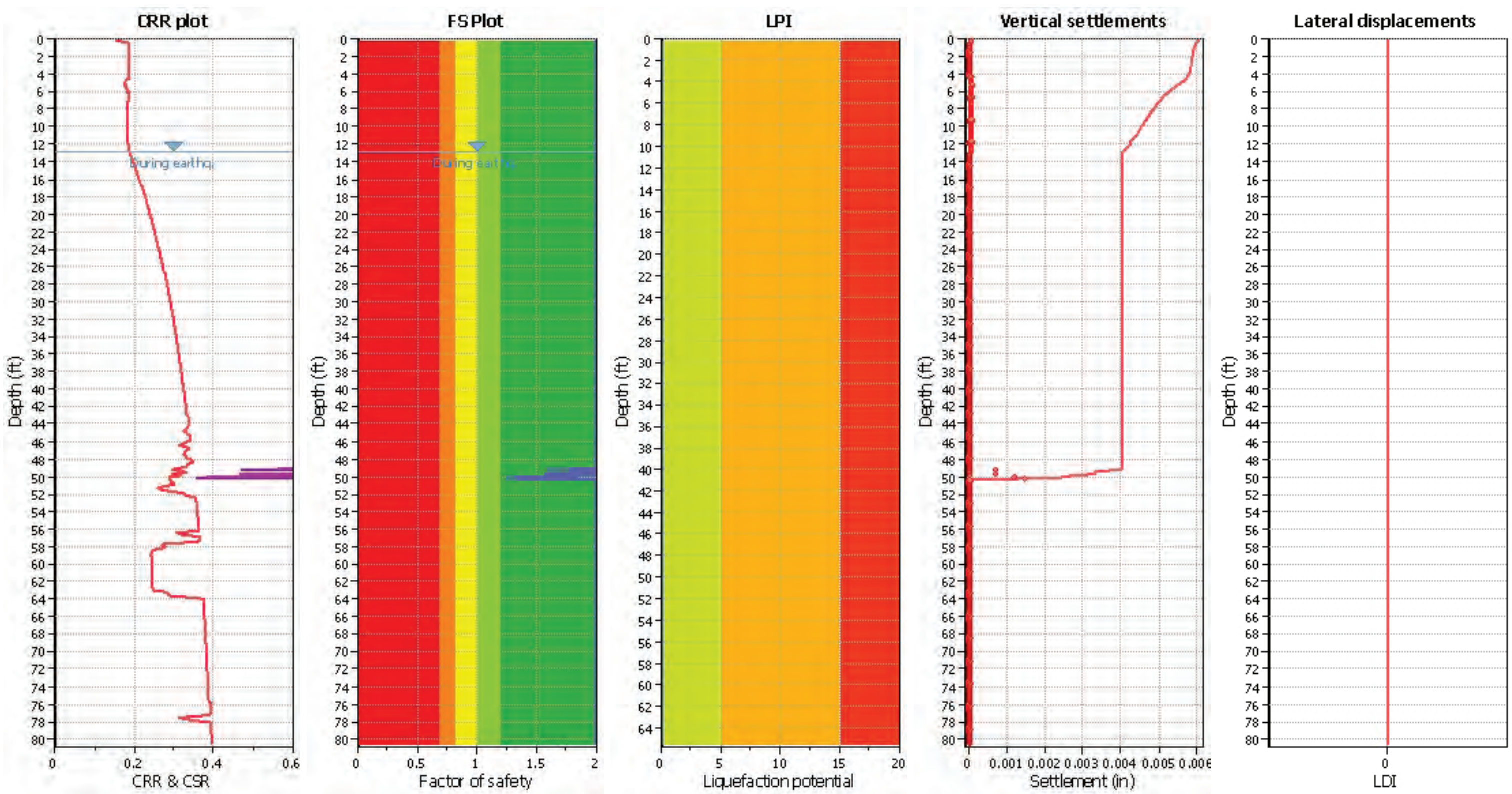
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.00 ft	Fill weight:	N/A		
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes		
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes		
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only		
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No		
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A		

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A

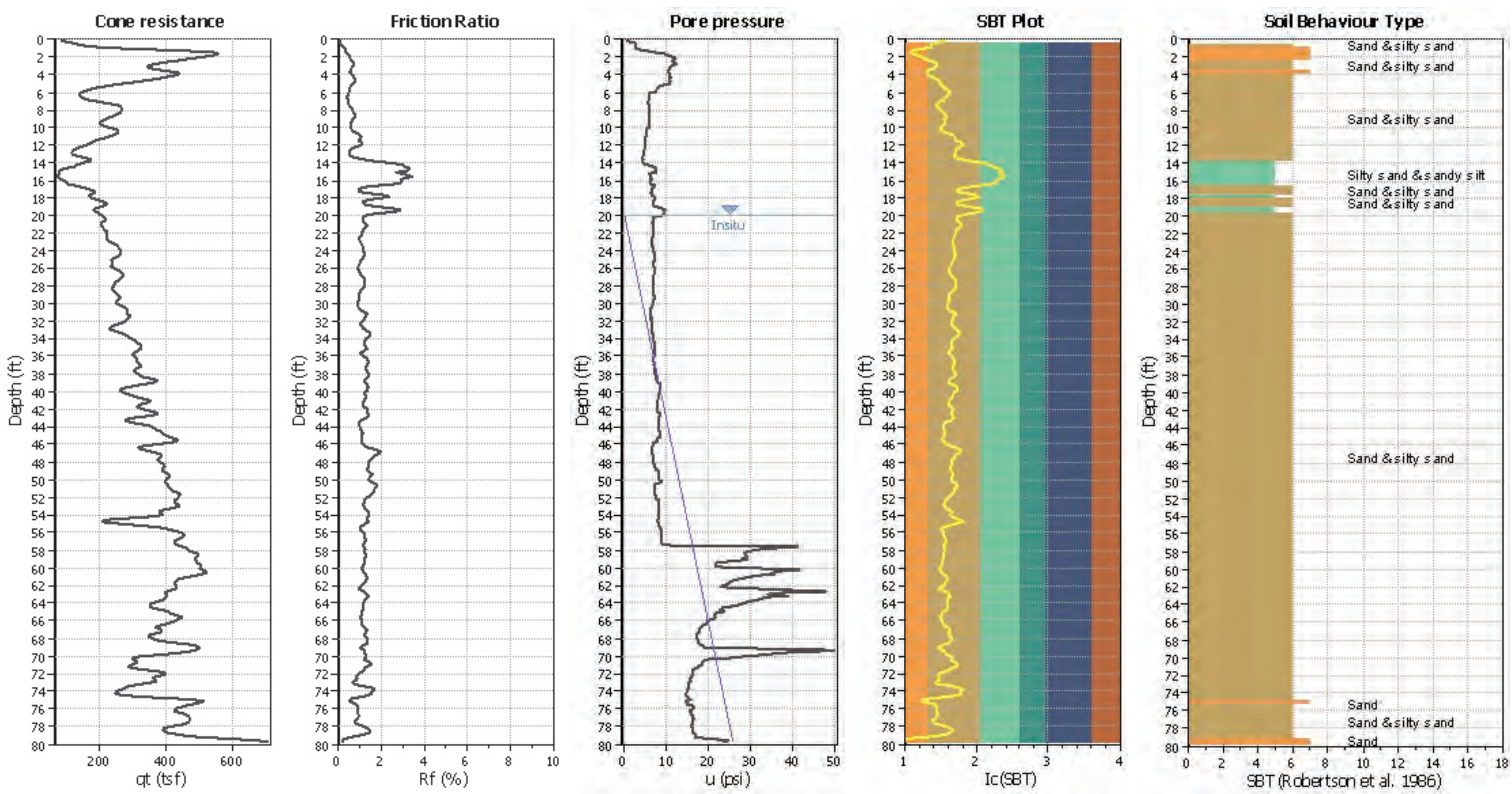
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



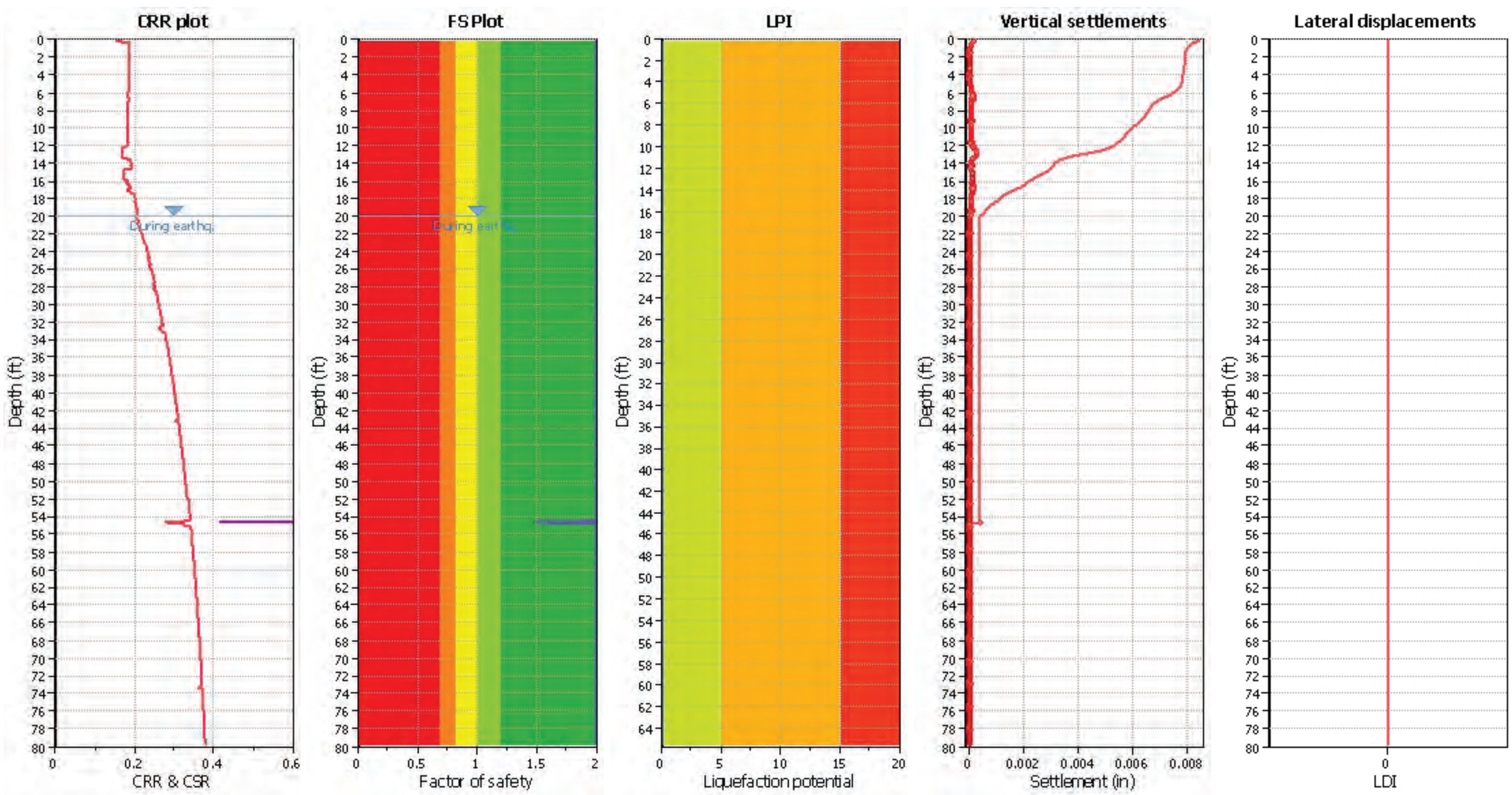
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	20.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	20.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to clayey sand
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	20.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	20.00 ft	Fill height:	N/A	Limit depth:	N/A

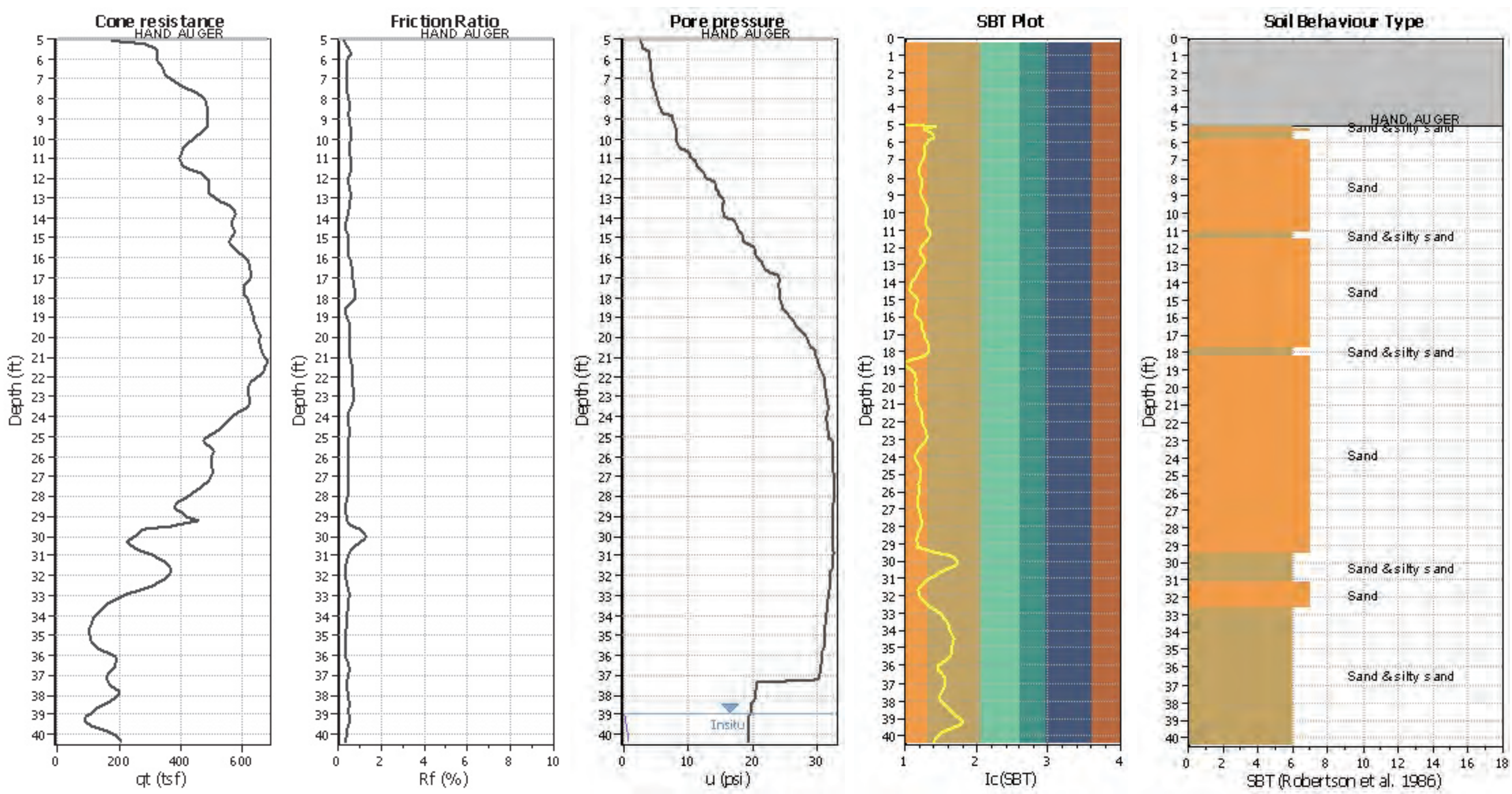
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



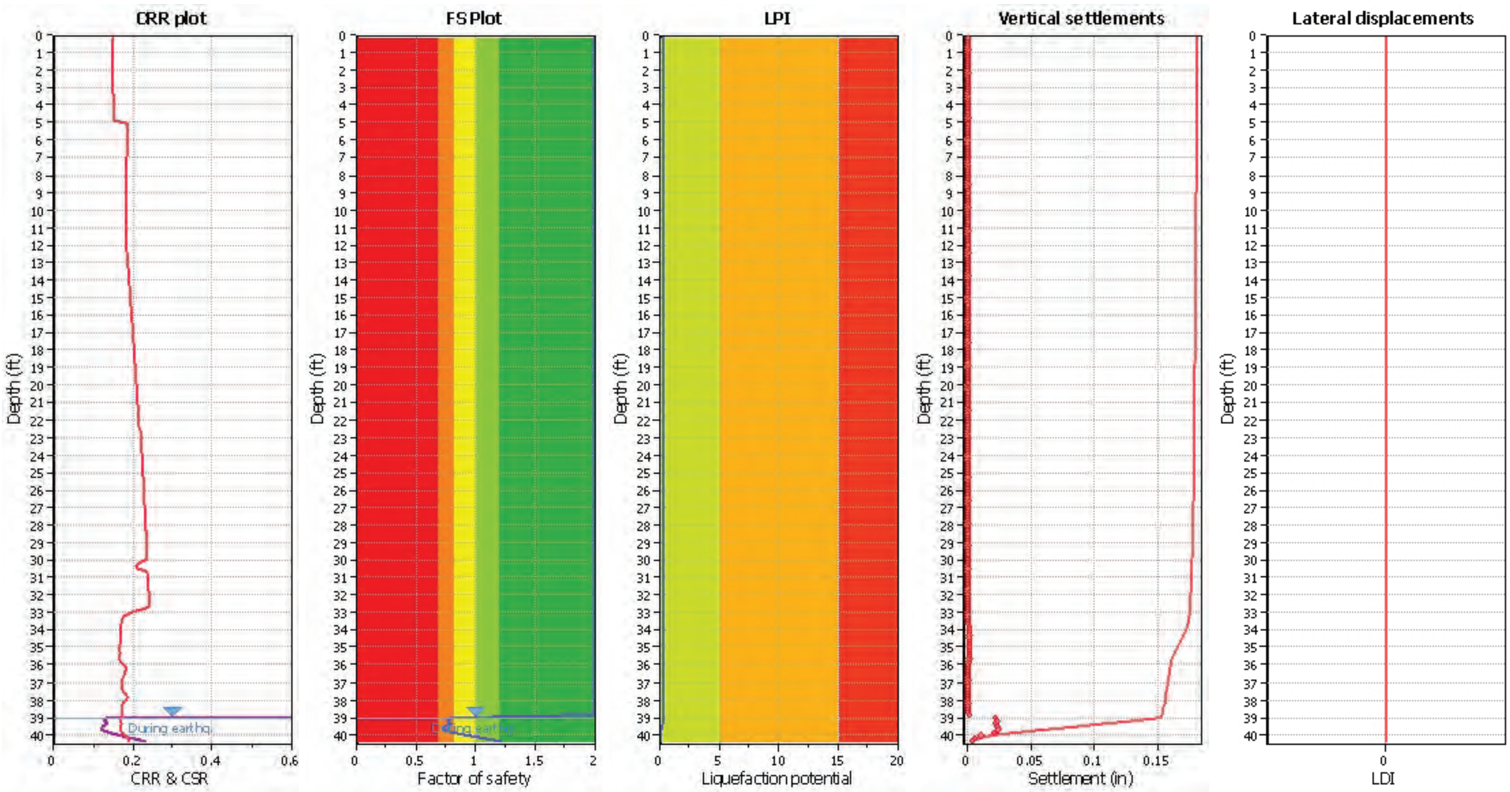
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	39.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	39.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	39.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	39.00 ft	Fill height:	N/A	Limit depth:	N/A

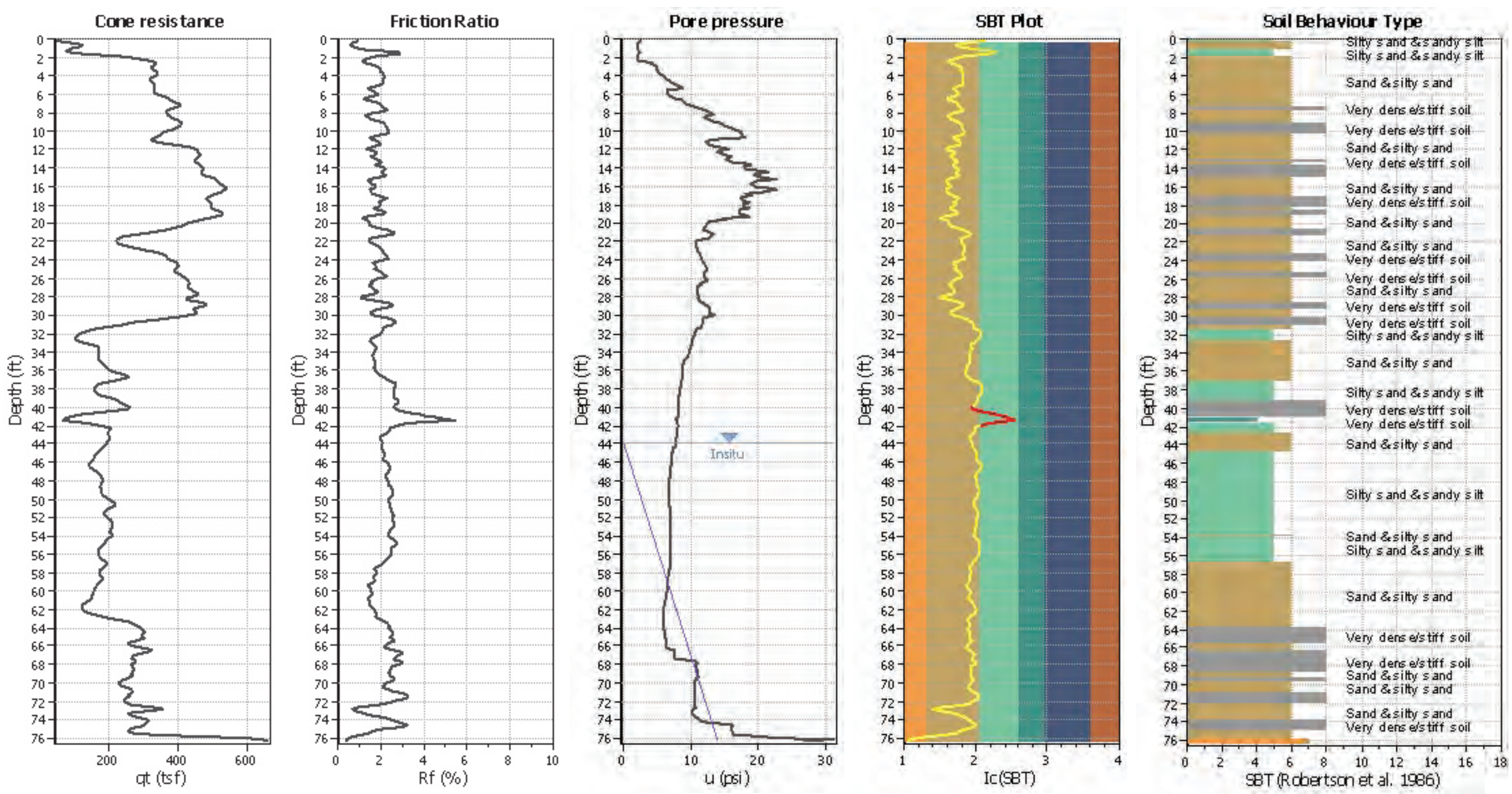
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



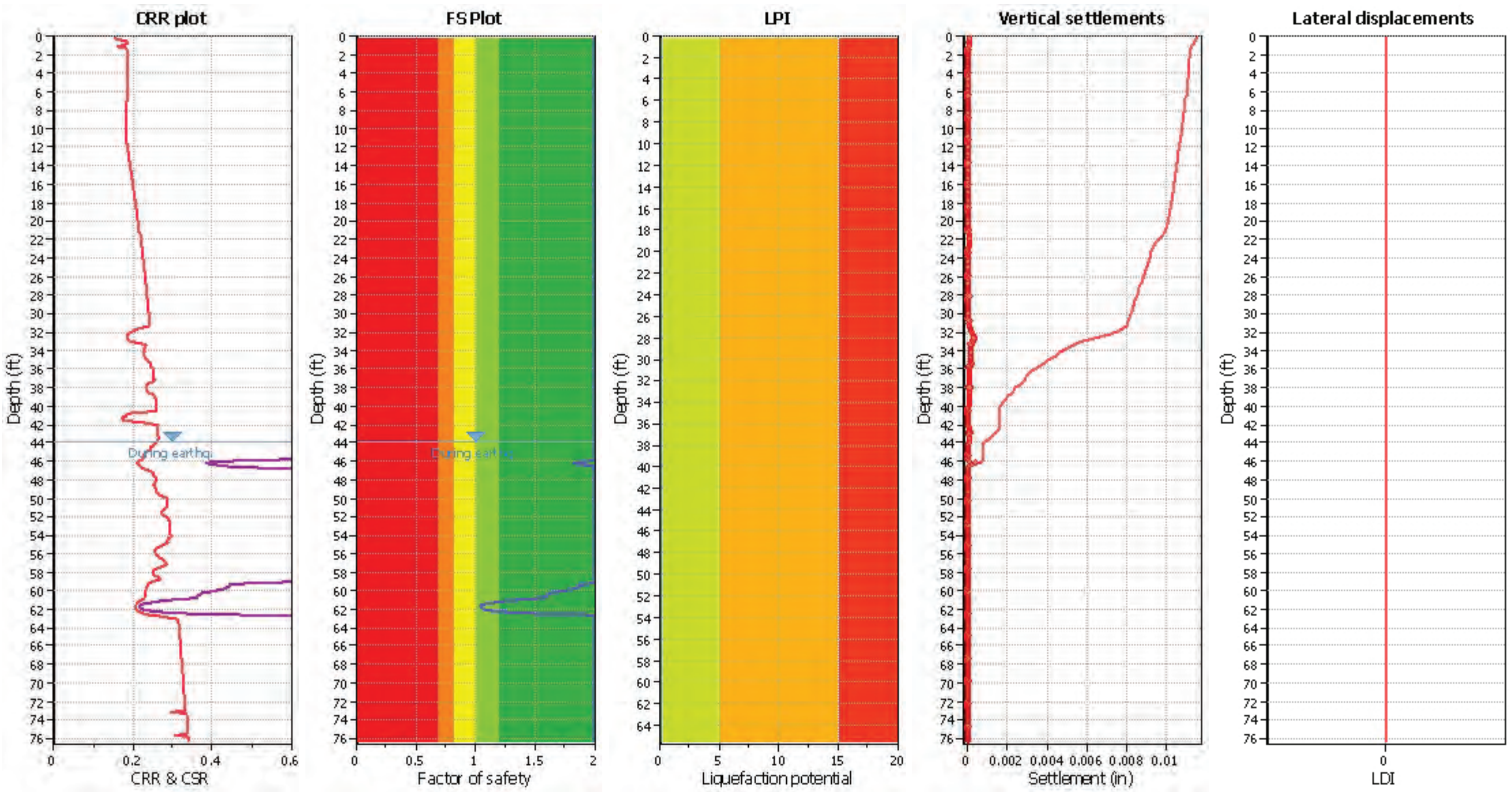
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	44.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	44.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	44.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	44.00 ft	Fill height:	N/A	Limit depth:	N/A

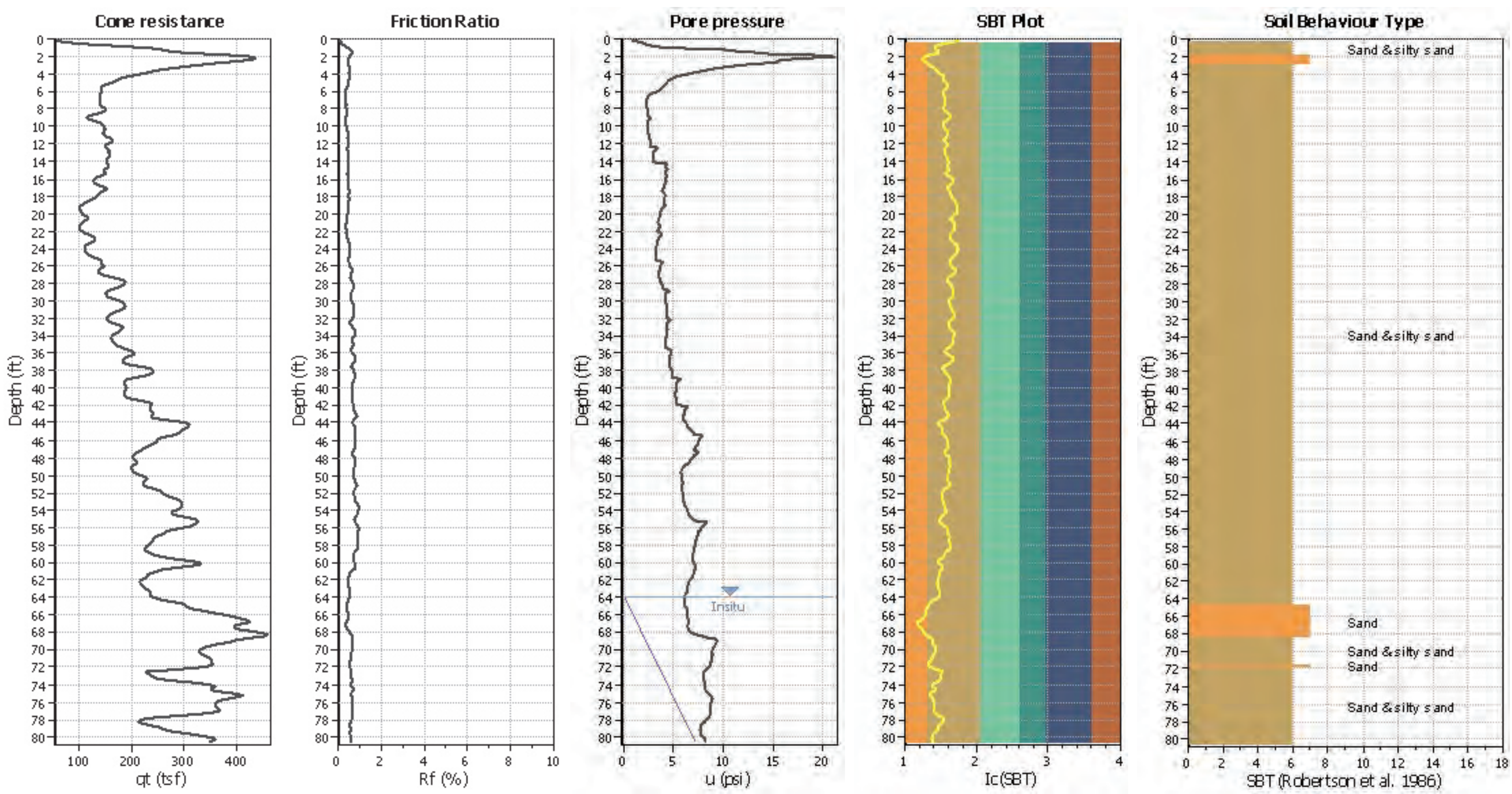
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



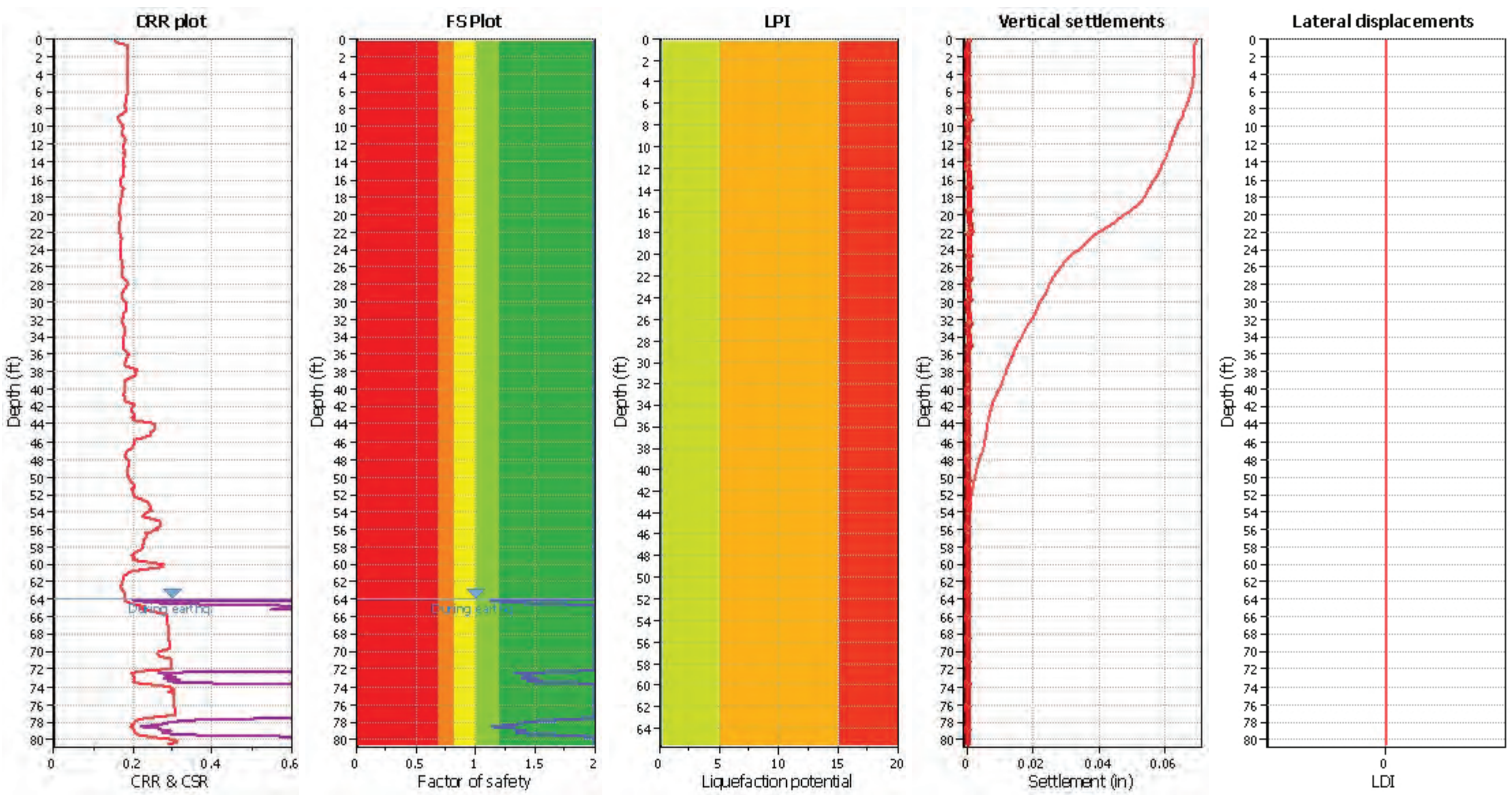
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	64.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	64.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	64.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.25	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	64.00 ft	Fill height:	N/A	Limit depth:	N/A

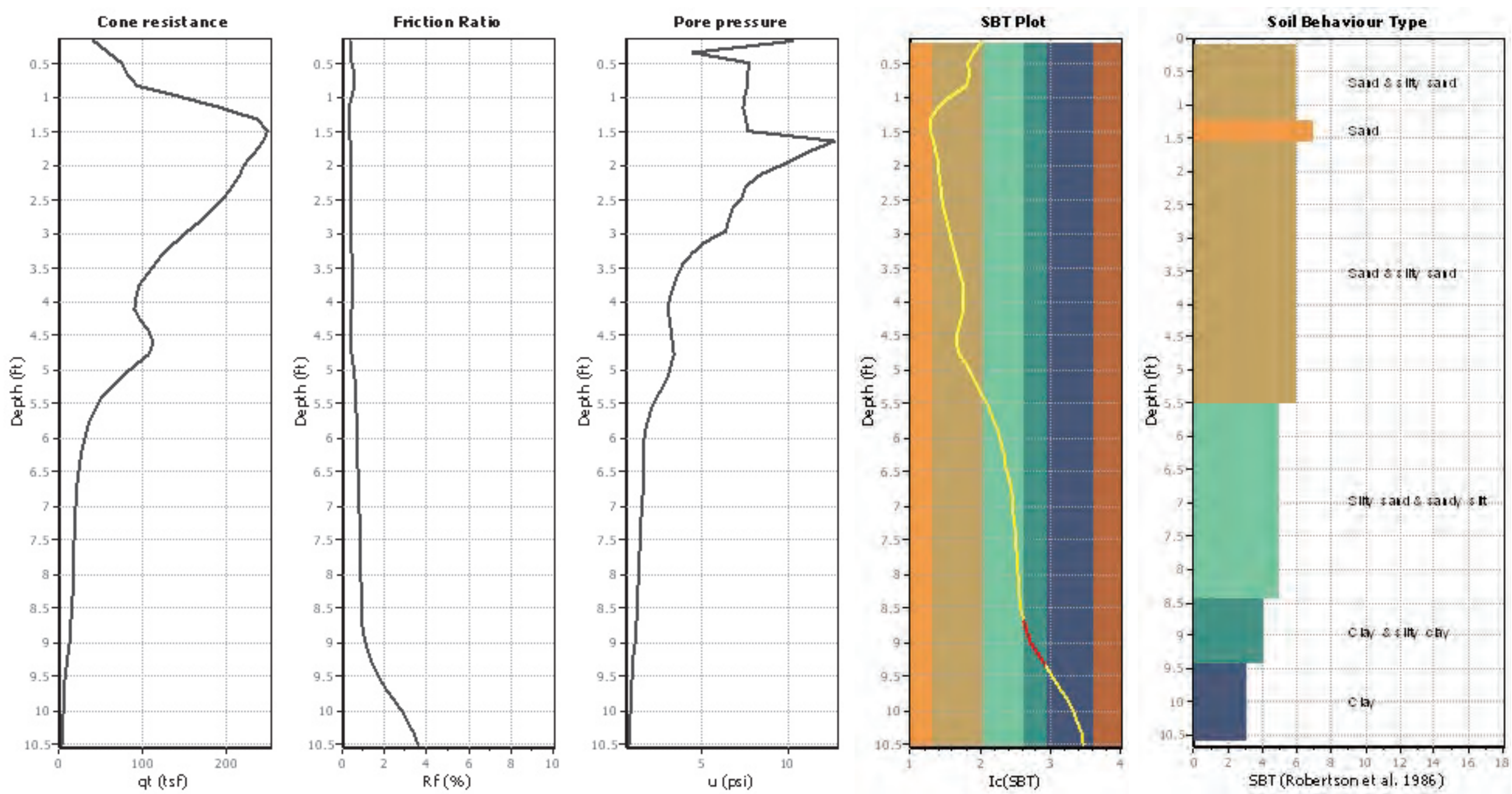
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



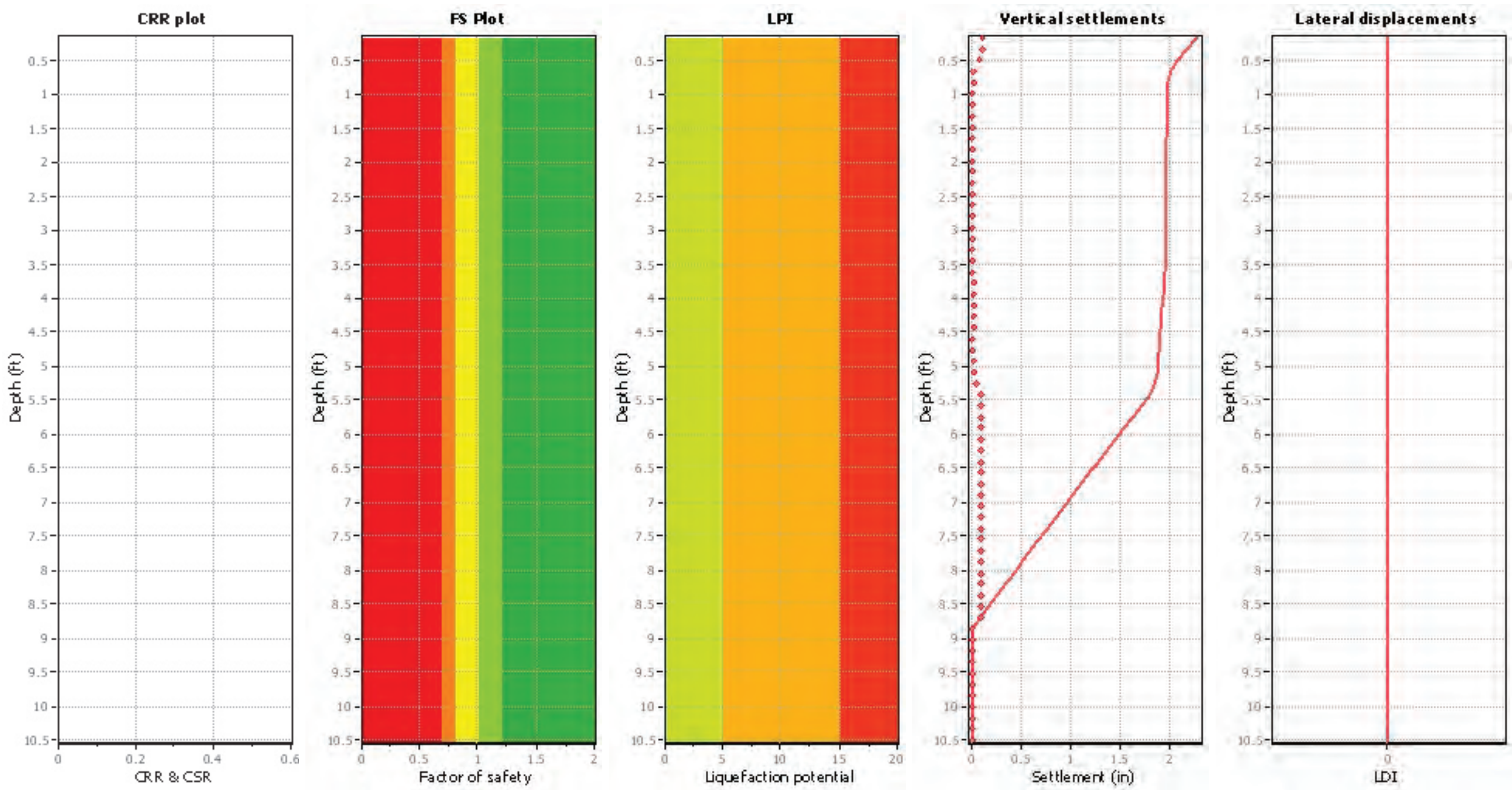
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	15.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	15.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Fill height:	N/A	Limit depth:	N/A

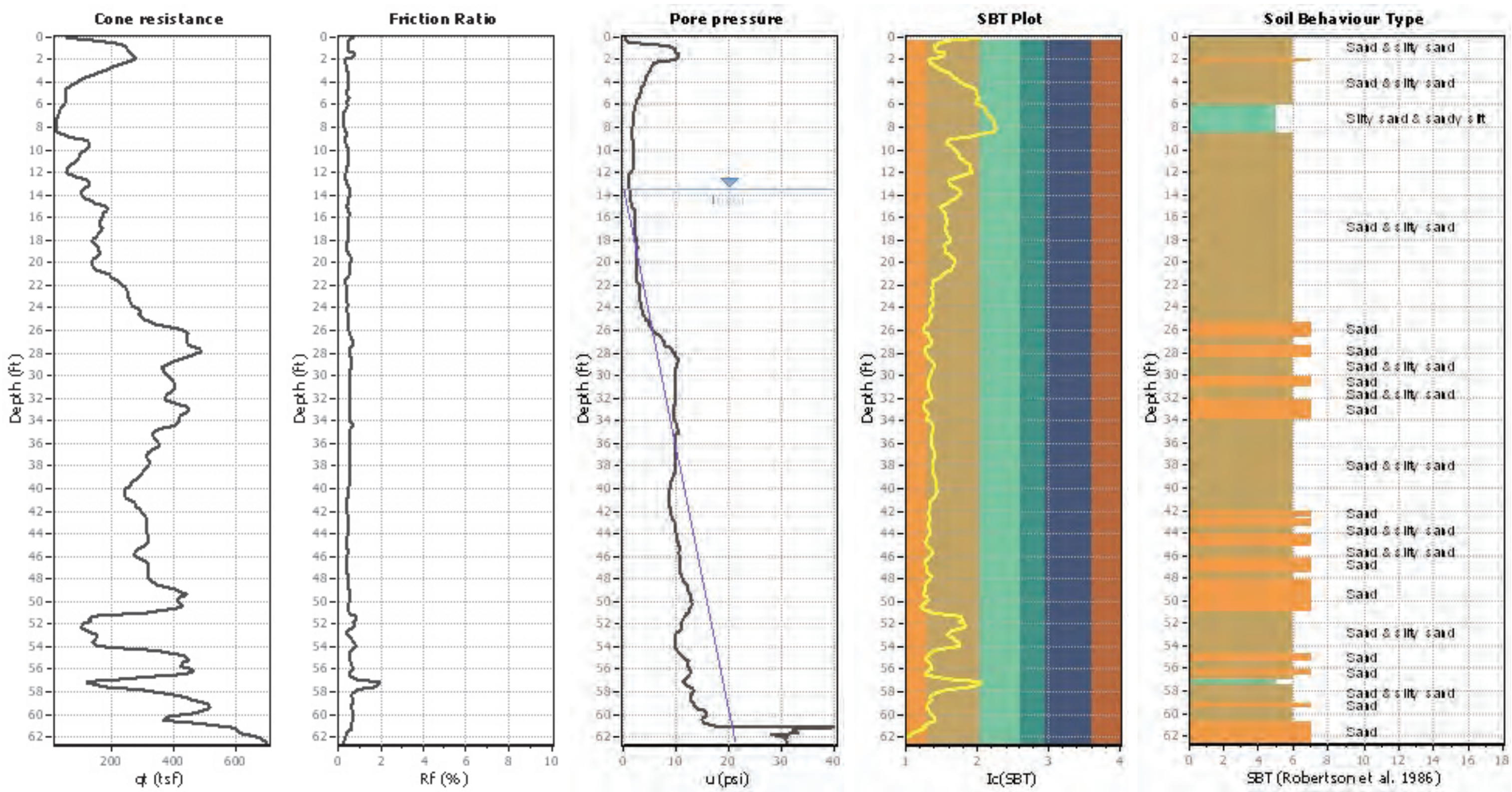
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



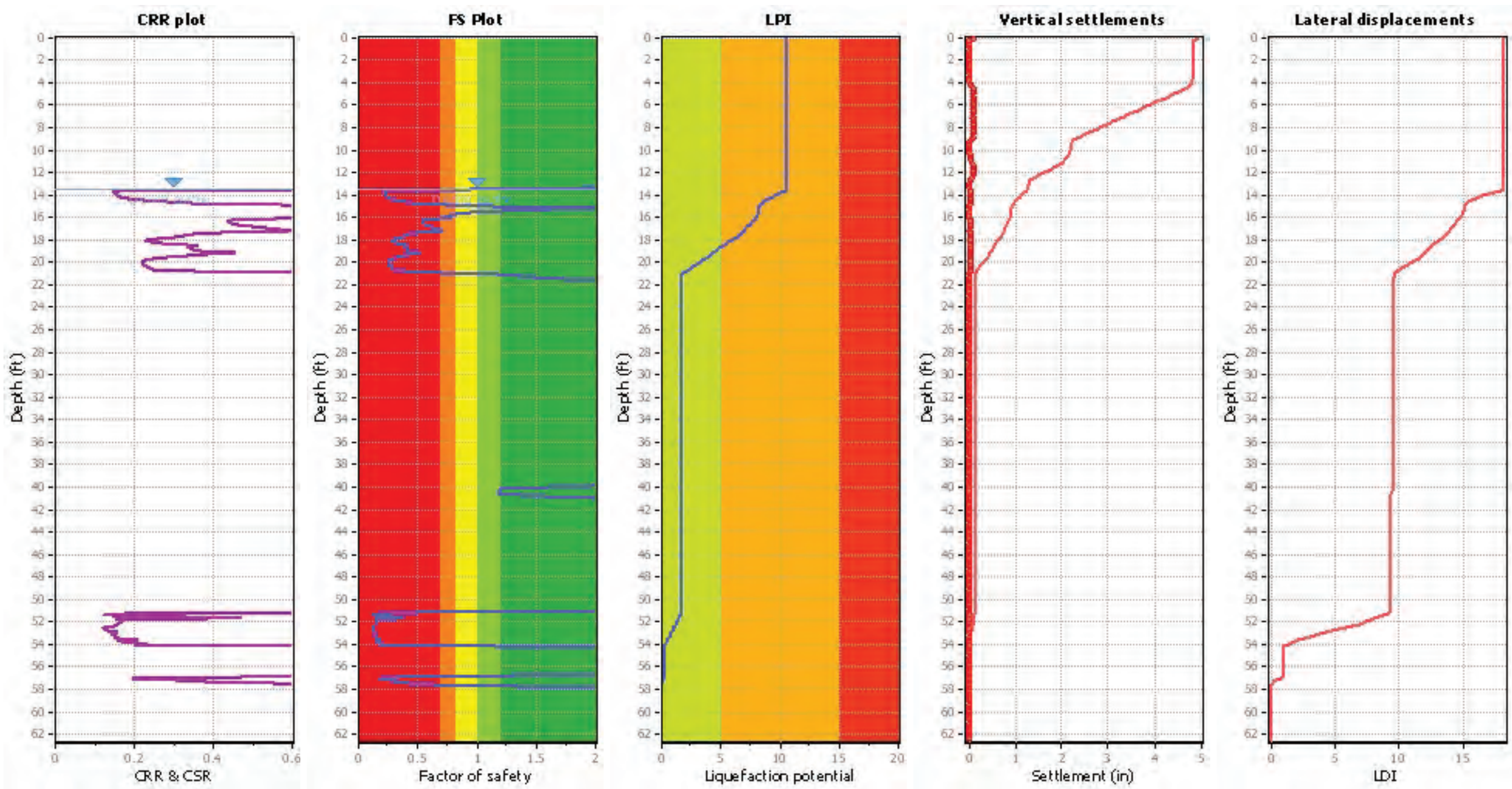
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.50 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.50 ft	Fill height:	N/A	Limit depth:	N/A

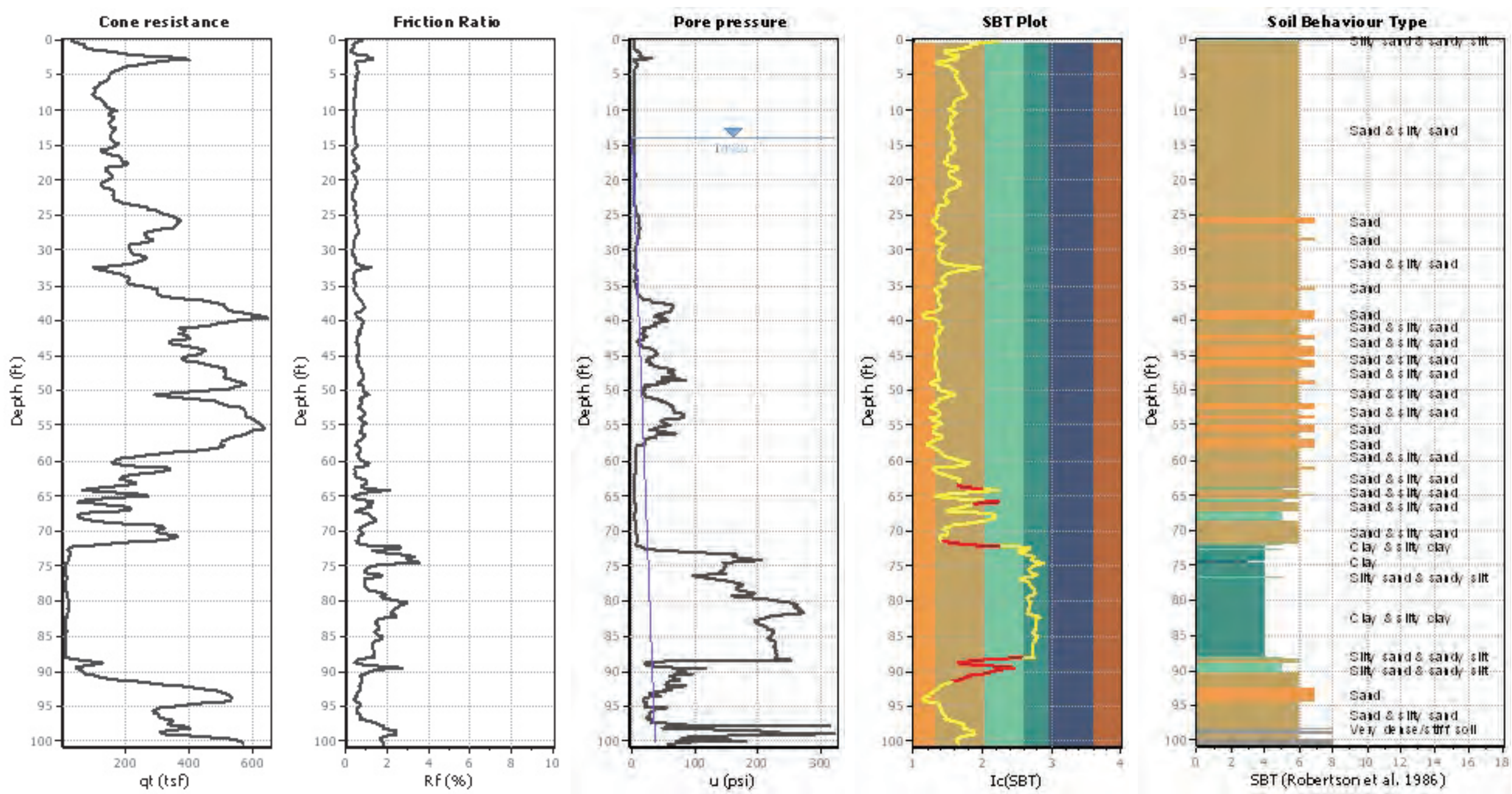
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



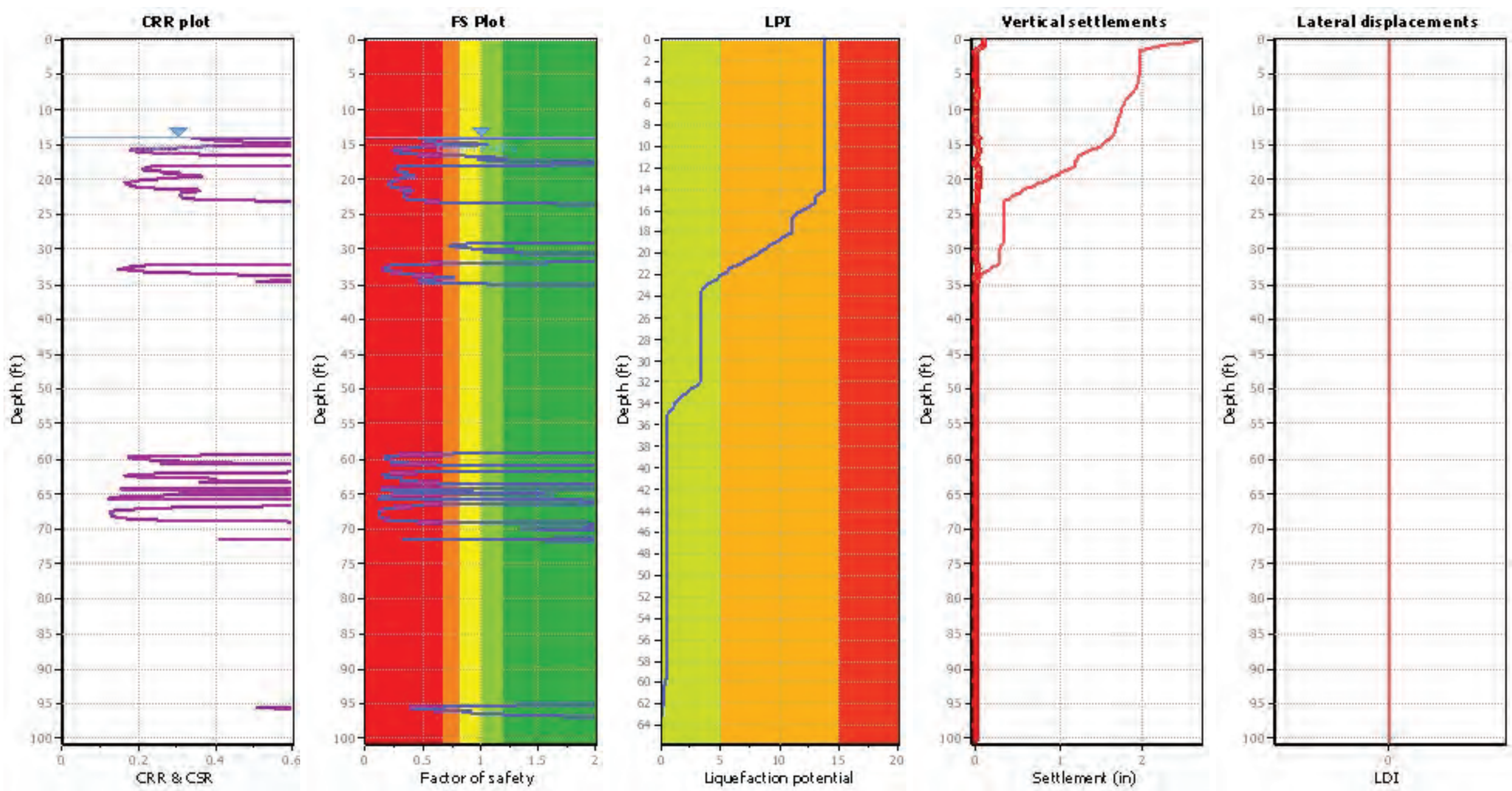
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	14.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to clayey sand
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	14.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Exhibit 8
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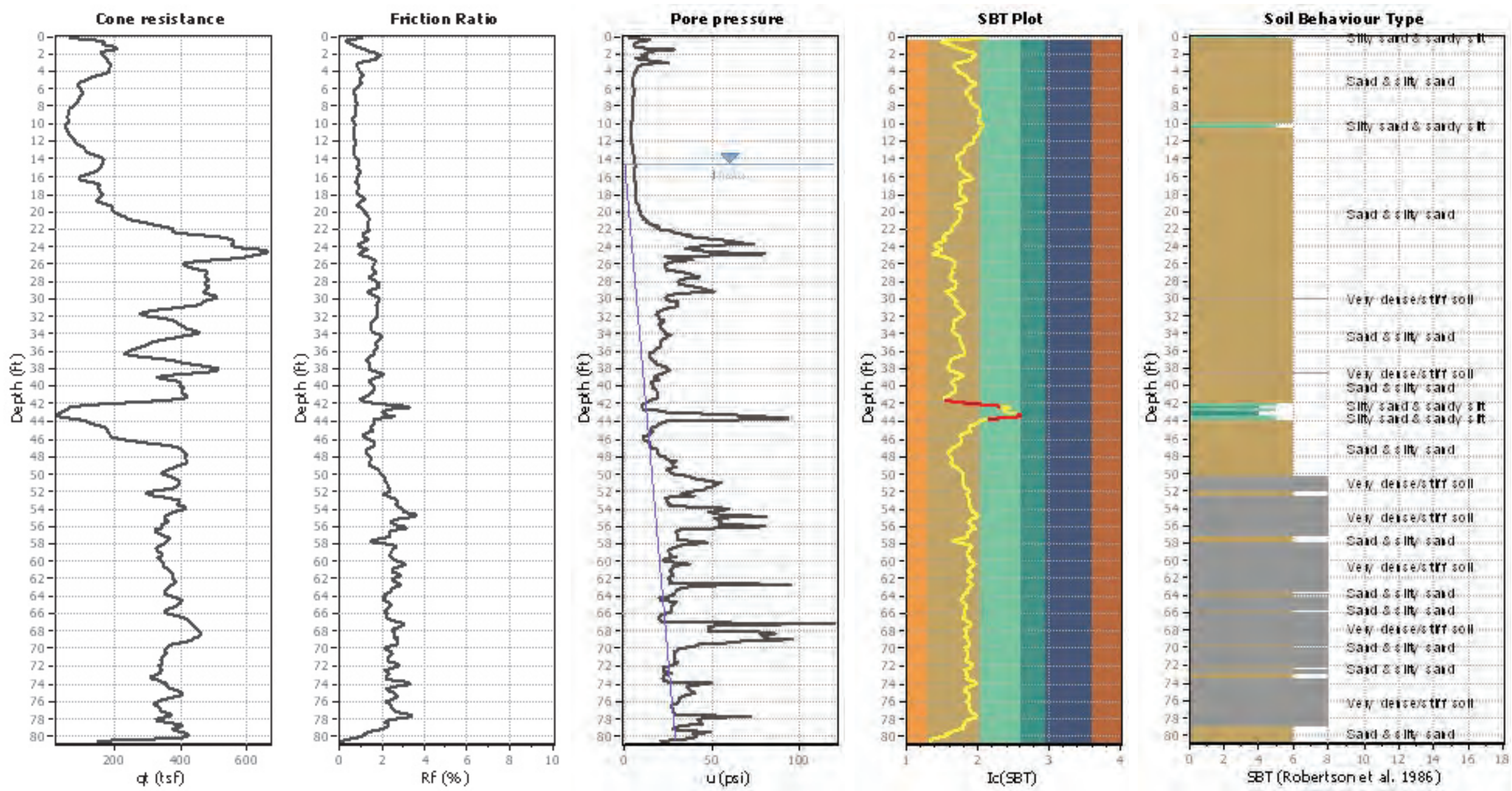
CPT basic interpretation plo



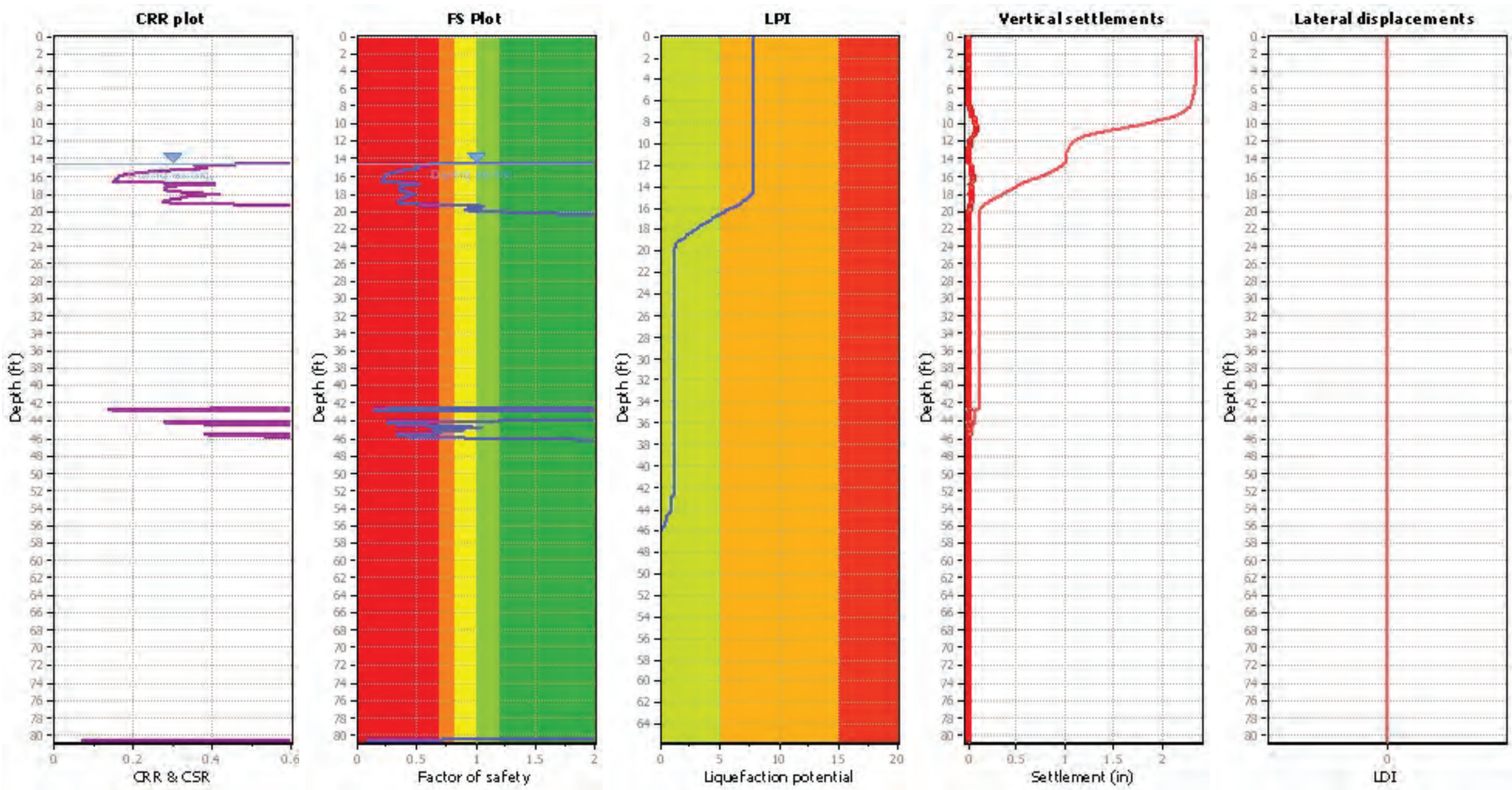
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	14.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.50 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	14.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.50 ft	Fill height:	N/A	Limit depth:	N/A

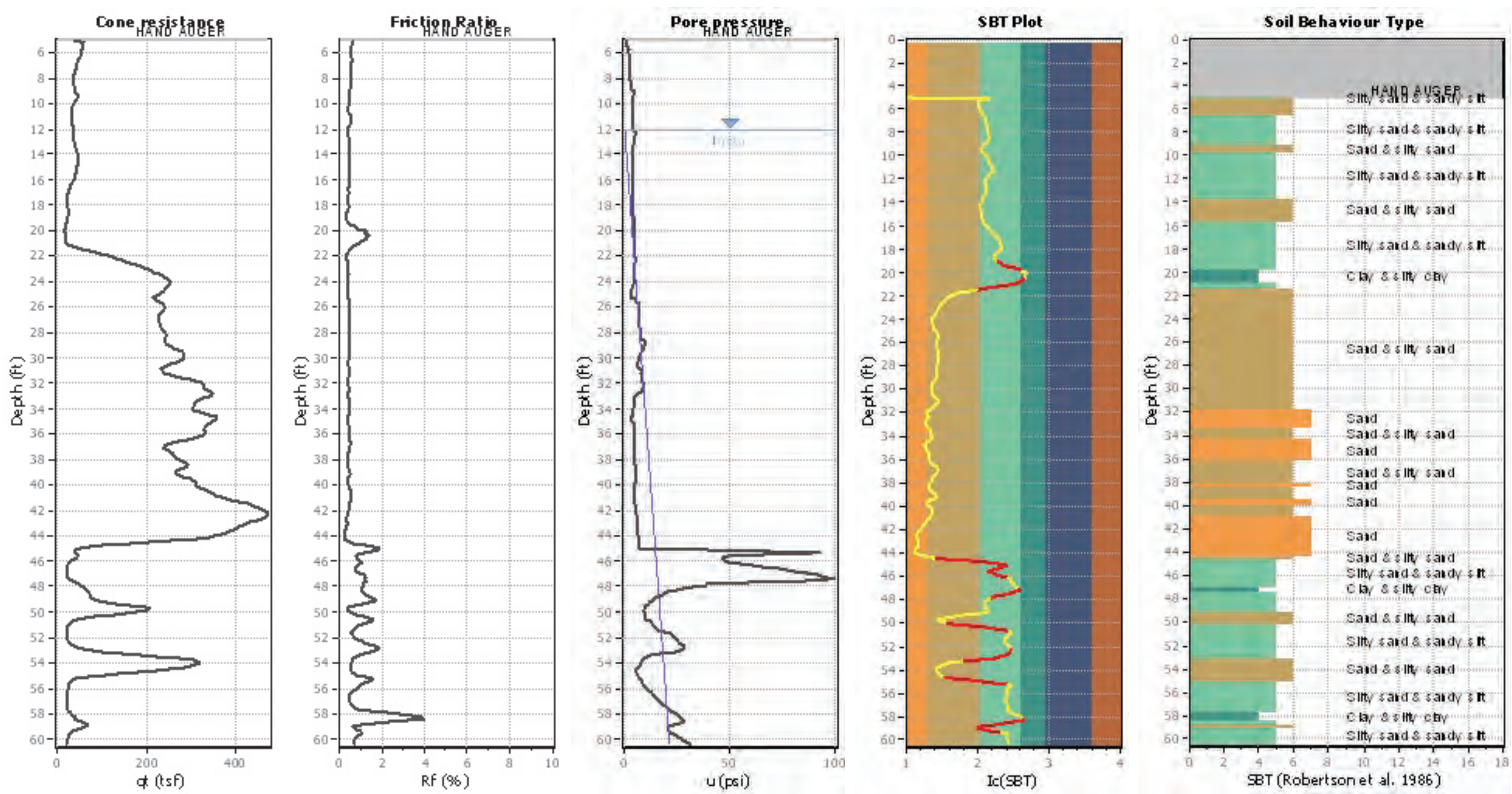
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



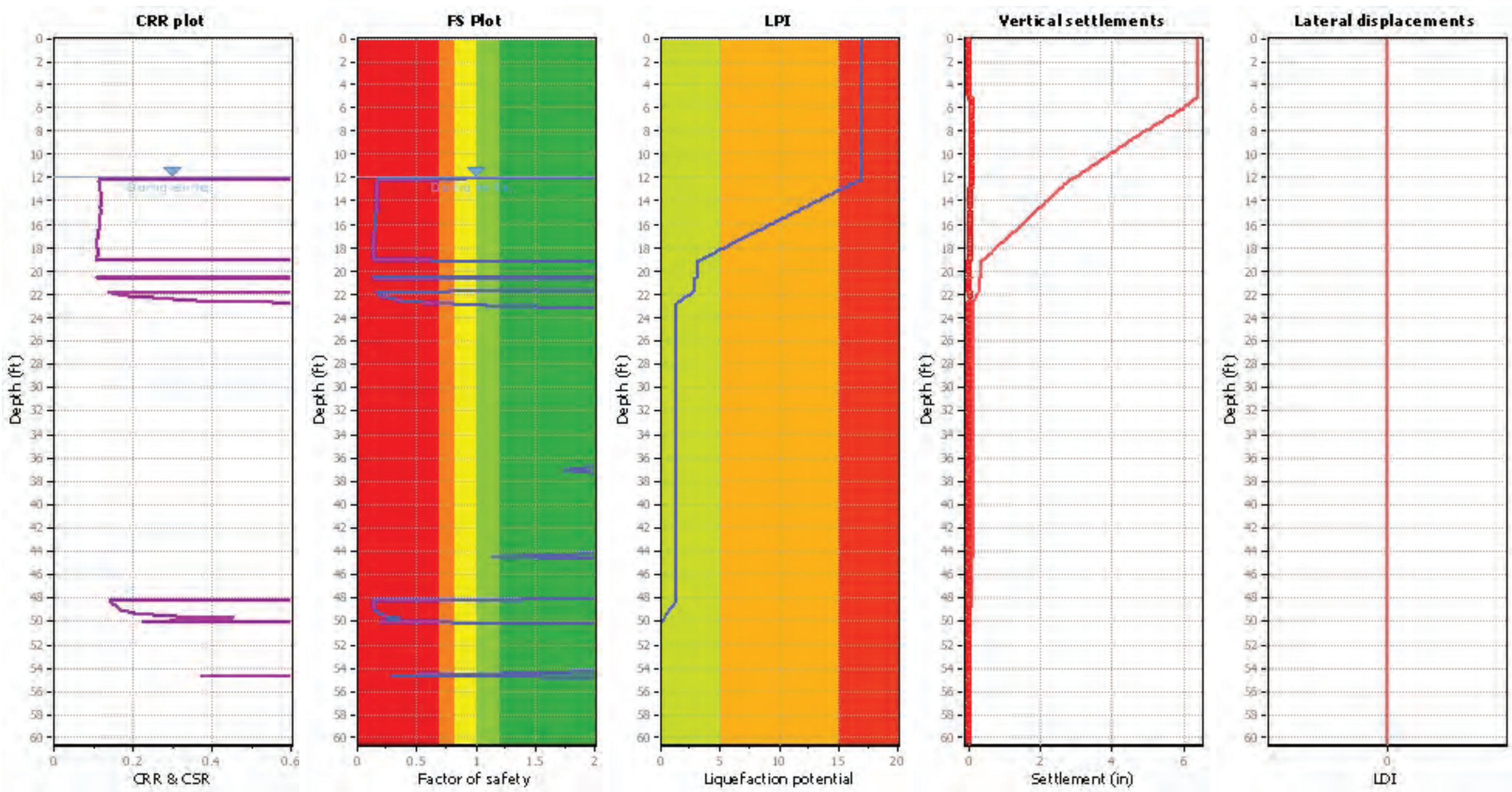
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	12.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	12.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	12.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	12.00 ft	Fill height:	N/A	Limit depth:	N/A

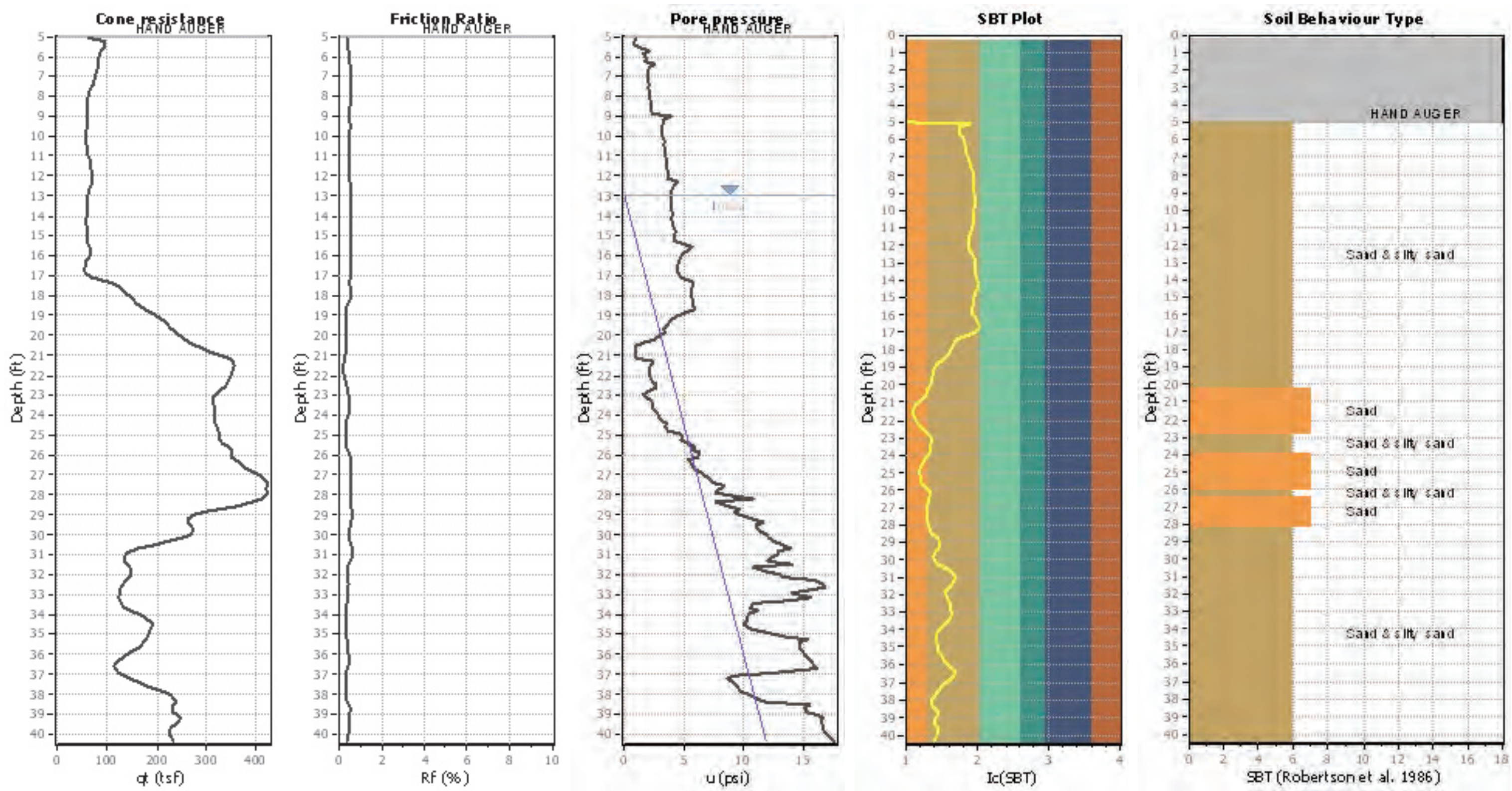
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

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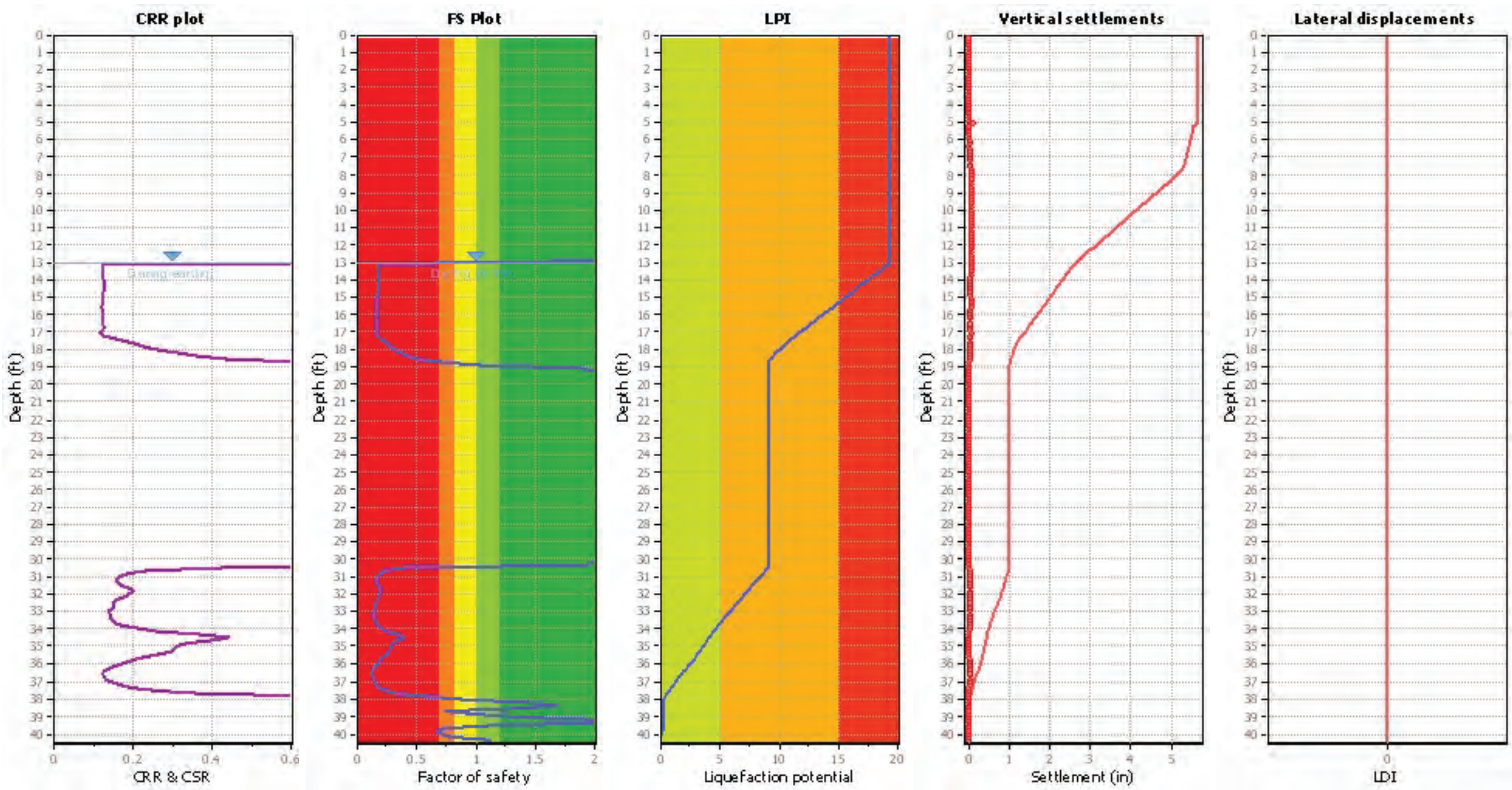
Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme
- Almost certain it will liquefy
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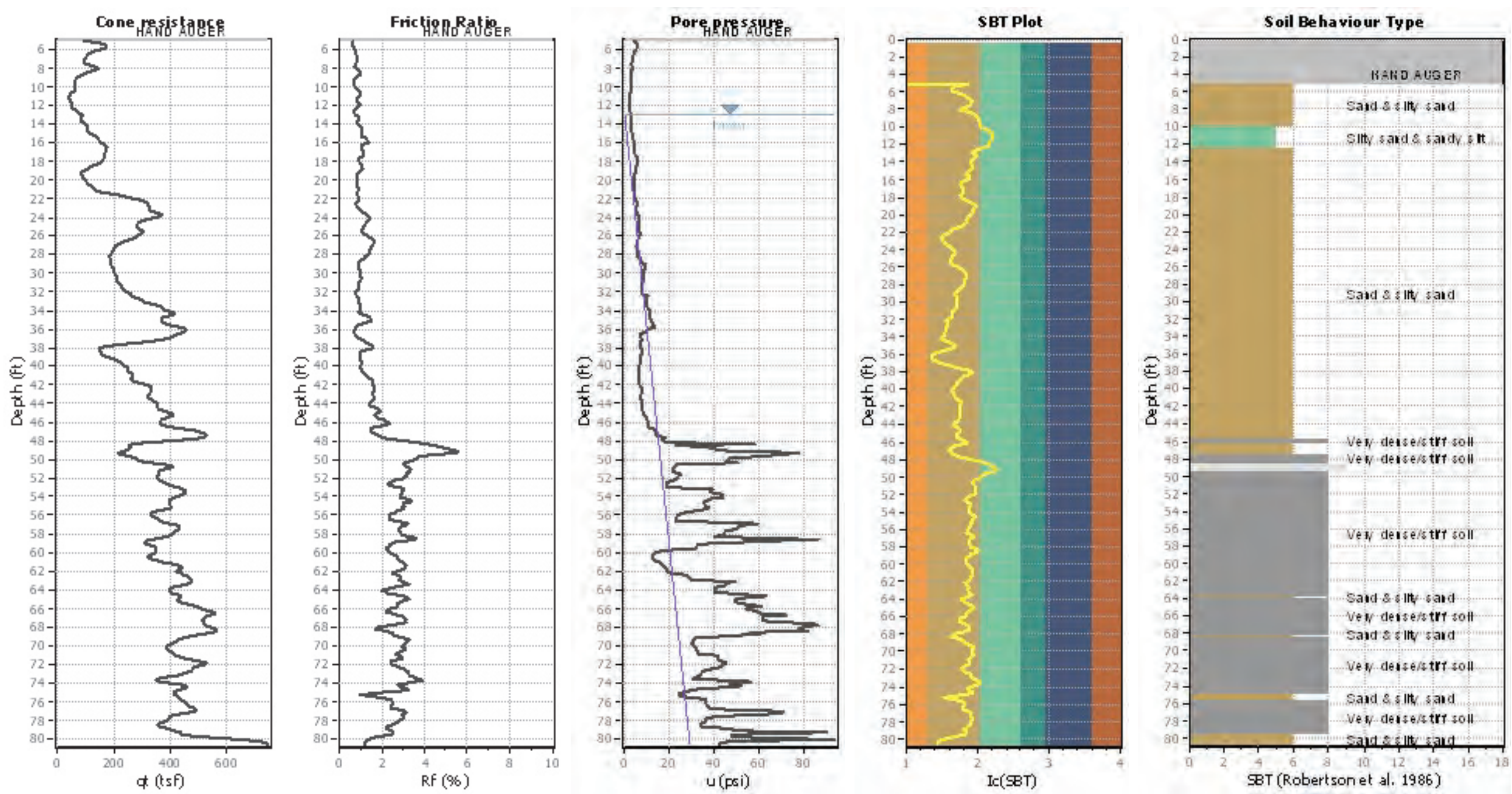
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme
- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



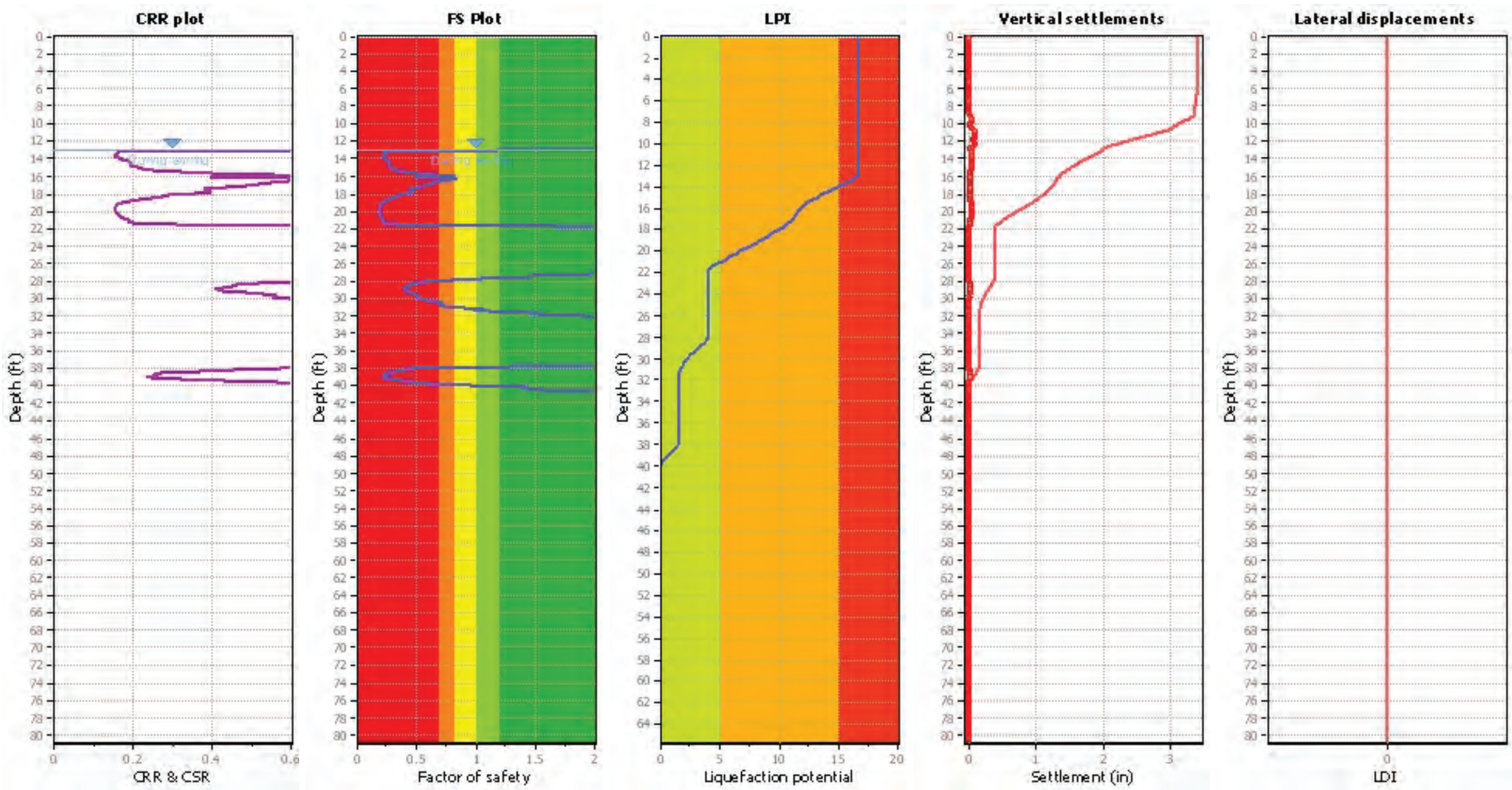
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.00 ft	Fill weight:	N/A		
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes		
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes		
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only		
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No		
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A		

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A

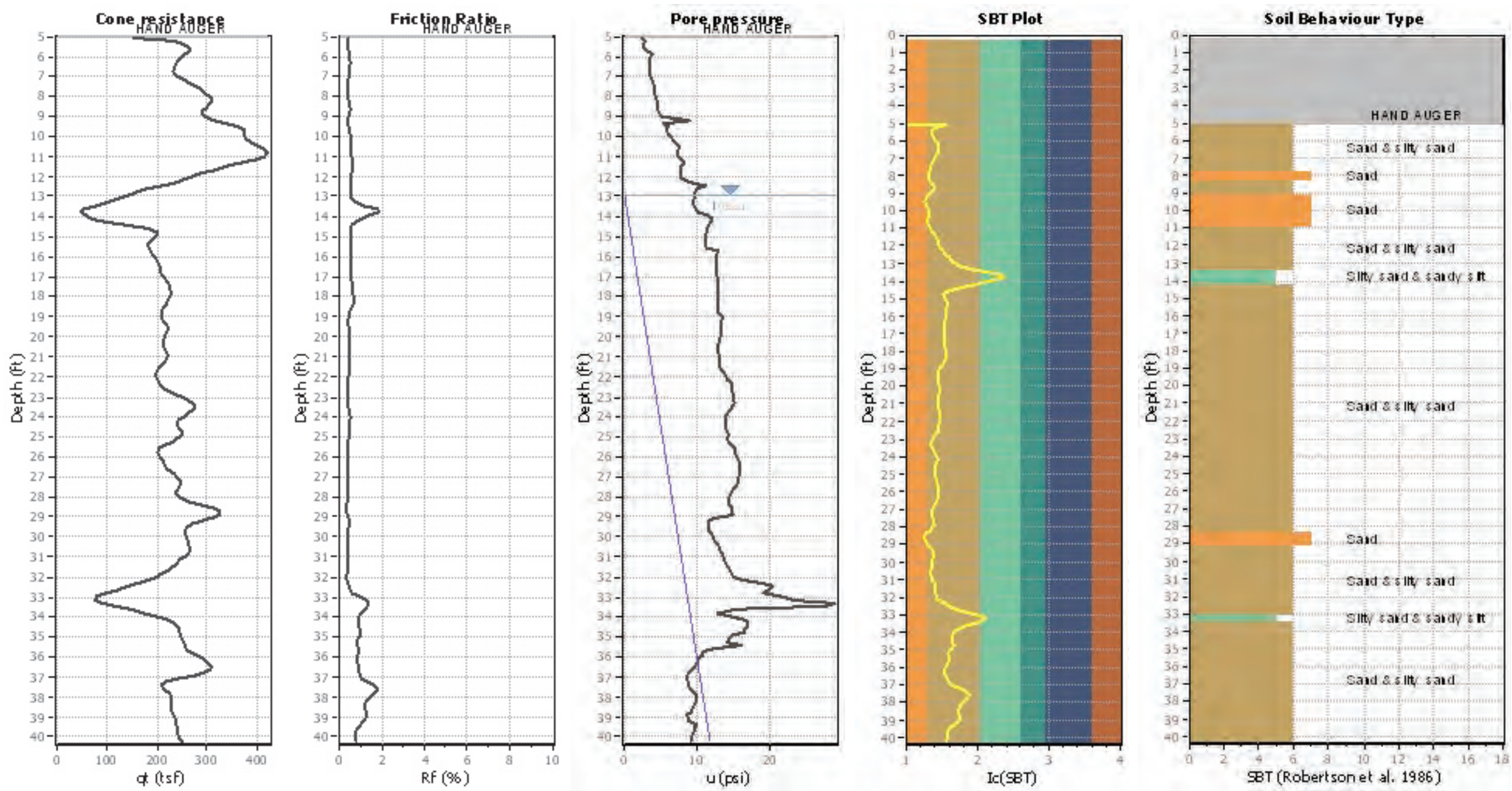
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



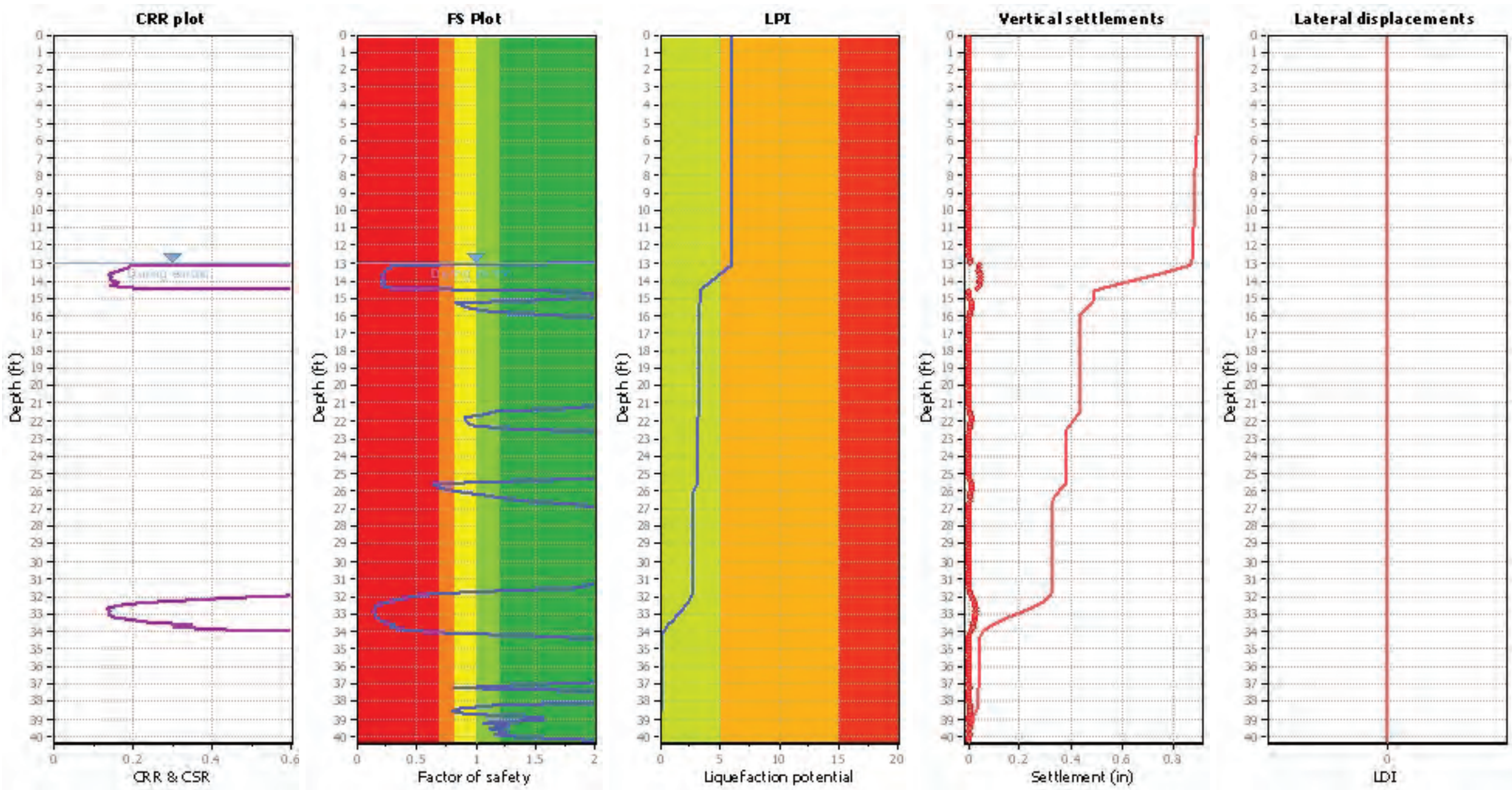
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A

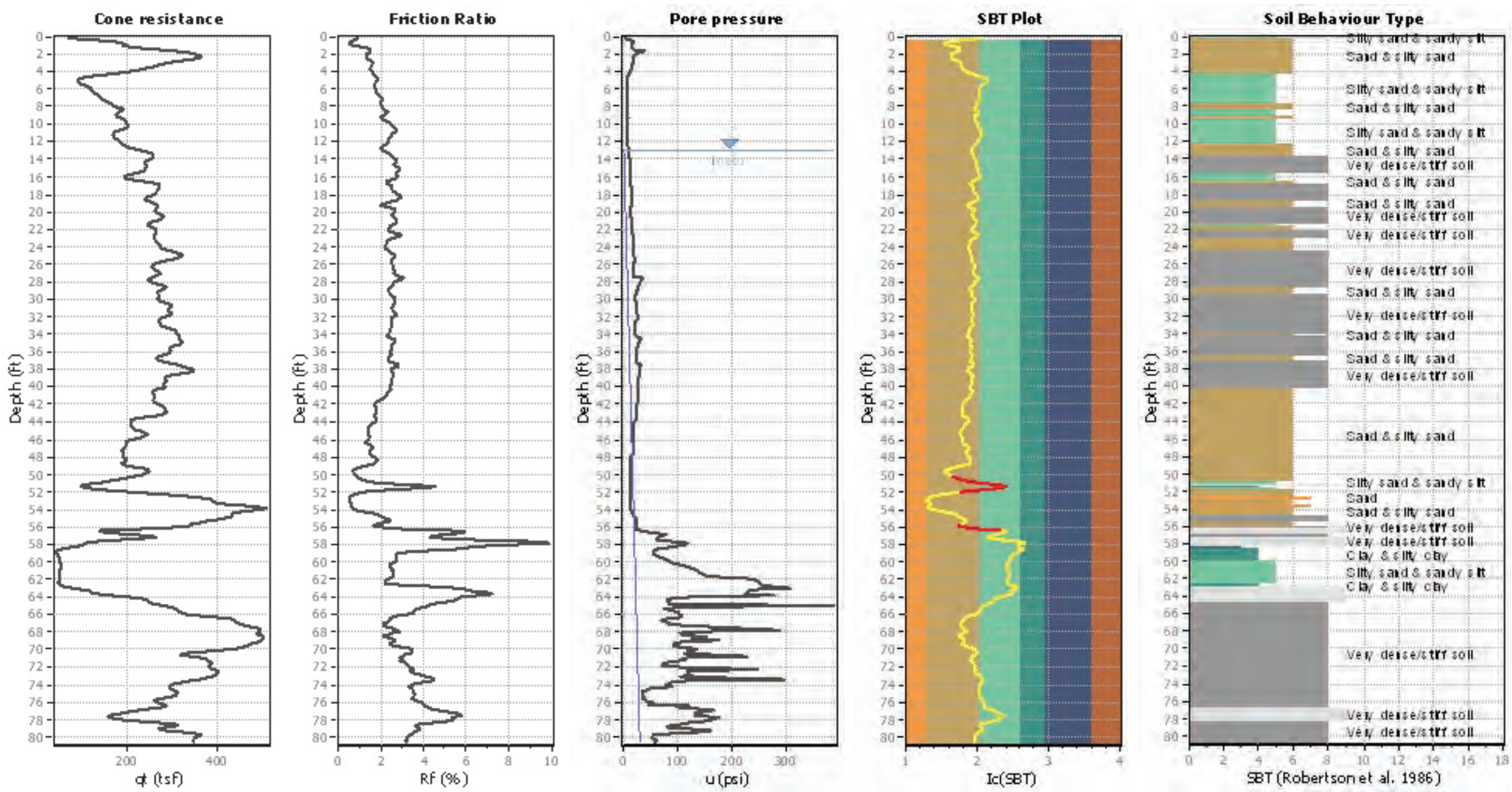
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



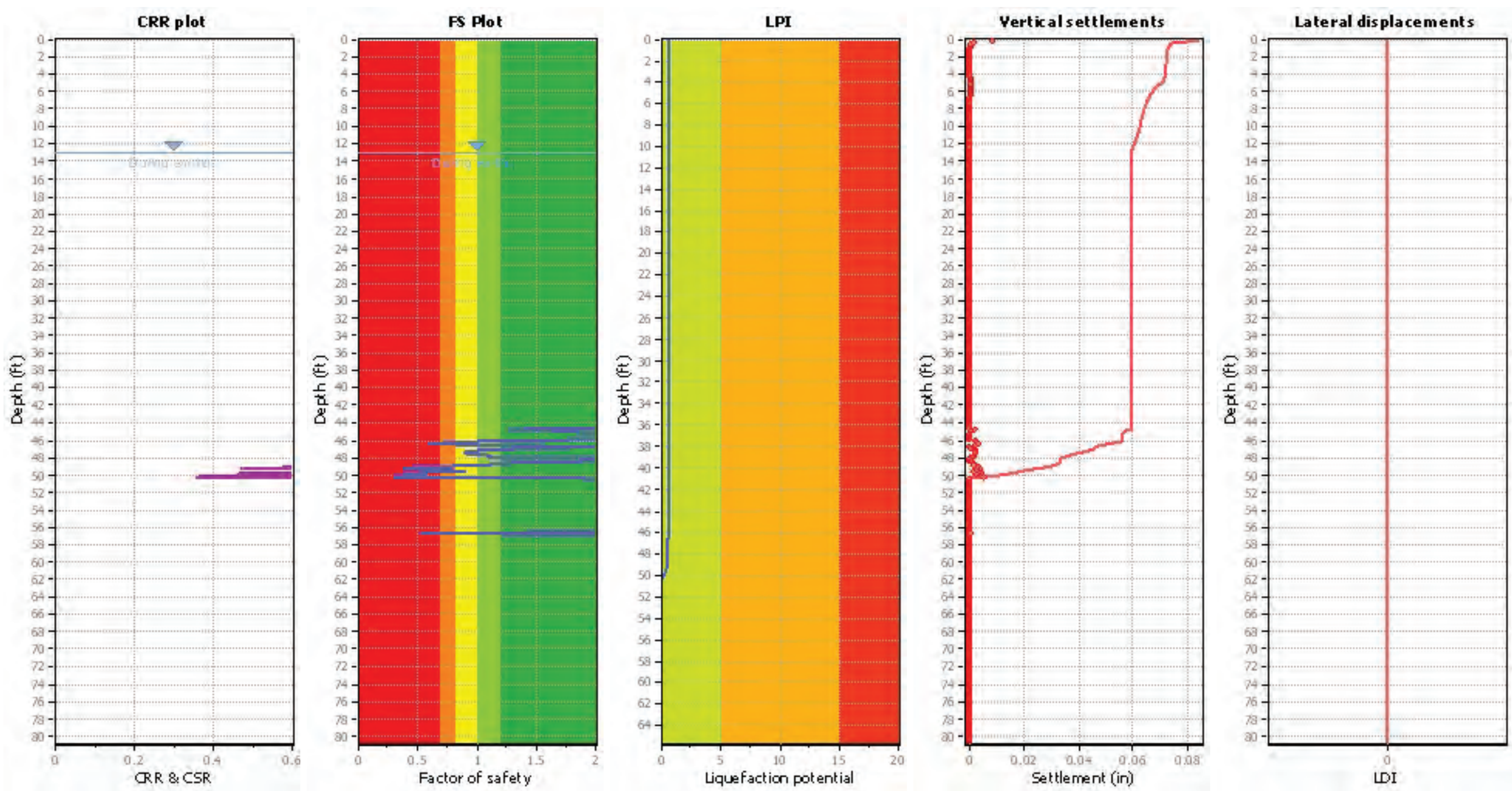
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	13.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A

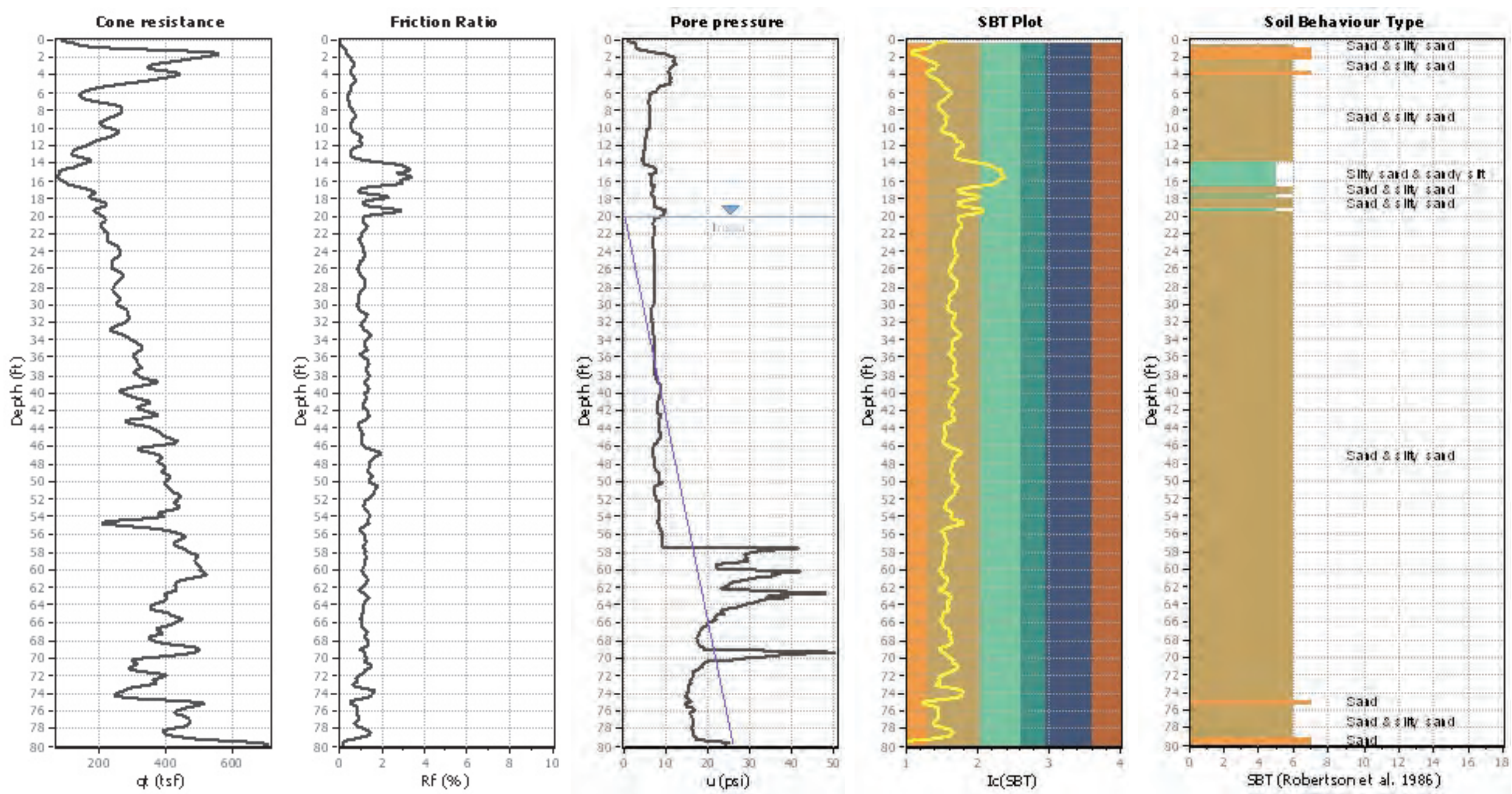
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



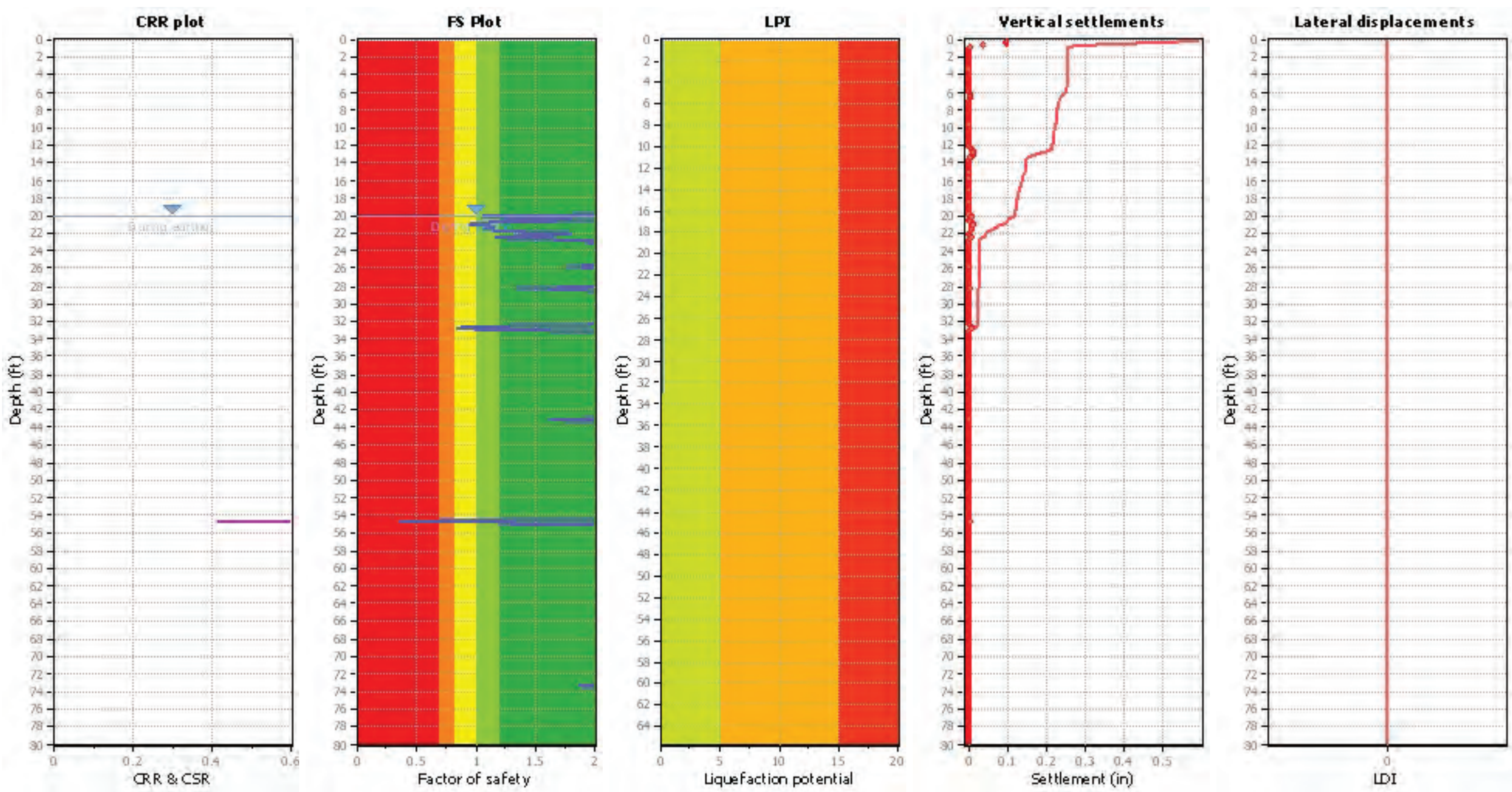
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	20.00 ft	Fill weight:	N/A		
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes		
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes		
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only		
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No		
Depth to water table (insitu):	20.00 ft	Fill height:	N/A	Limit depth:	N/A		

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



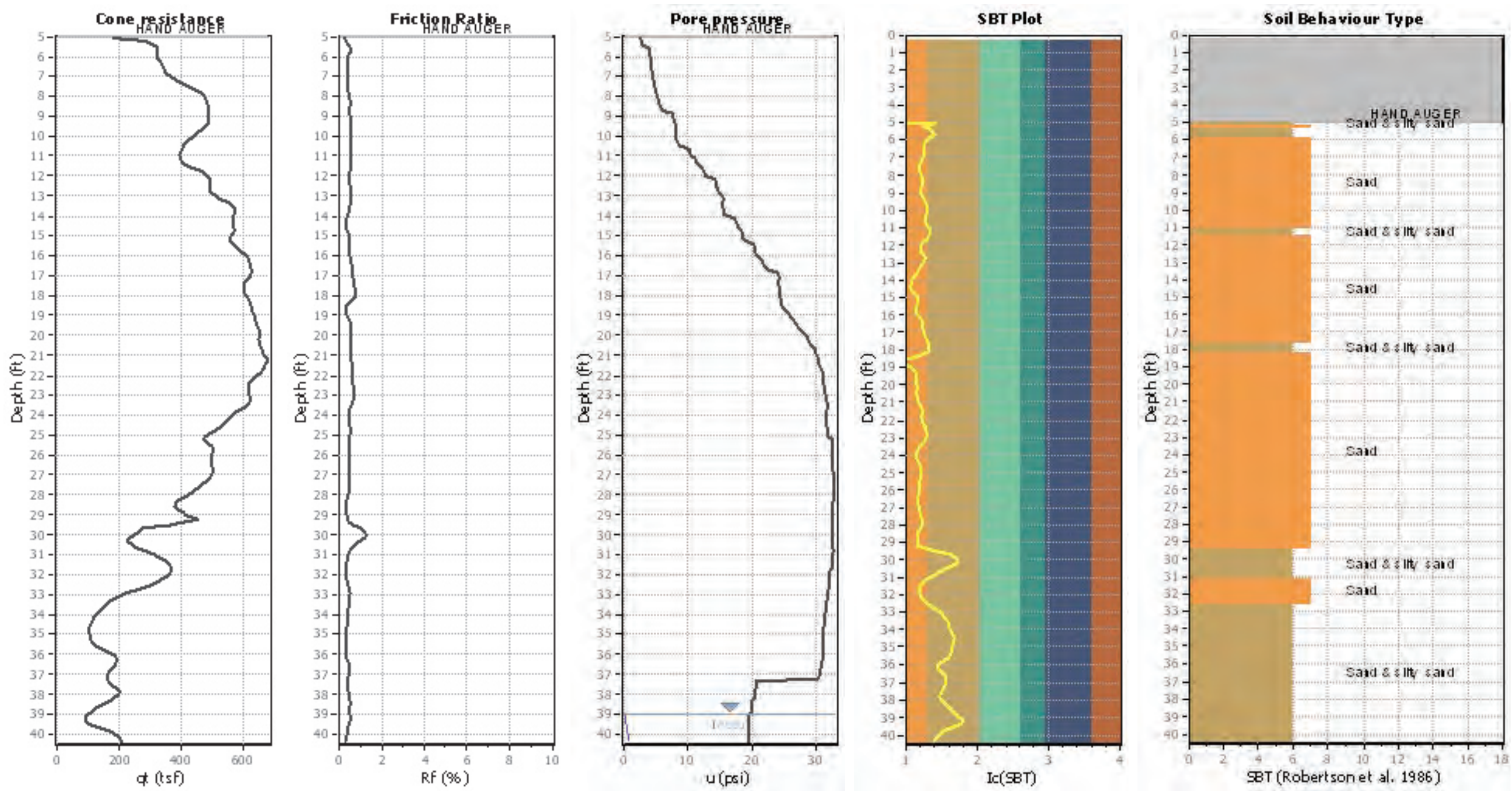
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	20.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	20.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme
- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



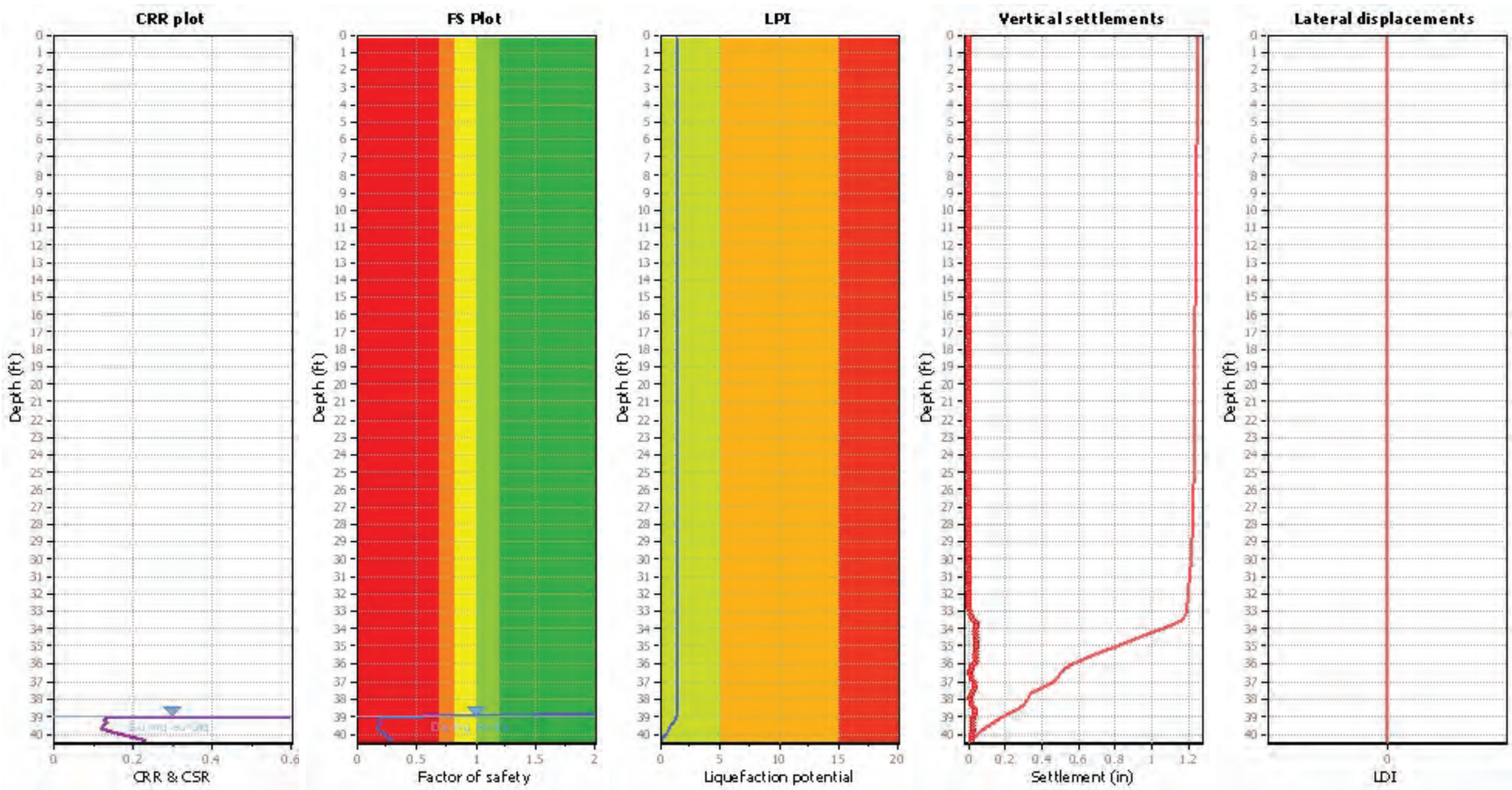
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	39.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	39.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	39.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	39.00 ft	Fill height:	N/A	Limit depth:	N/A

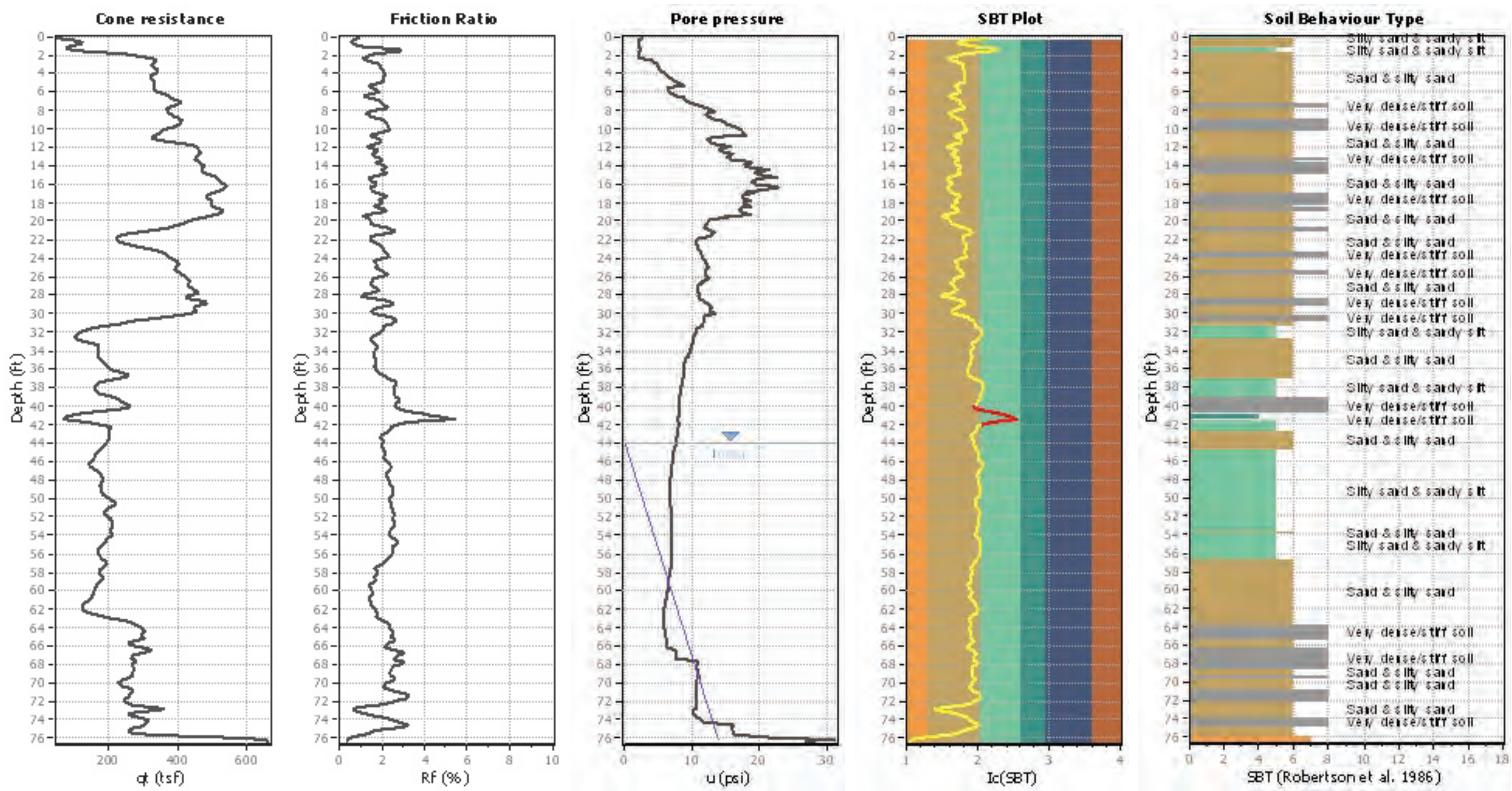
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



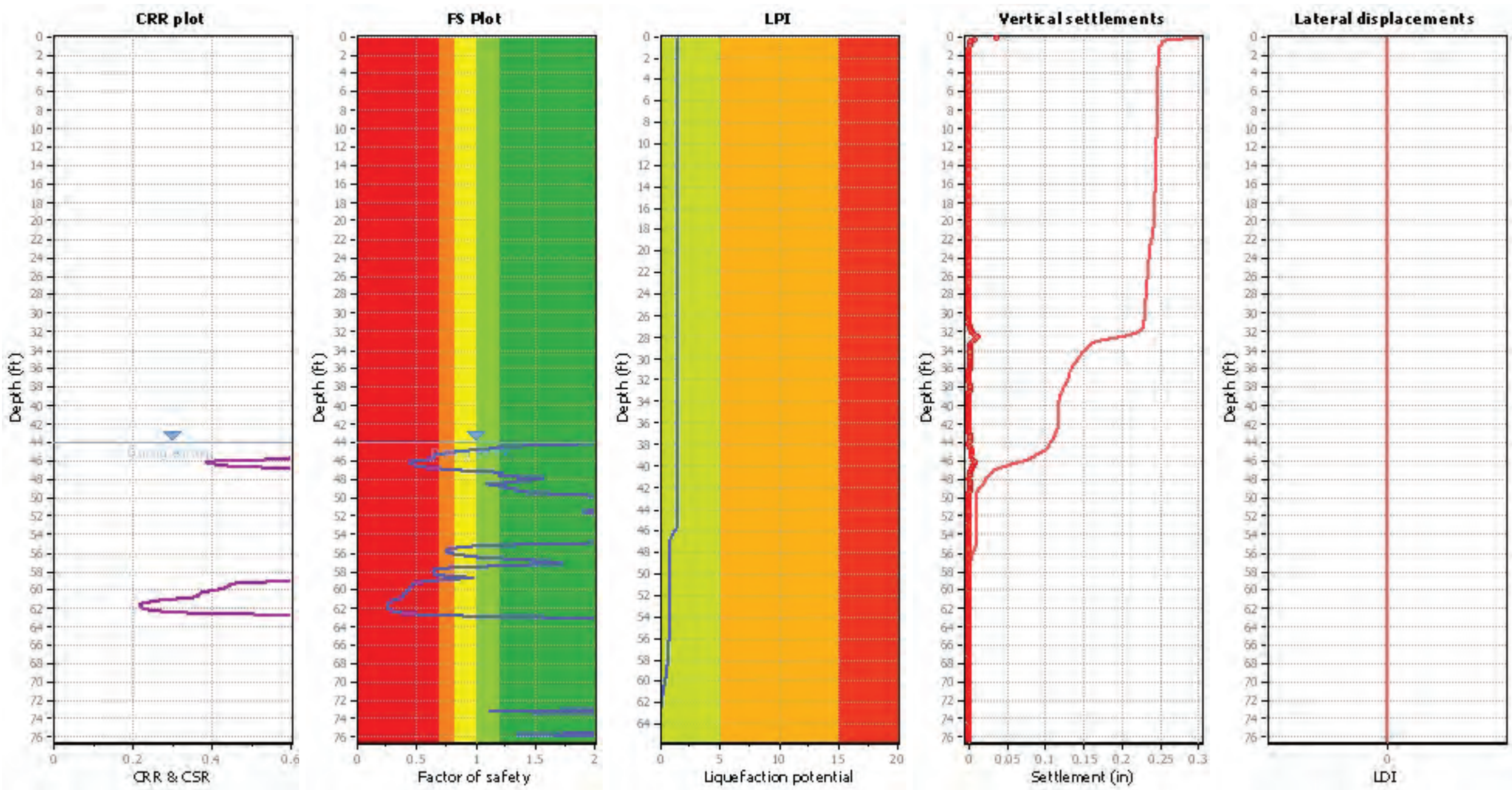
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	44.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	44.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



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Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	44.00 ft	Fill height:	N/A	Limit depth:	N/A

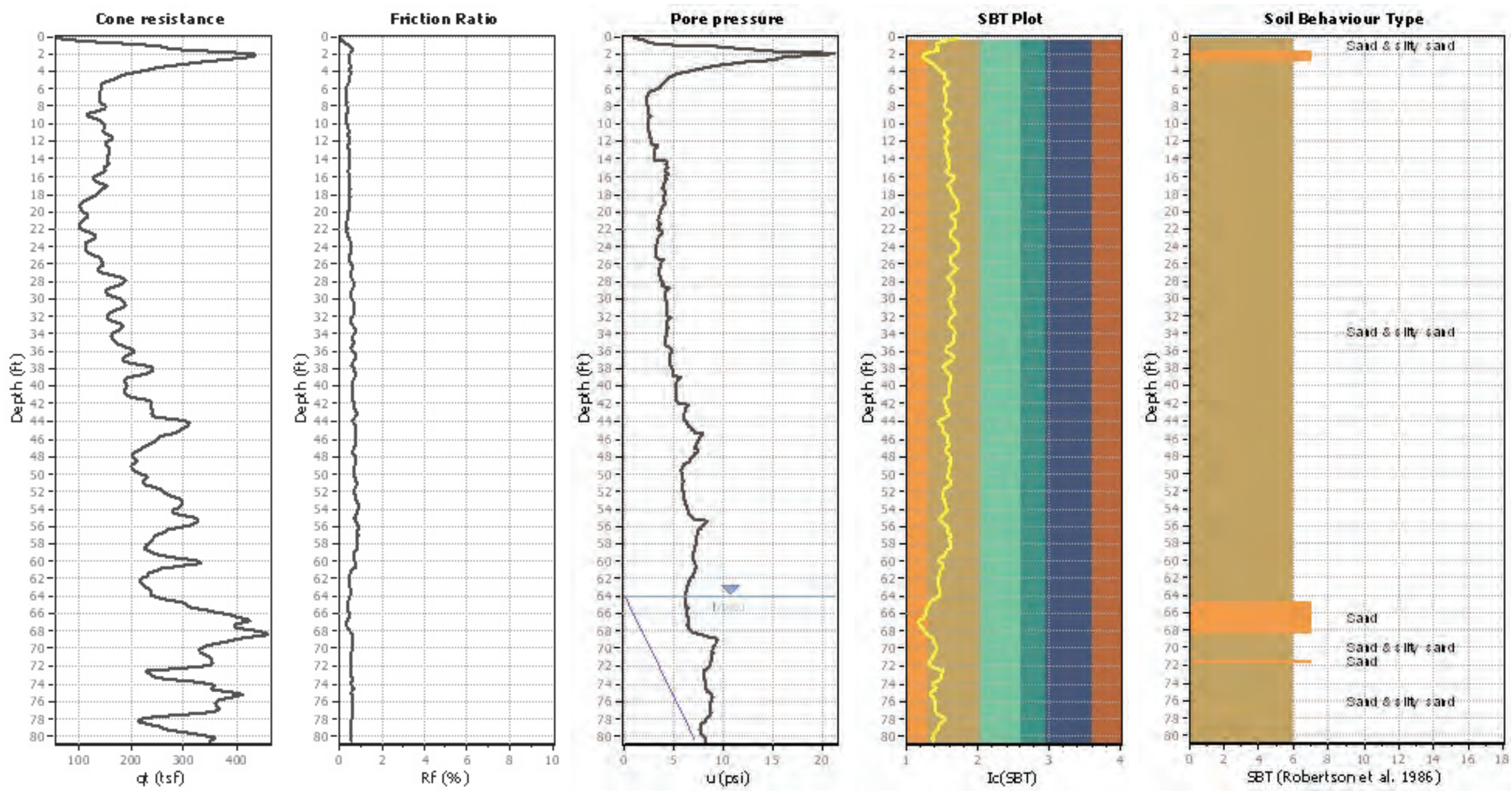
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plo



Input parameters and analysis data

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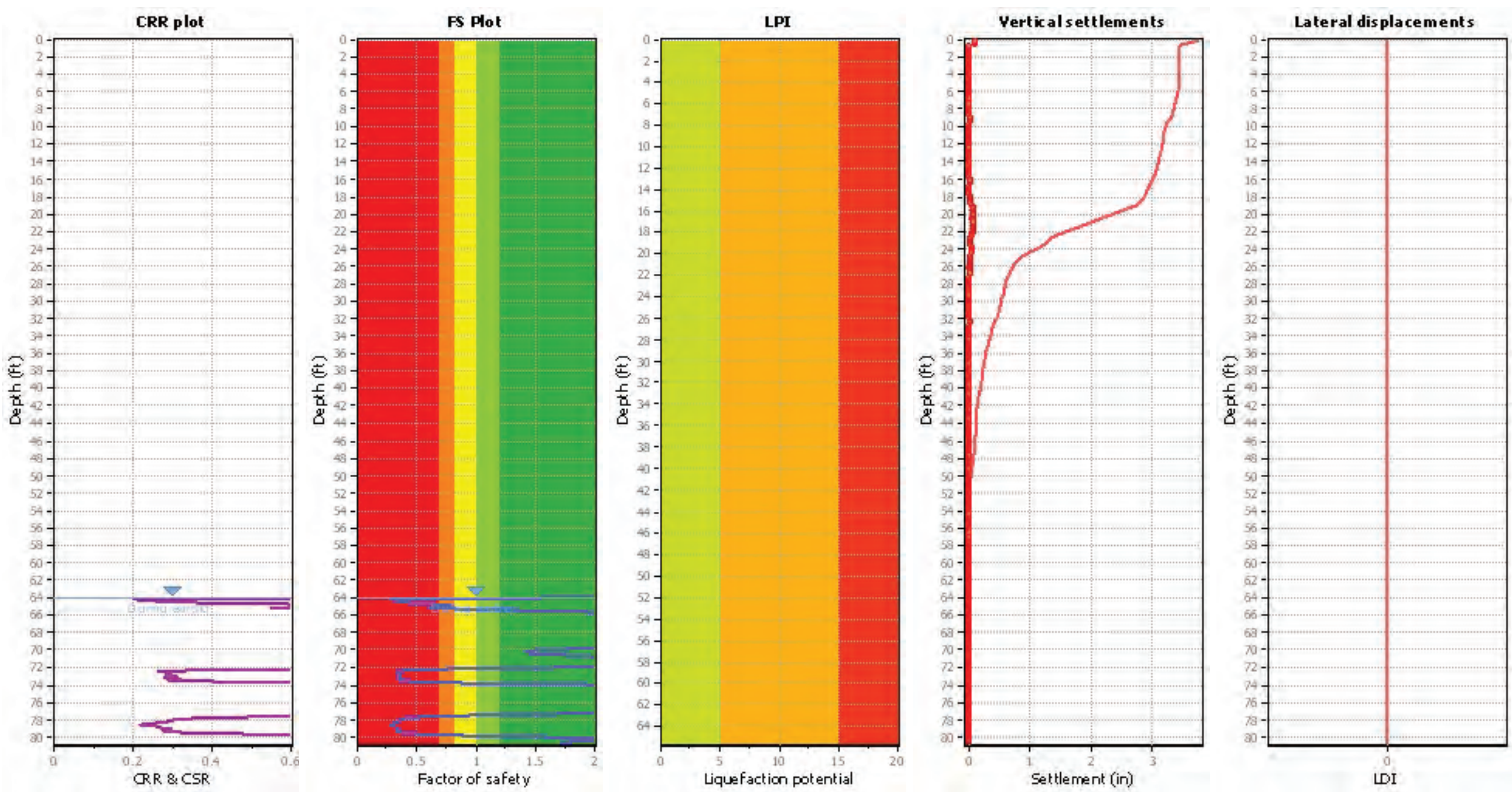
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	64.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



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Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.02	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	64.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

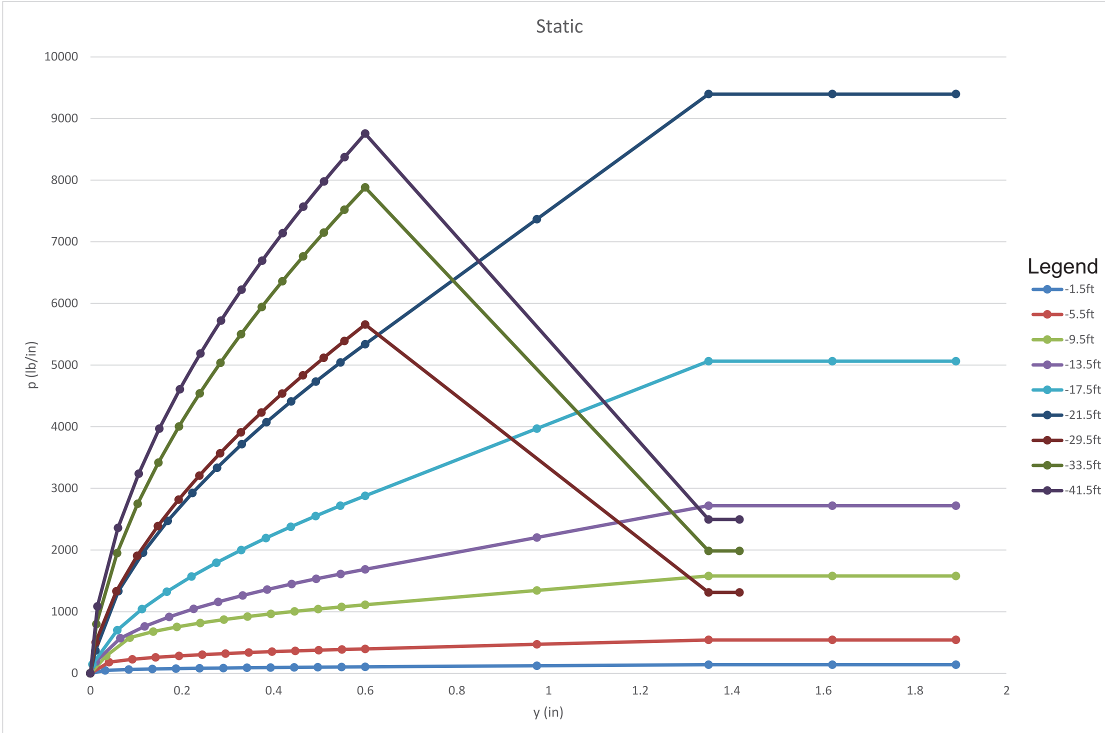
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

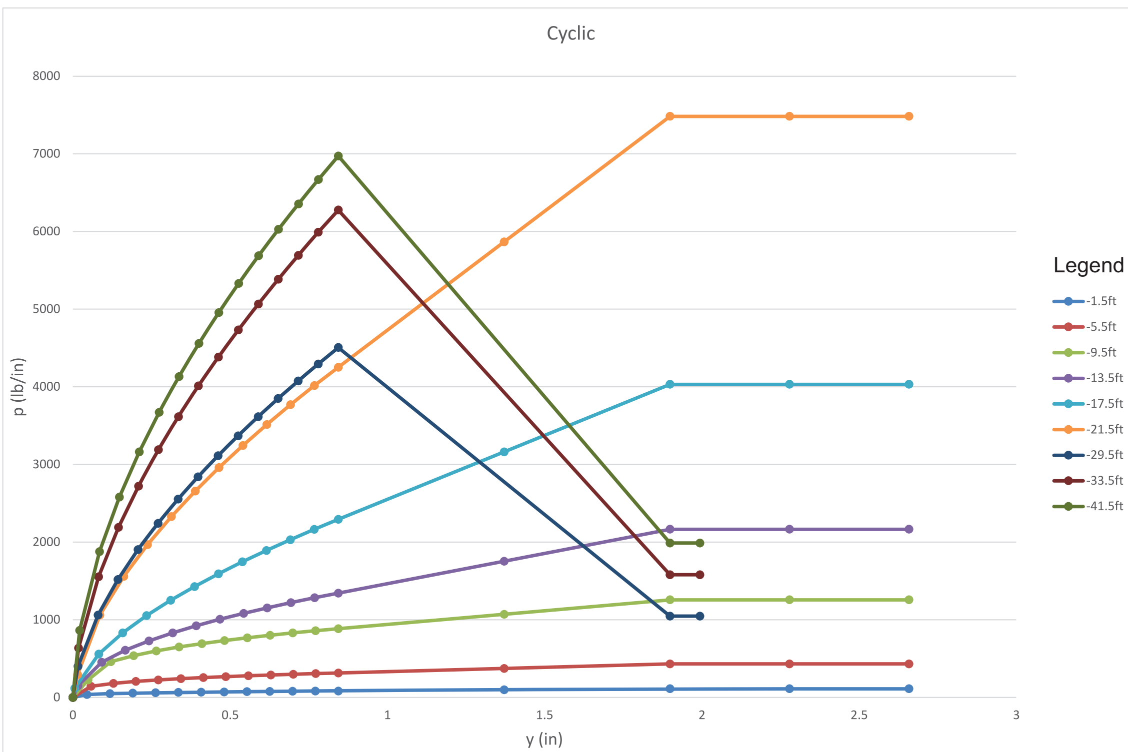
LPI color scheme

- Very high risk
- High risk
- Low risk

APPENDIX B

SOIL SPRING DATA



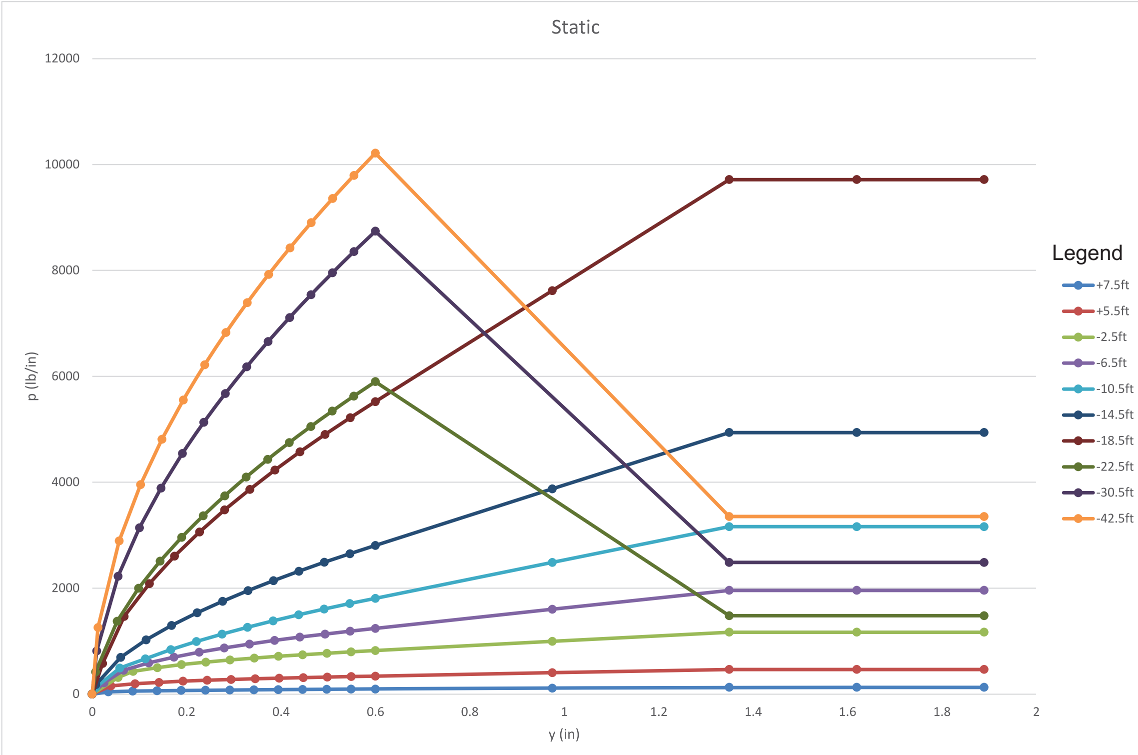


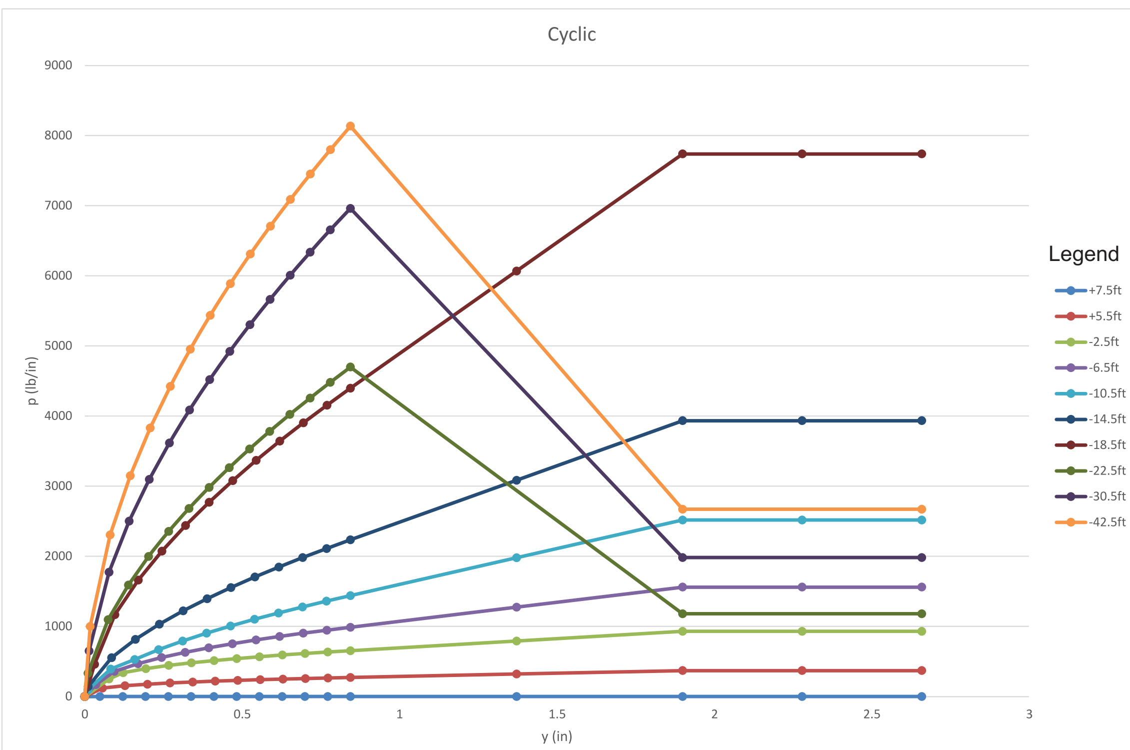
P-Y CURVES AT NORTH REACH STA. 13+55 (72 YEARS)

SOUTH OCEAN BEACH COASTAL EROSION AND
WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: JULY 2021	PLATE C-1



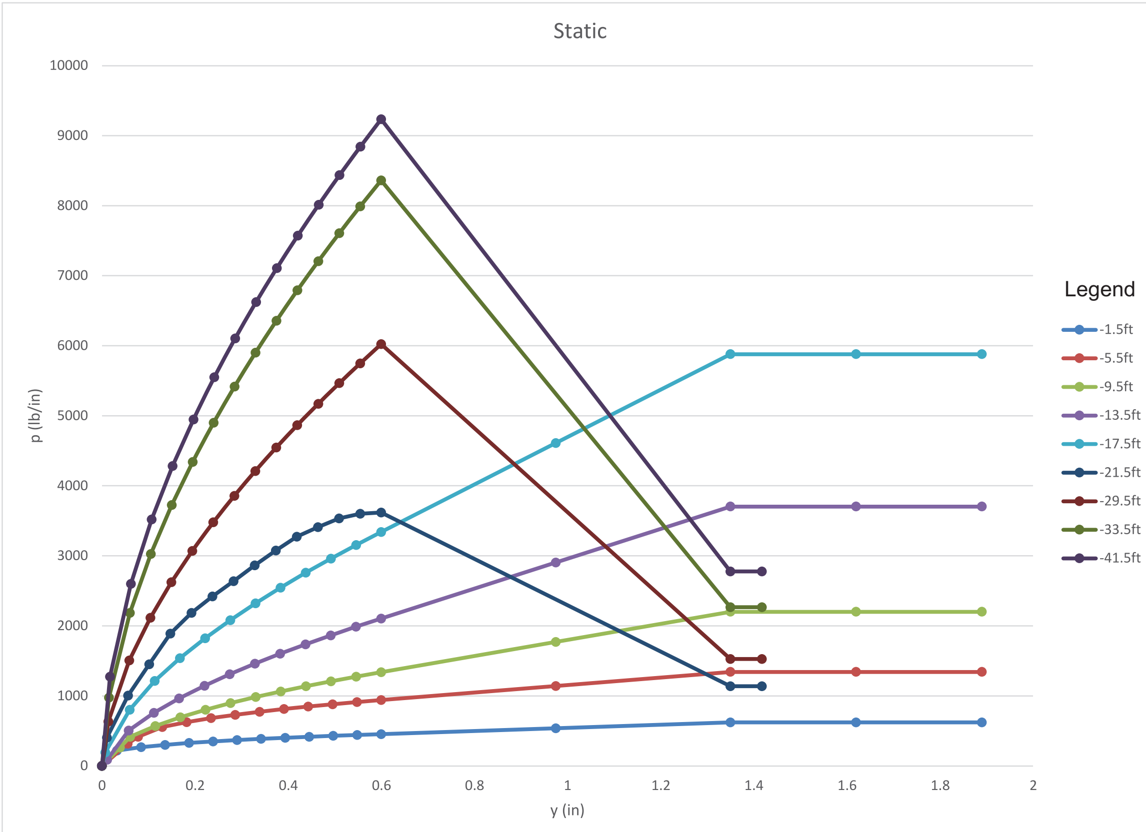


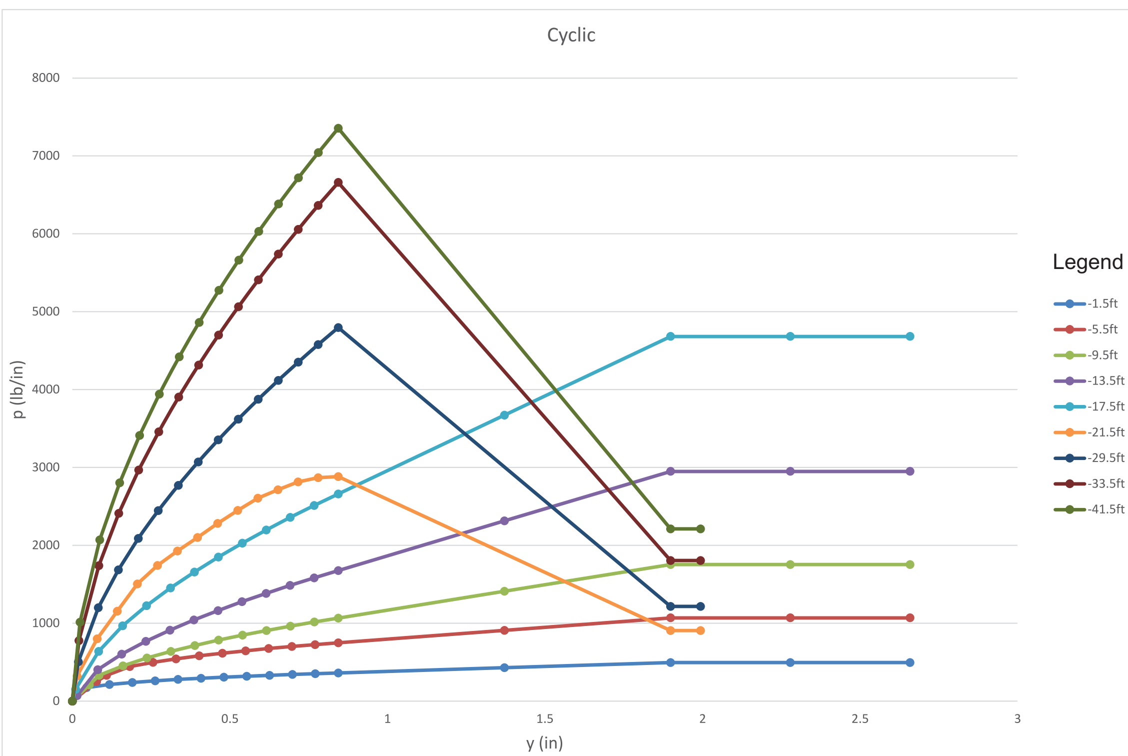
P-Y CURVES AT NORTH REACH STA. 13+55 (975 YEARS)

SOUTH OCEAN BEACH COASTAL EROSION AND
WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: JULY 2021	PLATE C-2



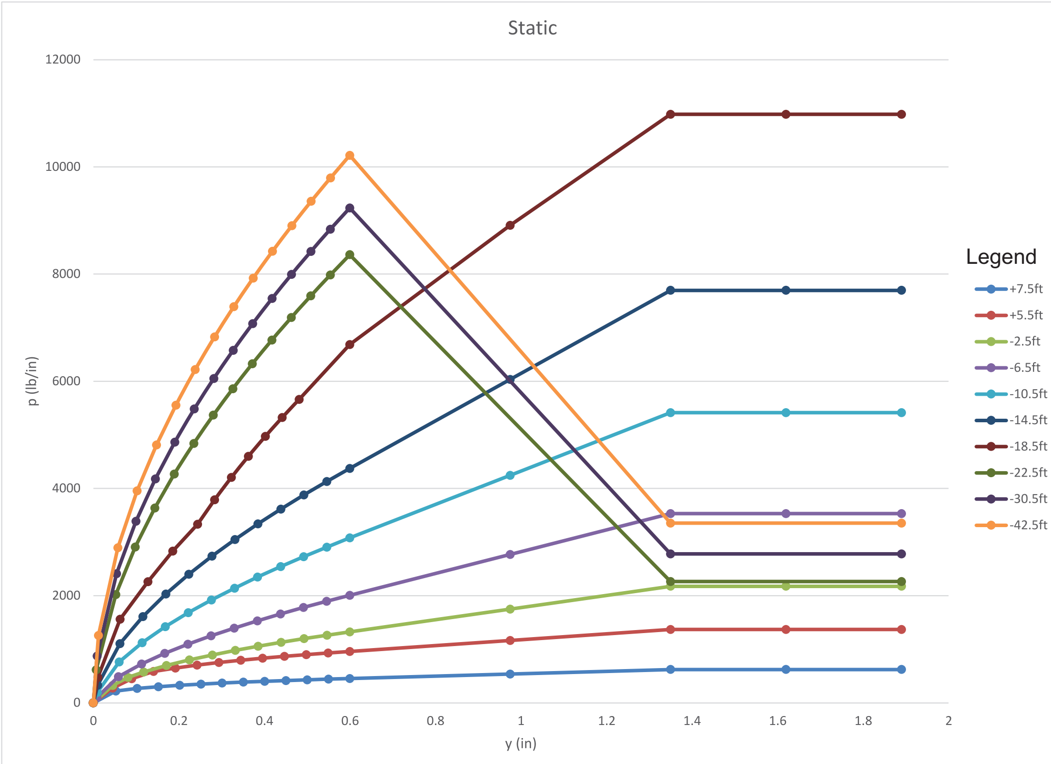


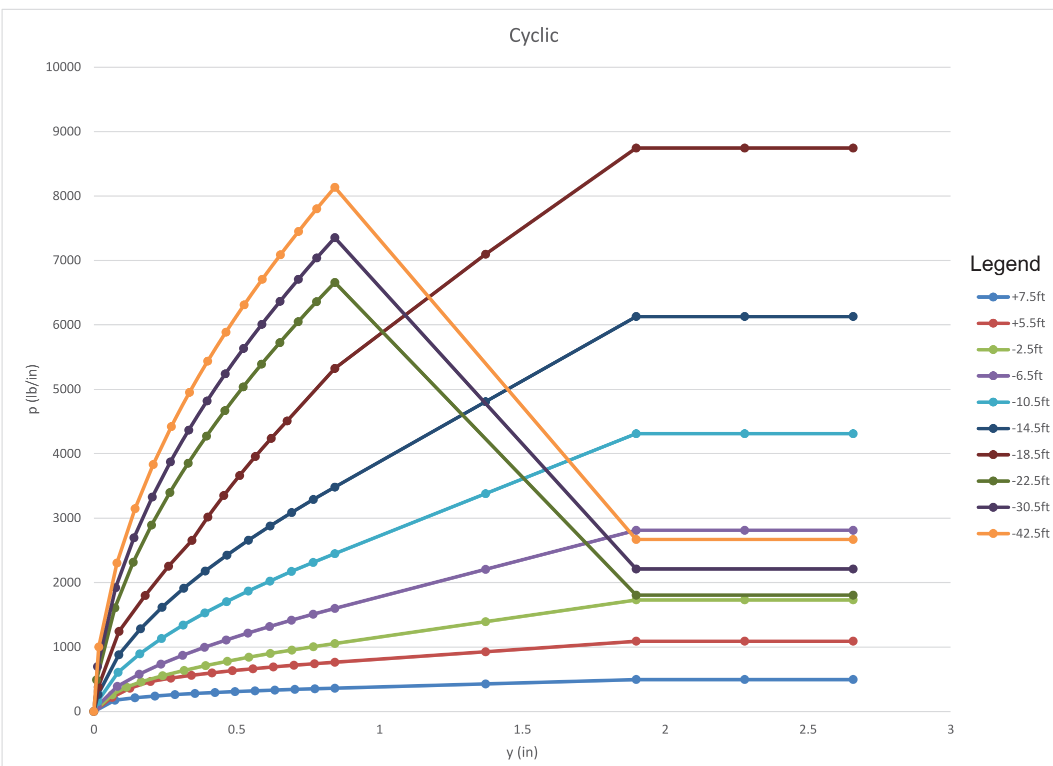
P-Y CURVES AT EQR REACH STA. 22+30 (72 YEARS)

SOUTH OCEAN BEACH COASTAL EROSION AND
WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: JULY 2021	PLATE C-3



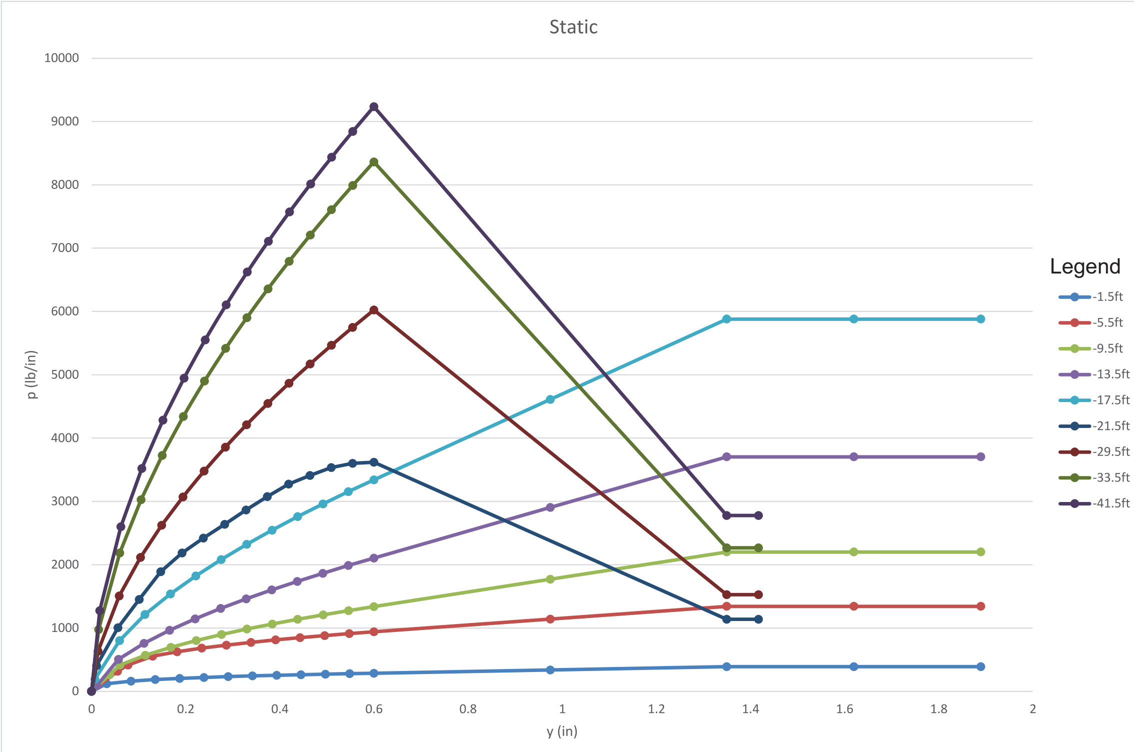


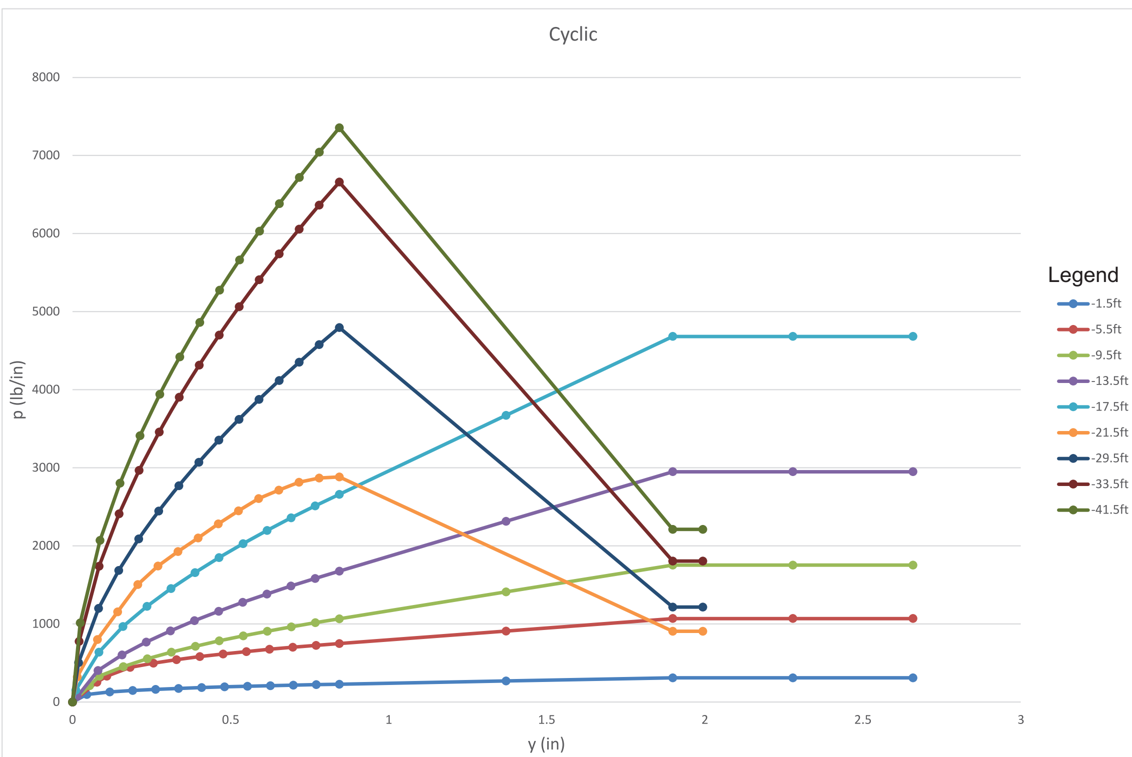
P-Y CURVES AT EQR REACH STA. 22+30 (975 YEARS)

SOUTH OCEAN BEACH COASTAL EROSION AND
WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: JULY 2021	PLATE C-4



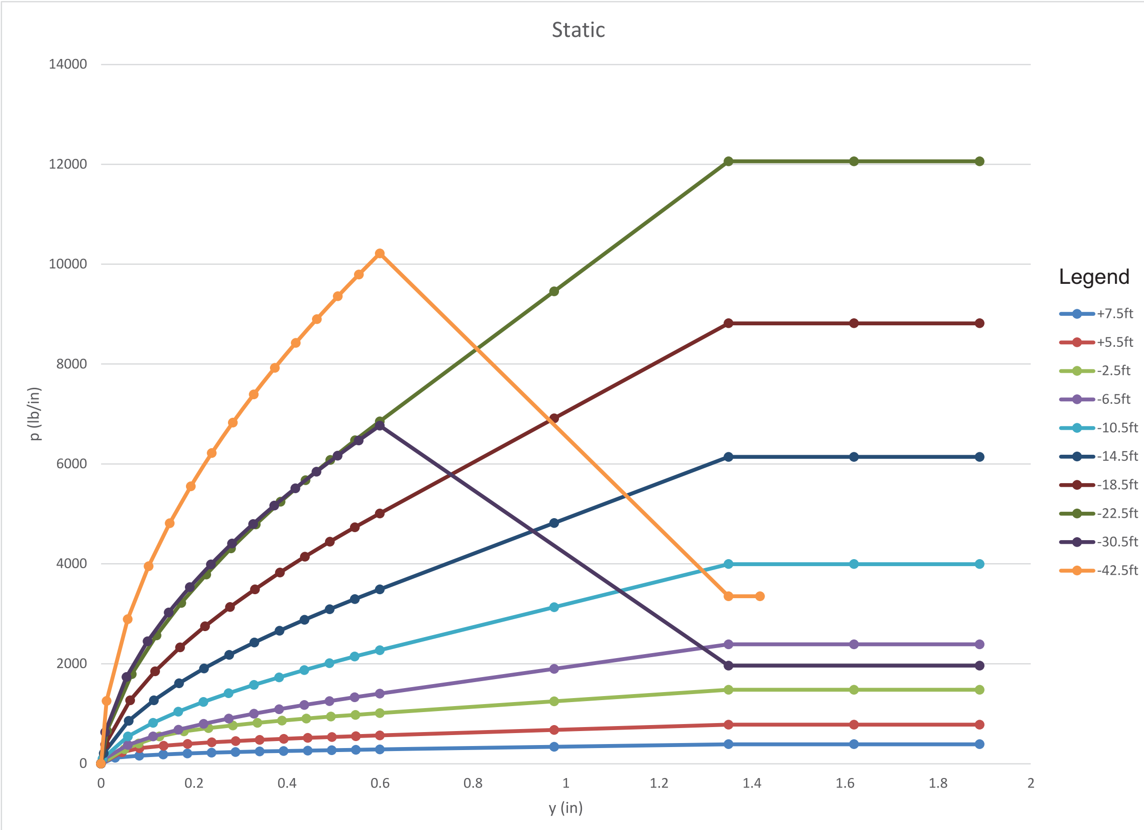


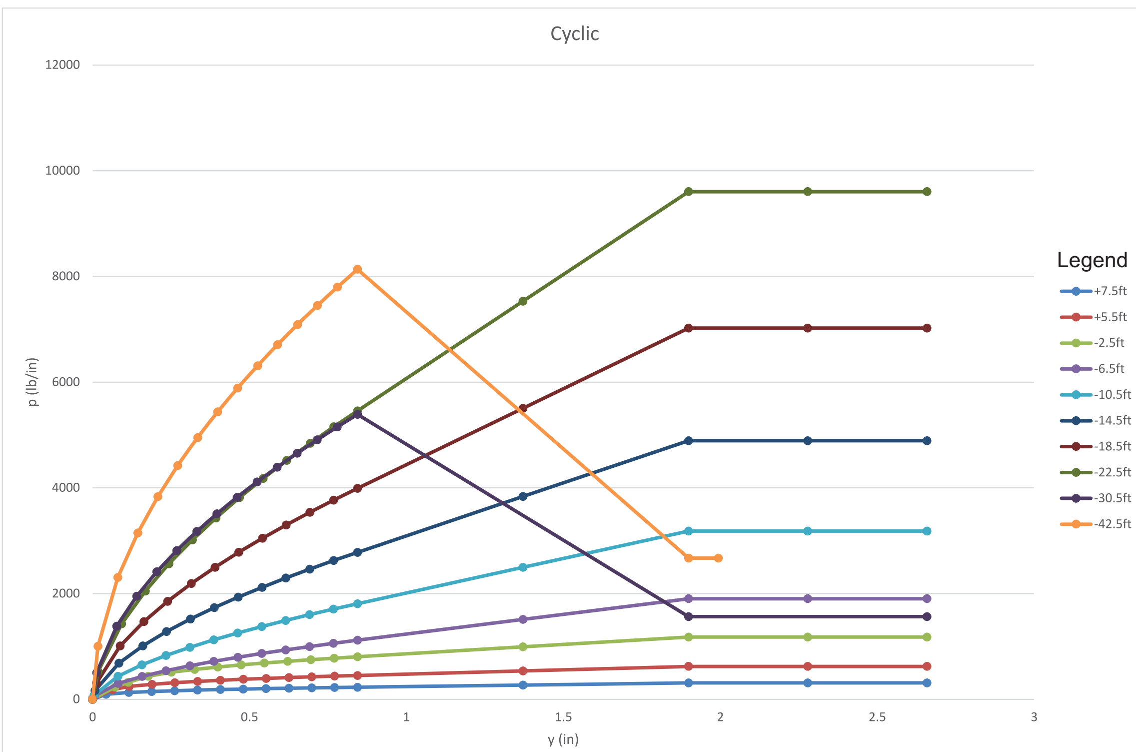
P-Y CURVES AT RUBBLE REACH STA. 27+40 (72 YEARS)

SOUTH OCEAN BEACH COASTAL EROSION AND
WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: JULY 2021	PLATE C-5





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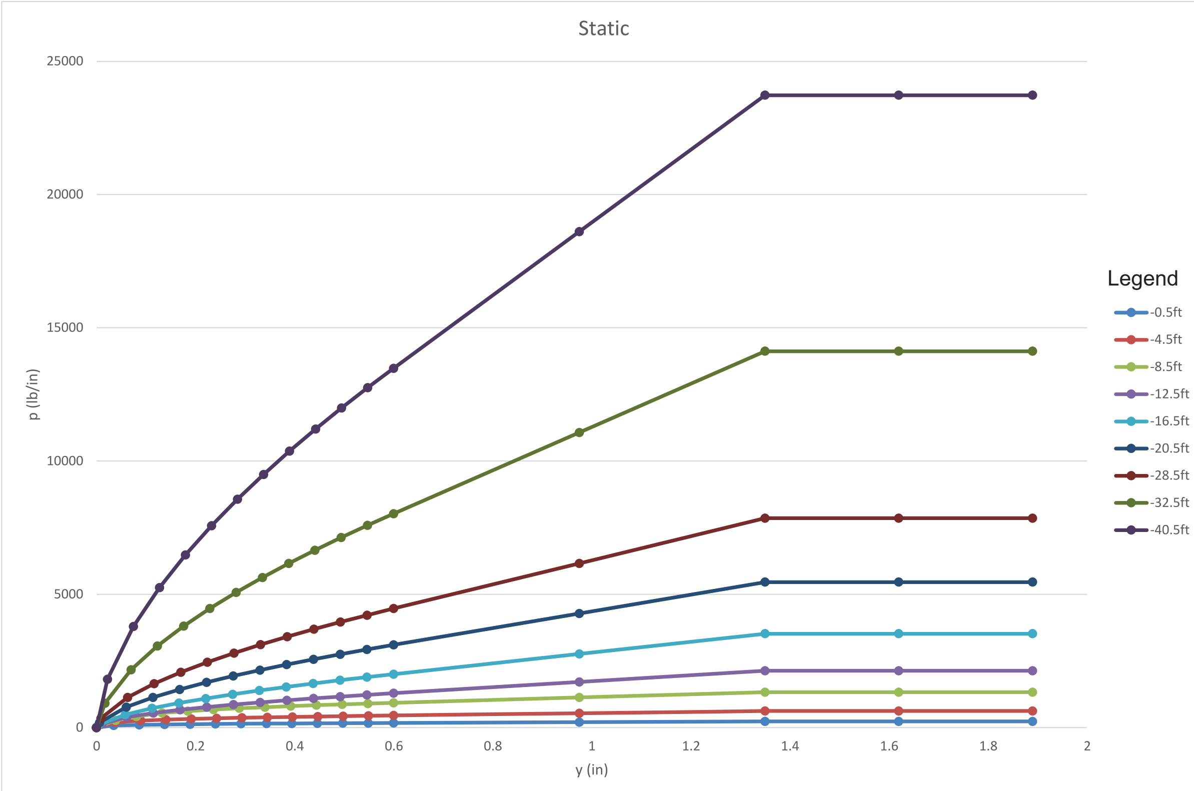
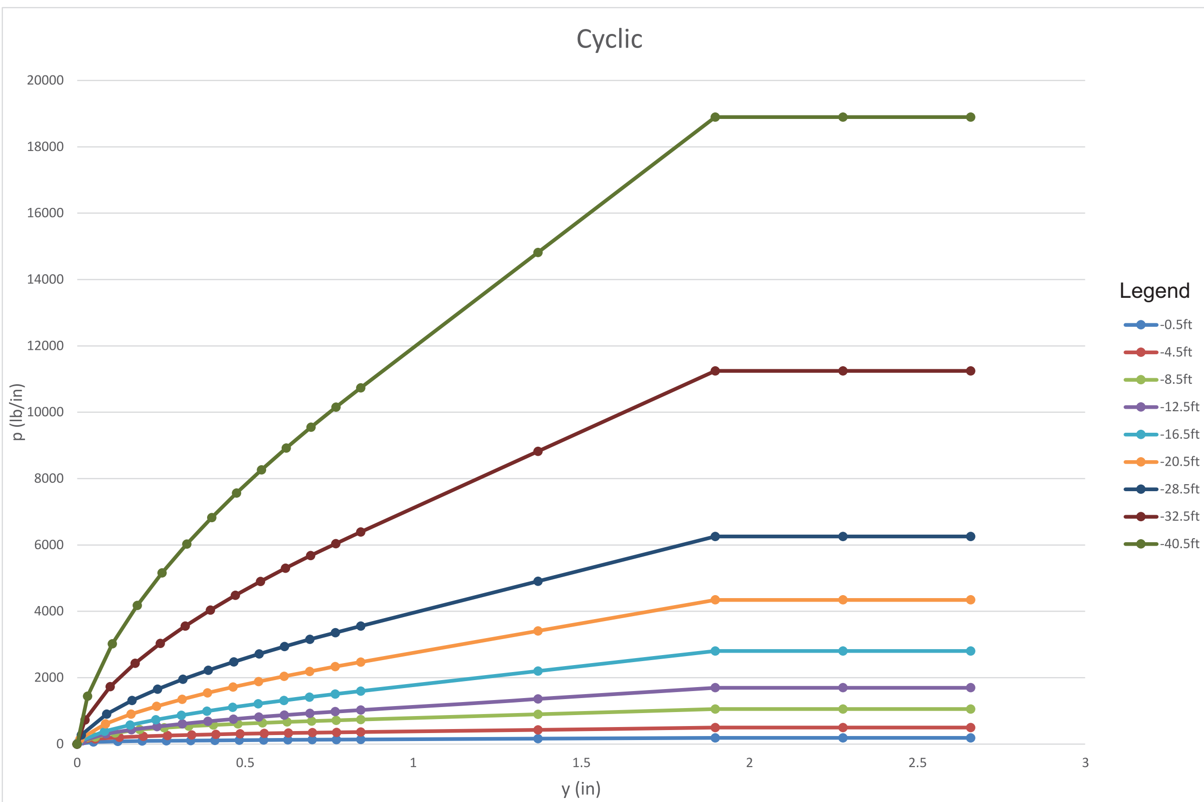
JOB NO. AGS-18-003	DATE: JULY 2021	PLATE C-6



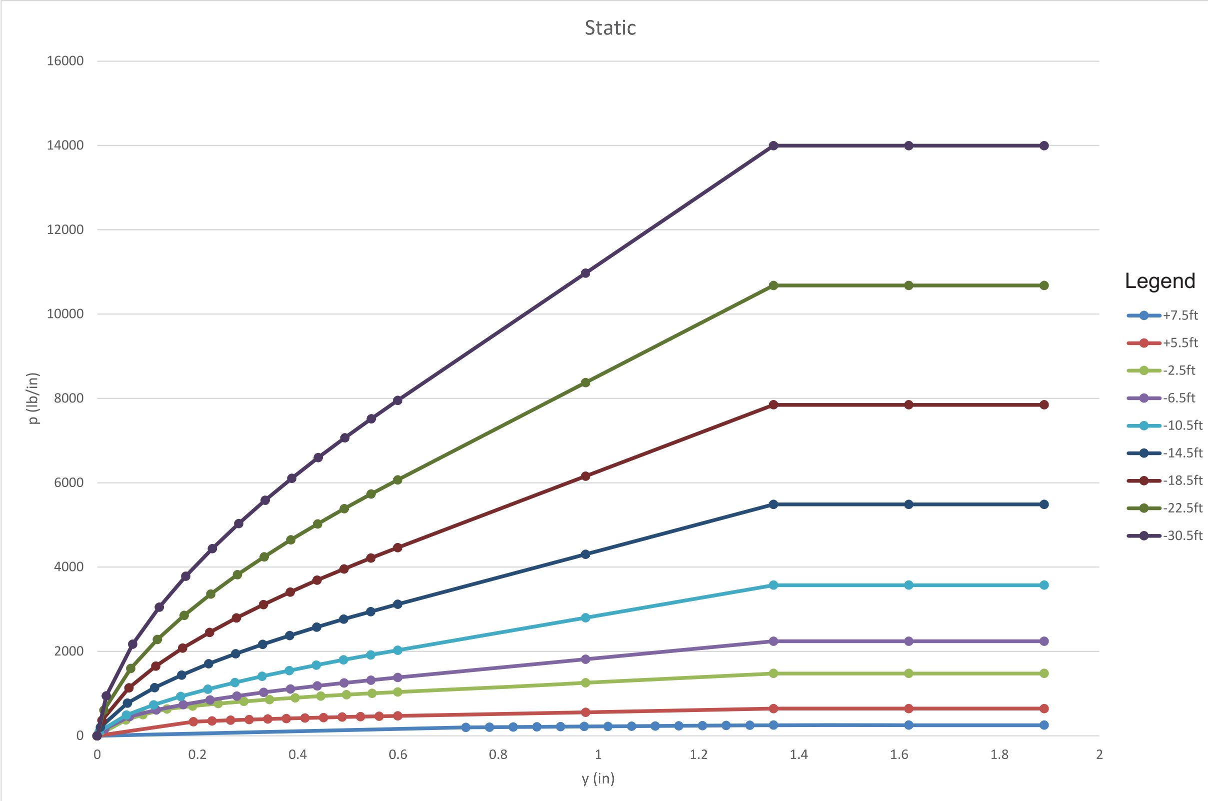


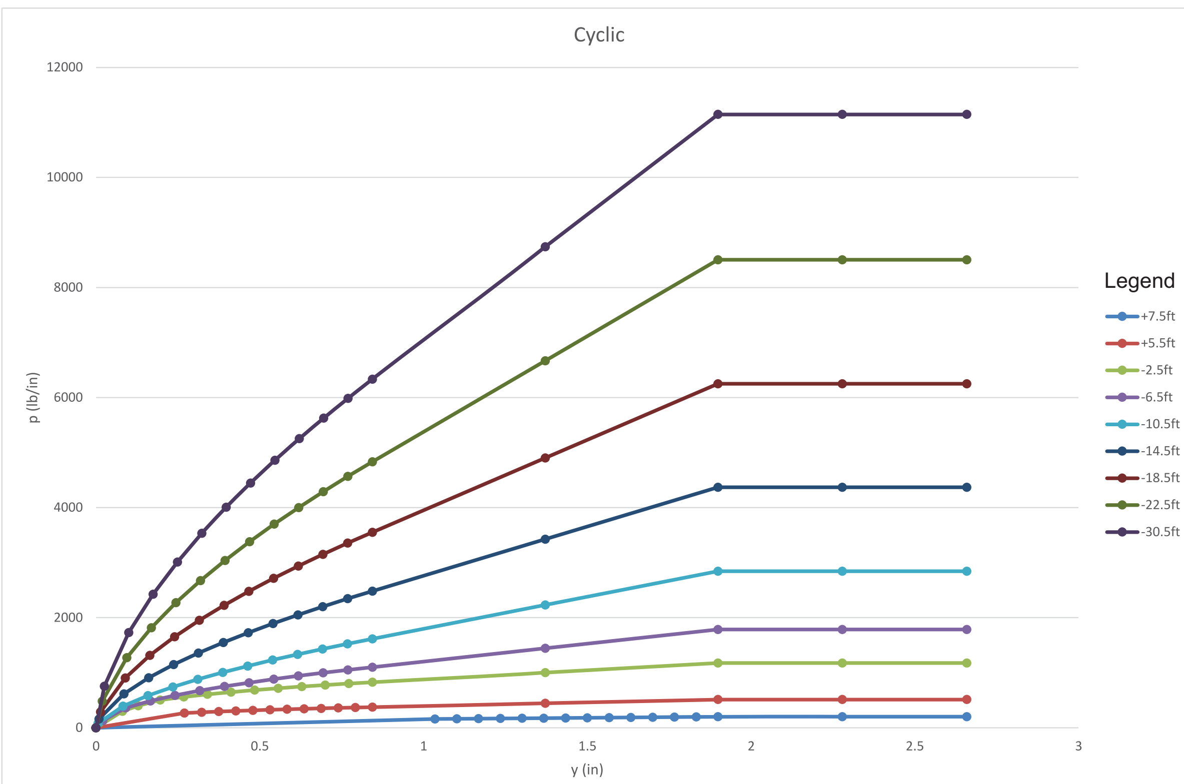
P-Y CURVES AT BLUFF REACH STA. 35+05 (72 YEARS)

SOUTH OCEAN BEACH COASTAL EROSION AND
WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: JULY 2021	PLATE C-7



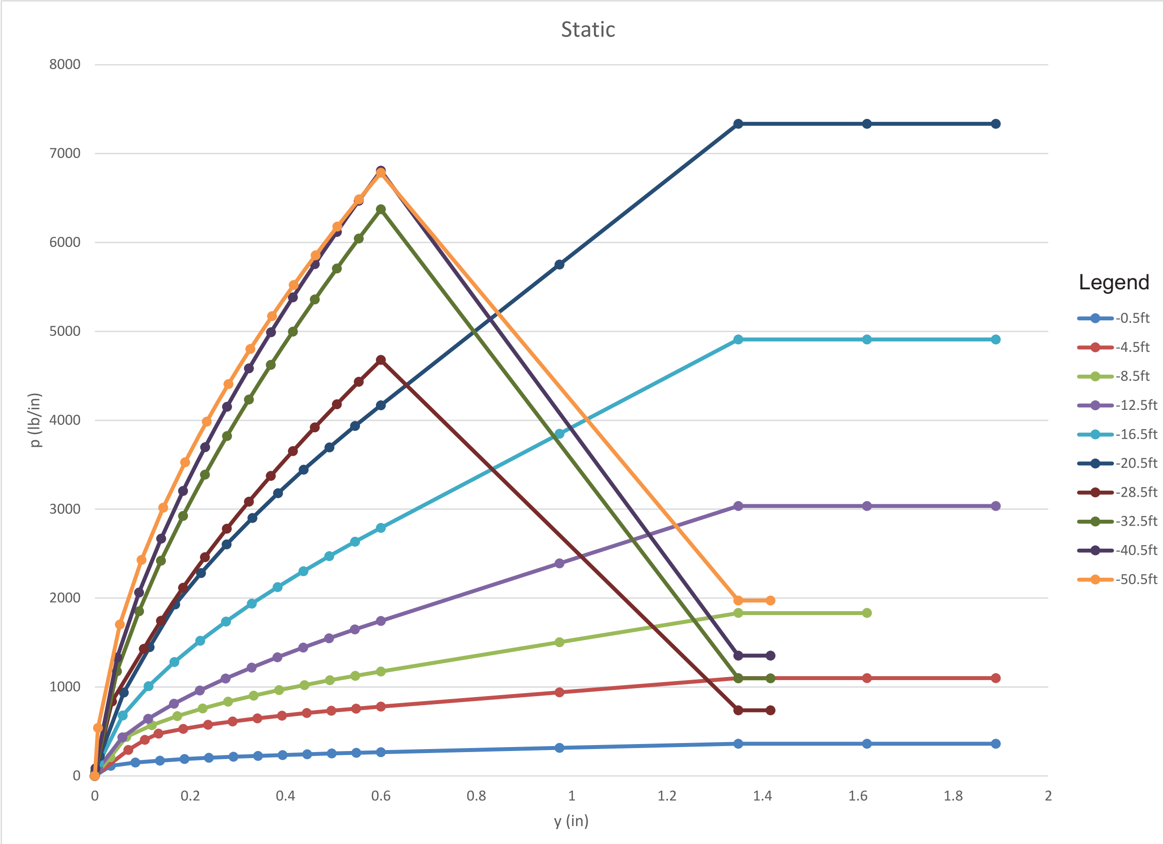


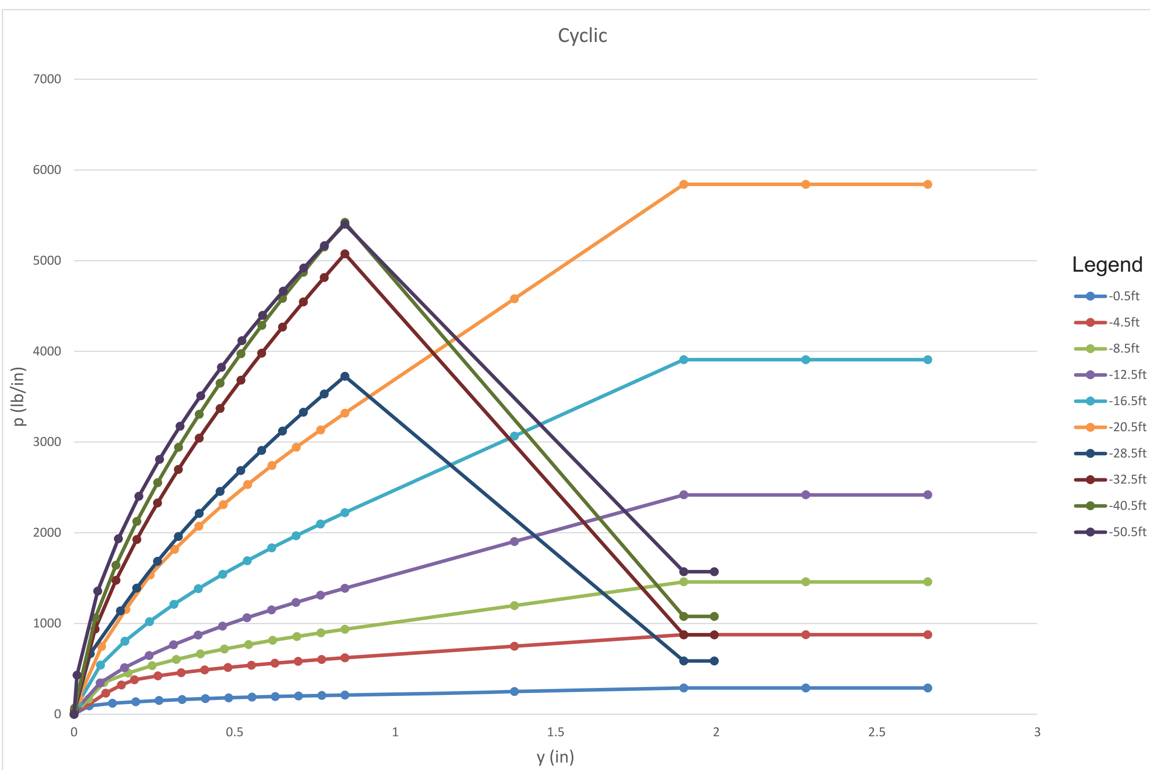
P-Y CURVES AT BLUFF REACH STA. 35+05 (975 YEARS)

SOUTH OCEAN BEACH COASTAL EROSION AND
WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: JULY 2021	PLATE C-8





P-Y CURVES AT SOUTH REACH STA. 41+90 (72 YEARS) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA	AGS
JOB NO. AGS-18-003 · DATE: JULY 2021	PLATE C-9

Exhibit 8
2-21-0912
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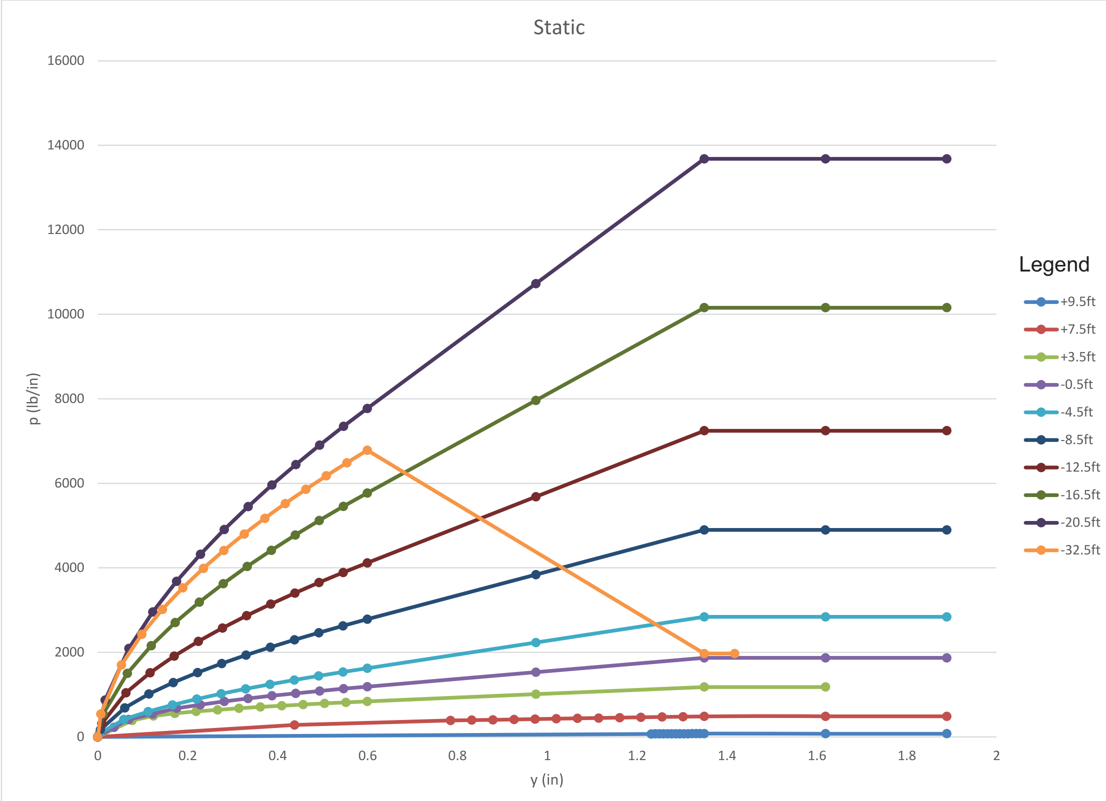

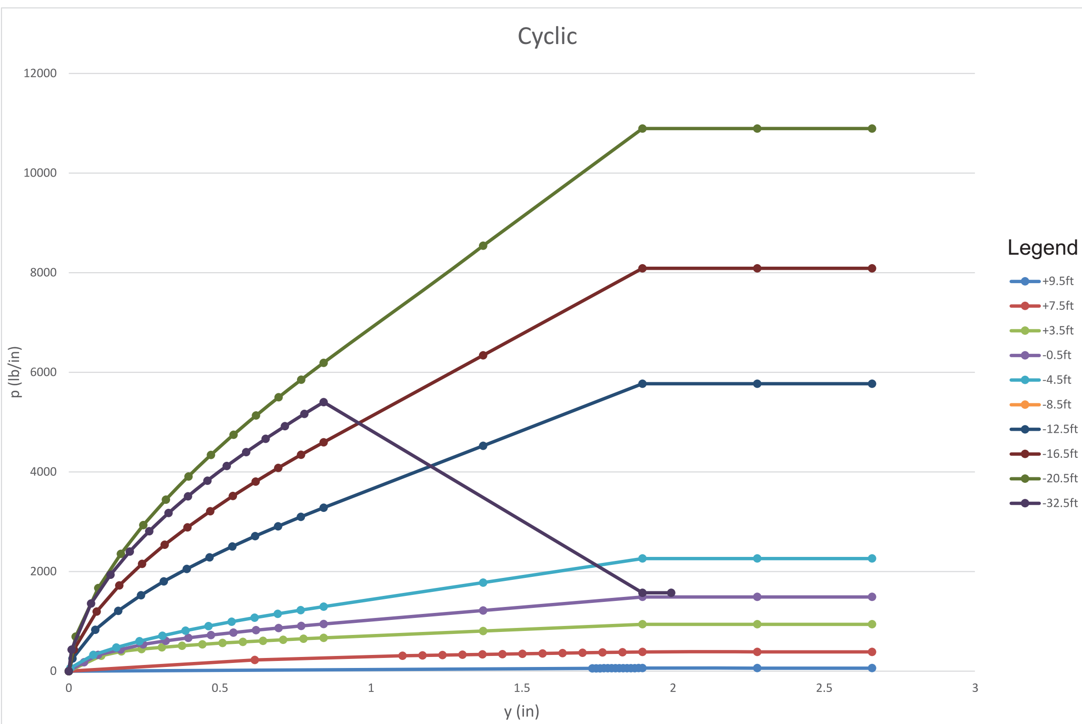

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JOB NO. AGS-18-003	DATE: JULY 2021
PLATE C-10	

APPENDIX C

FLAC3D NUMERICAL ANALYSES TECHNICAL MEMORANDUM

Technical Memorandum

South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection, San Francisco, California

Task 3 – FLAC3D Numerical Analyses

Prepared for

San Francisco

Public Utilities Commission

June 2021

Prepared by

AGS

AGS Project No. AGS-18-003

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Appendix E - **FLAC3D NUMERICAL SSSI DYNAMIC ANALYSES RESULTS FOR BLUFF REACH**

Appendix F - **FLAC3D NUMERICAL SSSI DYNAMIC ANALYSES RESULTS FOR SOUTH REACH**

1. Introduction

1.1 Background and Purpose

This technical memorandum presents the results of a Seismic Soil Structure Interaction (SSSI) study performed by AGS Inc. (AGS) for the South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection (CEWIP) Project in San Francisco, California. The project alignment is located along the Great Highway, from Sloat Boulevard to approximately 3,200 feet south of Sloat Boulevard as shown on Plate A-1.

According to the Ocean Beach Master Plan (SPUR, 2012), sea level rise and climate instability are causing an increasing rate of shoreline retreat and coastal bluff erosion in the vicinity of South Ocean Beach. The purpose of the South Ocean Beach CEWIP Project is to protect vital infrastructure that may be threatened by the advancing coastal retreat, such as the Lake Merced Transport (LMT). The LMT is a 14-foot inside diameter, 16-foot outside diameter sewer and storm water tunnel that was constructed in 1992 and extends subparallel to the shoreline, underneath the southbound lane(s) of the Great Highway. The LMT begins at the Westside Pump Station, continues 2.6 kilometers south, and terminates at the Lake Merced Pump Station at the intersection of John Muir Drive and Skyline Boulevard.

The Southwest Ocean Outfall (SWOO) is a 23,400 (4.4 mile) feet outfall structure and pipe that crosses underneath the LMT in the vicinity of the Oceanside Water Pollution Control Treatment Plant (OSP) and extends westward carrying sanitary and storm water flows into the ocean. The onshore SWOO structure is a 12 by 12 feet concrete box at the beginning of the tunnel and becomes a 12-foot inside diameter concrete pipe extending into the ocean from the headwall.

The proposed South Ocean Beach CEWIP Project includes the following two elements:

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

The long-term LMT Protection Feasibility Study Report which was part of the Ocean Beach Master Plan: Coastal Management Framework (ESA, 2015) outlined a concept of protecting the tunnel with a low-profile retaining wall, such as a secant pile wall with tie-backs. Initially the secant pile wall will be buried underground but as the erosion takes place, the wall will be exposed with a backslope of 3H:1V. The purpose of this study is to evaluate the response of the proposed secant pile wall, tie-backs and the existing LMT under the design earthquake shaking.

AGS developed 3-dimensional (3-D) finite difference models (FDM) for purposes of seismic analyses of the project alignment. SSSI is incorporated in the analyses. Response of LMT, secant pile wall and supporting structures (including tie-backs and pile caps) are analyzed for three different orthogonal site-specific Earthquake Scenarios developed for the site location.

The geotechnical data used to create the numerical analyses presented in this technical memorandum are based on characterization of the subsurface conditions at the location of the project alignment using the geotechnical investigation performed by AGS and presented in the Geotechnical Data Report (GDR) (AGS, 2020) and Geotechnical Interpretative Report (GIR) (AGS, 2019), as well as the available data which were obtained from several geological and geotechnical sources including the database of the United States Geological survey (USGS), California Geological Survey (CGS), previous geotechnical reports prepared by AGS and additional reports prepared by other consultants. Structural data were obtained from the proposed and as-built structural plans. Plate A-2 presents the site plan of the project and boring logs used to establish the subsurface stratigraphy. Details of the geology of the site and site conditions are discussed in Section 2 of this Technical Memorandum.

1.2 Proposed Project

The low-profile retaining wall will be constructed below grade on the seaward side of LMT. The offset of the low-profile retaining wall from the seaward edge of the tunnel will be a minimum of 16 feet, except for the northern part adjacent to the existing Sloat beach access parking lot, where the offset will be increased to about 42 feet.

The secant pile retaining wall will consist of overlapping unreinforced and reinforced drilled, cast-in-place concrete piles (called “primary unreinforced” and “secondary reinforced” piles, respectively) installed at approximately 5-foot spacing between reinforced piles. Both the primary unreinforced and secondary reinforced piles will be approximately 3 feet in diameter. The primary unreinforced piles will be drilled first and filled with concrete, followed by the secondary reinforced piles drilled between and partially cutting into the primary piles. The toe of the primary unreinforced piles will be set at approximately Elevation -10 feet (NAVD88[1]). The secondary reinforced piles will be extended to a minimum depth of 60 feet as determined by structural analysis. A 4'X4' square continuous pile cap will be constructed for the secant pile retaining wall with the top at approximately 6 feet above the crown of the LMT. As currently planned, tie-backs spaced every 10 feet will be installed at inclination of 1½ horizontal to 2 vertical (1½H:2V) downward from the pile cap to provide additional lateral support.

Initially, the secant pile retaining wall will be concealed. However, over time, as beach recession occurs, the secant pile retaining wall will be exposed (with the seaward side lowered to minimum Elevation +2 feet in front of the wall for 72-year event and Elevation +10 feet for 975-year return period). Our numerical analyses were performed for the worst case Scenario of Elevation +2 feet. Ultimately, the landward side of the secant pile retaining wall will become a 3 horizontal to 1 vertical (3H:1V) backslope. For erosion protection of the ultimate 3H:1V backslope against extreme wave runup in the future, an approximately 4 feet thick layer of either controlled low strength material (CLSM) or soil-cement (soil-crete) mix will be constructed as a cover for the ultimate 3H:1V.

1.3 Scope of Work

The primary expected results from coupled SSSI analyses are the stresses and displacements of the structure and the soil. In this project, SSSI problems relate to two-way interactive coupled systems, where the state of structural deformations of secant pile wall and stresses depend on the earth pressures and movements, while the earth pressures depend on the loading and deformations of the structure. FLAC3D (Itasca Consulting Group Inc, 2019) which is an explicit finite difference program was used to numerically simulate the large strain condition induced during and post liquefaction event. The results of dynamic SSSI analysis were used to estimate the deformation of the proposed secant pile wall and supporting piles.

1.4 Report Organization

This technical memorandum consists of six Sections, including this introductory Section, and 6 appendices as follows:

---

[1] Elevations in this study are expressed as North American Vertical Datum of 1988 (NAVD88), unless otherwise noted.

- Section 1 includes the summary of the background and purpose, proposed project, and scope of the work.
- Section 2 describes regional and local geology, faults and seismicity based on the available geotechnical databases as discussed and published geologic reports.
- Section 3 presents the site and subsurface conditions based on previous field, and laboratory investigations.
- Section 4 presents the seismic design ground motions.
- Section 5 discusses the Results of FLAC3D soil structure interaction analyses.
- Section 6 presents conclusions and recommendations.
- The appendices present the supplemental information and supporting documents.

2. Geology

2.1 Overview

In our GIR (AGS, 2020) and GDR (AGS, 2019), AGS identified six characteristic geologic units underlying the site which include: Artificial Fill, Dune Sand, Beach Sand, Colma Formation, Merced Formation and Franciscan Bedrock. Plates A-3 and A-4 present the generalized geological cross Section along the tunnel. Engineered backfill will be placed around the proposed pipelines.

2.2 Project Area Geology

Details of the geology of the project site are presented in detail in the GIR (AGS, 2020). AGS performed a field exploration program consisting of drilling seven (7) soil borings to depths of up to 101.5 feet below ground surface. In addition, three (3) monitoring wells, fourteen (14) CPT soundings, twelve (12) vacuum potholes and three (3) test pits were also performed as part of the exploration program. The geological units that were encountered in the explorations included Artificial Fill, Dune Sand, Beach Sand, Colma Formation and Franciscan Complex. Locations of the boreholes in the vicinity of the project site are shown on Plate A-2.

Details of the underlying materials of the site are explained in the GDR and GIR (AGS, 2020) and (AGS, 2019). However, in the following paragraphs we briefly summarize geologic conditions and geological units.

The site was previously reclaimed through placement of artificial fills in the early 1900's. Geological units of the geologic units underlying the project site and its vicinity are briefly described below:

- Artificial Fill**:** Near Sloat Boulevard, Ocean Beach is separated from the Great Highway by a rubble wall about 100 feet wide and 20 to 23 feet above high water. This rubble wall was built in the late 1800s to early 1900s in order to provide a surface for the construction of the Great Highway. The rubble consists of angular fragments of red chert, sandstone, and greenstone of the Franciscan Formation, in a mixture of sand and clay. Artificial fill that resulted from grading operations during development of the project alignment is derived from native sediments, making it difficult to distinguish from dune sands and weathered unconsolidated Colma Formation sands.

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

- Dune Sand: In the project vicinity, Holocene dune sand deposits (Qd) extend from the western edge of Lake Merced to the coast. These deposits were fed by sand blown east from Ocean Beach and were deposited in the post-glacial period within the last 10,000 years. The thickness of the dune sand ranges from light cover at the tops of the highest bluffs, to up to 50 feet inland of the coast in the project. Near-surface dune sands tend to be poorly graded, fine to medium grained clean sand, whereas sands at depth may have light cementation or laminations.
- Beach Sand: Beach sand in the project vicinity is comprised of loose, well-sorted quartz and feldspar sand, which grades fine to coarse depending on its location in the surf zone. The beach sand at Ocean Beach has heavy mineral laminations, as well as thick layers of sand comprised of magnetite at the beach surface at the toe of the bluffs. These sands primarily originated from the Sierra Nevada during the previous low-stand sea level, when the San Francisco Bay was above water. Some of the sands are also comprised of continental (Franciscan) sediment outwash.
- Colma Formation: The Colma Formation overlies the tilted beds of the Merced Formation at an angular unconformity. Inland units of the Colma Formation include poorly consolidated colluvial, stream and eolian deposits, whereas coastal deposits of the Colma are more likely to be marine (nearshore and backshore) and estuarine in nature. Like the Merced Formation, the facies of the Colma Formation were formed with similar sedimentary material as well as in a similar depositional environment created by the rising and falling sea level causing transgression and regression of the shoreline.

  Yi (2005), McGuire (2009) and others have described the Colma as an approximately 3-foot thick “thin erosional remnant” at Ocean Beach in the project vicinity, and up to about 40 feet thick progressing south to Thornton Beach. However, Woodward-Clyde (1977) indicated that the Colma is up to 200 feet thick in the vicinity of the SWOO and overlies the Merced, and Bonilla (1988) maps the Colma as underlying the northern two-thirds of the site.

The Colma Formation generally consists of oxidized, reddish brown, predominantly medium-grained quartz-feldspar arkosic sand with heavy mineral laminations, and bedding ranging from horizontal up to dipping 5 degrees East. Facies of the Colma Formation at depth may include fine-grained micaceous silty sand, silt, thin clay lenses, and lenses of rounded fine gravels consisting of red chert, green chert, Monterey formation laminated rock, and blue schist. According to Bonilla (1998) and Kennedy (2002), the Colma Formation is of latest Pleistocene age and was deposited between about 70,000 to 130,000 years ago.

- <u>Merced Formation</u>: The Merced Formation overlies the Franciscan Complex in the project vicinity, and consists of an accumulation of poorly consolidated sand, clay, gravel and silt sediments, which were deposited almost continuously in the late to early Pleistocene. Clifton and Hunter (1988) mapped a sequence of approximately 40 facies in the variably tilted and uplifted Merced exposures in the seaside cliffs, from the southern edge of the project alignment near Boring B-6 and continuing south approximately 7 kilometers to Mussel Rock. These sequences consist of marginal marine sediments, such as shelf, nearshore, backshore, embayment and fluvial facies, and their arrangement is indicative of alternate transgression and regression of the sea during geologic time. Yi (2009) mapped exposures of the Merced Formation in the project vicinity, from Sloat Boulevard to Thornton Beach, which is approximately 4.5 kilometers south of Sloat.

  According to Hall (1965), Clifton (1988), Yi (2009), Kennedy (2002) and McGuire (2005), the Sequence X and Y of the Merced Formation are exposed closest to the project vicinity. These facies are generally weakly lithified to well-cemented, thinly bedded silts, sands, clays and pebbly shell hash deposited in a shallow marine environment. Merced Formation at depth in the project vicinity is assumed to be characterized by light gray to dark gray and black fine-grained sand high in heavy minerals such as magnetite, and dark bluish gray fat clays with silty interbeds. Micaceous material indicative of backshore deposits is also possible in the Merced.

  Based on the tectonic history of the Serra Fault, the Merced Formation can show bedding ranging from near-horizontal in the project vicinity, to up to 25 degrees and striking northeast in the vicinity of Fort Funston and Mussel Rock. Based on Woodward-Clyde's Boring B-1, the Merced Formation in the project vicinity can extend to approximately 400 feet depth at the contact with the Franciscan Formation.

Franciscan Complex: Franciscan Complex rocks underlying the project alignment and its vicinity include graywacke sandstone, siltstone, claystone and shale.

The representative soil layers are summarized in Table 1 below. The soil profile information presented in Table 1 was used for analysis of the 35 percent design.

TABLE 1 - SOIL PROFILES AT REPRESENTATIVE STATIONS

Reach Designations	North Reach	EQR Reach	Rubble Reach	Bluff Reach	South Reach
Stations	13+55	22+30	27+40	35+05	41+90
Boring/CPT Designation	B-1	CPT-5	B-3	R2-1	WC-10
Elevation NAVD88 - ft	31	30	28.5	37	56
Top of the wall EL NAVD88 - ft	14.7	15.90	16.6	17.6	21.0
Bottom of Pile Cap EL NAVD88 ft	10.7	11.90	12.6	13.6	17.0
Fill Layer Thickness-ft	20	13	13	10	28
Dune Layer Thickness-ft	10	11	4	10	10
Fill + Dune Thickness-ft	30	24	17	20	38
Colma Layer Thickness-ft	20	25	27	45	22
Merced Layer Thickness-ft	>20	>20	>20	>20	>20

2.3 Faults and Seismicity

The site is not located within an Alquist-Priolo earthquake fault zone (CGS, 2007). Details of the Historical Seismicity of the site are explained in detail in the GDR and GIR (AGS, 2020) and (AGS, 2019). The project area is in a seismically active region subject to periodic earthquakes causing strong to violent ground shaking of the site. The San Andreas Fault is the major fault system in the region. The Maximum magnitude earthquake on the San Andreas Fault will be a magnitude 8.05 event occurring approximately 2.6 km (1.6 miles) southwest from the project site.

3. Subsurface Conditions

This Section presents available subsurface exploratory information in the vicinity of the site project. Subsurface exploratory data from past studies by AGS and others was combined to develop subsurface profiles, assess the constraints of any potential liquefaction, groundwater conditions, and any potential ground instabilities. Locations of subsurface data collected by AGS in the vicinity of the study area are shown on Plate A-2.

3.1 Subsurface Conditions

3.1.1 Soil Stratigraphy

Based on material encountered in our borings, CPTs, and potholes, as well as the results of the geotechnical and geological lab test results, a preliminary site stratigraphy profile was developed. The site stratigraphy shown in Table 2 and on Plate A-4 represents AGS's estimate of the thicknesses of the units. The subsurface stratigraphy along the SWOO cross section trending east to west towards the ocean, crossing the LMT, is presented on Plate A-4.

Boring B-1 was drilled in the beach access parking lot at the intersection of Sloat Boulevard and the Great Highway. Boring B-1 encountered approximately 20 feet of brown, loose to medium dense poorly-graded sand, with lenses of silty sand. Approximately 10 feet of dense, black and gray poorly graded sand with silt, underlies the medium dense sand. Approximately 40 feet of dense to very dense, bluish gray poorly graded sand, underlies the dense sand, which is underlain by 30 feet of bluish gray, medium dense to very dense silty sand and poorly graded sand with silt. Boring B-1 encountered very stiff fat clay at about 100 feet depth to the bottom of the boring at about 101.5 feet depth.

Boring B-2 encountered approximately 12 feet of loose to medium material comprised of grayish brown, poorly graded sand with trace silt. This is underlain by approximately 13 feet of dark gray, medium to dense poorly graded sand. This is underlain by about 25 feet of reddish brown and gray, dense to very dense, poorly graded sand and 31.5 feet of dark gray, loose to very dense silty sand and sand with silt, or very soft sandy clay.

Boring B-3 encountered approximately 12 feet of loose to medium dense brown and grayish brown sand overlying approximately 5 feet of medium dense dark gray poorly graded sand. These layers are underlain by approximately 40 feet of dense to very dense poorly graded sand with silt and silty sand. This layer is underlain by 40 feet of very dense silty sand and poorly graded sand and very stiff silt with sand. At about 100 feet depth, fat clay was encountered, to the bottom of the boring at about 101.5 feet depth.

Boring B-4 encountered approximately 13 feet of loose to medium dense, poorly graded sand, underlain by about 5 feet of medium dense to dense poorly graded sand with silt and silty sand. Underlying this layer is 50 feet of dense to very dense silty sand and dense clayey sand, including a five feet layer of very stiff fat clay. Abundant mica was identified in samples between about 35 feet to 45 feet depth. The dense silty sand layer is underlain by 5 feet of dense silty sand and 6.5 feet of fat clay up to the bottom of the boring at about 81.5 feet depth.

Boring B-5 encountered approximately 9 feet of loose to medium dense, reddish brown silty sand, underlain by approximately 6 feet of medium dense, yellowish brown, poorly graded sand with silt. This layer was underlain by approximately 36.5 feet of dense to very dense brown poorly graded sand with silt and very dense reddish brown silty sand, to the bottom of the boring at about 51.5 feet depth.

Boring B-6A encountered fill up to the bottom of the boring at about 38 feet depth. The fill was brown and reddish brown, dense to very dense, poorly graded sand with silt. Boring B-6 encountered refusal on concrete. Boring B-6B was drilled adjacent to Boring B-6A with rotary wash up to a depth of about 30 feet without sampling and continued to about 35.5 feet with sampling. Boring B-6B encountered reddish brown, dense, poorly graded sand with silt fill from 30 to 35.5 feet depth. Boring B-6B refused at about 35.5 feet depth.

TABLE 2 - SUMMARY OF THE SUBSURFACE CONDITIONS

Reach	Representative Borings and CPTs Identification	Loose to Medium Dense Sand Thickness (feet)	Medium Dense Sand Thickness (feet)	Sand / Sand with Silt[1] Layer Thickness (feet)	Sand[1], Silty Sand[1], Silt[2], Clay[2] Layer Thickness (feet)	Maximum Depth of Exploration (feet)
North	B-1, R3-1*, B-6**, CPT-2	20	5-15	50-65	>31.5	101.5
EQR	B-2, CPT-3, CPT-4	10-20	0-10	30-40	>31.5	100.5
Rubble	B-3, B-4, B-5**, CPT-7, CPT-9	5-15	5-10	35-55	>31.5	101.5
Bluff	B-5, R2-1*, CPT-10	10-20	5-15	40-60	>6.3	79.7
South	B-6, R1-C2*, R1-C3*, R1-C1*, R1-B3*, R1-B1*, R1-A1*, CPT-11, CPT-12, CPT-13	30-40	0-10	>10	>40	76.3

*Boring from AGS 2010, **Boring from AGS 1989

Notes: 1. Sand, Sand with Silt, and Silty Sand were generally dense to very dense in these layers.
2. Silt and Clay were generally stiff to hard in these layers.

3.1.2 Groundwater

Depth to groundwater was measured in MW-1, a companion well to Boring B-1, and MW-2 and MW-3, companion holes to Borings B-4 and B-5, respectively. Groundwater was encountered at a depth of about 22 feet below grade in MW-1, at the approximate elevation of sea level at the time of the first groundwater reading. The rotary wash method prevented groundwater readings in the remaining borings, Borings B-2, B-3 and B-6.

Table 3 presents depth to groundwater encountered in each of the exploration monitoring wells, as well as in borings from previous explorations.

TABLE 3 – GROUNDWATER DATA

Well ID	Date Measured (feet)	Ground Surface Elevation (feet)	Groundwater Elevation (feet)	Depth to Groundwater (feet)	Total Depth (feet)	Source
MW-1	3/15/2019	+31.4	+9.4	21.96	25.09	AGS 2019
	5/31/2019	+31.4	+8.7	22.70	25.04	AGS 2019
	6/28/2019	+31.4	+8.1	23.31	26.02	AGS 2019
	7/27/2019	+31.4	+8.2	23.18	25.08	AGS 2019
	10/2/2019	+31.4	+9.0	22.41	25.05	AGS 2019
	11/6/2019	+31.4	+8.3	23.08	25.05	AGS 2019
	12/10/2019	+31.4	+8.9	22.46	25.05	AGS 2019
	2/5/2020	+31.4	+9.1	22.28	25.05	AGS 2020
	4/1/2020	+31.4	+8.6	22.80	25.05	AGS 2020
	4/8/2020	+31.4	+8.9	22.55	25.05	AGS 2020
	5/29/2020	+31.4	+8.9	22.48	25.05	AGS 2020
	6/26/2020	+31.4	+8.1	23.34	25.05	AGS 2020
MW-2	3/15/2019	+30.4	+8.4	22.01	27.70	AGS 2019
	5/31/2019	+30.4	+7.8	22.58	26.66	AGS 2019
	6/28/2019	+30.4	+7.4	23.02	27.63	AGS 2019
	7/24/2019	+30.4	+7.6	22.81	27.28	AGS 2019
	10/2/2019	+30.4	+8.6	21.76	27.64	AGS 2019
	11/6/2019	+30.4	+8.1	22.35	27.58	AGS 2019
	12/10/2019	+30.4	+8.4	22.04	27.57	AGS 2019
	2/5/2020	+30.4	+8.0	22.42	27.57	AGS 2020
	4/1/2020	+30.4	+7.7	22.75	27.34	AGS 2020
	4/8/2020	+30.4	+8.0	22.39	27.34	AGS 2020
	5/29/2020	+30.4	+8.6	21.85	27.34	AGS 2020
	6/26/2020	+30.4	+7.4	23.05	27.34	AGS 2020
MW-3	3/15/2019	+28.9	+5.6	23.33	28.30	AGS 2019
	5/31/2019	+28.9	+4.2	24.75	29.32	AGS 2019
	6/28/2019	+28.9	+5.1	23.79	28.22	AGS 2019
	7/24/2019	+28.9	+5.5	23.41	28.22	AGS 2019
	10/2/2019	+28.9	+5.8	23.07	28.23	AGS 2019
	11/6/2019	+28.9	+5.4	23.55	28.20	AGS 2019
	12/10/2019	+28.9	+5.4	23.50	28.22	AGS 2019
	2/5/2020	+28.9	+5.4	23.51	28.22	AGS 2019
	4/1/2020	+28.9	+5.3	23.62	28.22	AGS 2020
	4/8/2020	+28.9	+5.3	23.64	28.22	AGS 2020
	5/29/2020	+28.9	+5.9	23.05	28.22	AGS 2020
	6/26/2020	+28.9	+5.1	23.84	28.22	AGS 2020
B-5 (AGS)	5/24/1989	+29.4	+10.4	19.0	60	AGS 1989
B-6 (AGS)	5/24/1989	+31.4	+8.4	23.0	70	AGS 1989
HLA-54	6/24/1977	+31.9	+11.4	20.5	101.5	HLA 1977
WC-4	6/6/1977	+36.4	+6.9	29.5	80	W-C 1977
WC-10	6/6/1977	+48.5	+13.5	35.0	60	W-C 1977
B-2 (GTC)	10/8/2015	+32.4	+20.1	12.3	111.5	GTC 2016

3.2 Geotechnical Laboratory Tests

The index properties and engineering properties obtained for each of the geologic units underlying the project site are briefly discussed below and are shown on Table 4.

Engineering parameters for strength, moisture content, unit weight, compressibility, and permeability were derived from the results of field and laboratory data presented in the GDR (AGS, 2020).

The laboratory tests performed on selected soil and rock samples collected from the explorations, to evaluate the engineering properties of the materials, included moisture content and density, particle size, Atterberg limits, unconfined compressive strength, and corrosivity. The soil classifications and laboratory tests were conducted according to applicable ASTM Standards and other generally accepted professional standards, as discussed in the GDR.

Moisture content and density tests were performed on selected samples to evaluate their consistencies and the moisture variation throughout the explored profile. For calculation of dry density, the moisture content was considered representative of the entire sample. The results of the moisture content and density calculations are shown in detail in GDR (AGS, 2020a). Total unit weight and moisture content are obtained for artificial fill at several locations. The average unit weight was computed to be 120 pounds per cubic feet (pcf) for fill. Based on the data, the average total unit weight of 120 pcf is considered for dune sand in the numerical analyses.

Atterberg limits and particle size analysis were conducted on selected samples. The results of the particle size analyses are provided in detail in GDR (AGS, 2020).

Unconsolidated-undrained triaxial (UUTX) compression tests were performed on selected undisturbed soil samples, by Cooper Testing Laboratory, to evaluate soil's compressive strength under a triaxial confining stress approximately equal to the effective overburden stress of the in-situ sample. The results of the unconfined compressive strength tests are included in GDR (AGS, 2020).

A friction angle of 33 and zero cohesion was selected to represent the fill strength in drained static conditions.

Under rapid loading conditions, such as seismic loading, liquefaction may occur. In this scenario, AGS selected residual shear strength of 800 psf for the liquefiable soils, such as fill materials and Dune sand below groundwater level, for use in the dynamic analyses after liquefaction triggers. Colma Formation and Merced Formation will not liquefy and friction angles of 36 and 27 degrees were used in the analyses for these materials.

Shear wave velocities of 550, 550, 860 and 750 ft/sec were assigned to the artificial fill, dune sand, Colma and Merced Formations, respectively.

TABLE 4 – SUMMARY OF LABORATORY DATA

Boring ID	Material Type	Elevation (NAVD88)	Depth	Dry Density	Moisture Content	Atterberg Limits			Fines Content	UUTX
						Liquid Limit	Plastic Limit	Plasticity Index		Su-peak (=Deviator Stress/2)
		(feet)	(feet)	(pcf[1])	%	%	%	%	%	(psf)
B-1	SP	31	6	102.2	5.2				2	
B-1	SP-SM	31	25				NP	NP	5.0	
B-1	SP	31	33	134.1	14.6	NP	NP	NP	2.3	
B-1	SP	31	35						3.7	
B-1	SP-SM	31	45	124	13.3		NP	NP	5.8	
B-1	SP	31	50						7.9	
B-1	SM	31	62.5	106.7	21.1		NP	NP	13.4	
B-1	SP-SM	31	70			NP	NP	NP	5.6	
B-1	SM	31	81			19	20	NP	33.7	
B-1	CH	31	101	74.2	46.4	61	30	31		3115
B-2	SW-SC	30	1			31	23	8	12.5	
B-2	SP	30	5						2	
B-2	SP	30	13	114.2	2.3				0.7	
B-2	SP	30	15						3.3	
B-2	SP	30	20.5	104.2	19.5				3.0	
B-2	SP-SM	30	25			NP	NP	NP	8.8	
B-2	SP-SM	30	40			NP	NP	NP	10.0	
B-2	SP-SM	30	45				NP	NP	7.1	
B-2	SM	30	50			23	23	NP		
B-2	CL	30	60			32	22	10	61.4	
B-2	SP-SM	30	70							
B-2	SM	30	80				NP	NP	15.3	
B-3	SP	28.5	4							
B-3	SP	28.5	5.5							
B-3	SP	28.5	11							
B-3	SP-SM	28.5	25			NP	NP	NP		
B-3	SP-SM	28.5	32	109.3	15.3		NP		5.3	
B-3	SP-SM	28.5	40			NP	NP	NP	8.7	
B-3	SM	28.5	45				NP	NP	13.7	
B-3	SP-SM	28.5	53.5	101.8	25.2	NP	NP	NP	7	
B-3	SM	28.5	60			NP	NP	NP	13.3	
B-3	SM	28.5	70			NP	NP	NP	13.6	
B-3	CH	28.5	101	87.1	35.2	61	27	34		4944
B-4	SP-SM	29	2.5				NP	NP	9.4	
B-4	SP	29	5.5						0.8	
B-4	SP	29	7							
B-4	SP-SM	29	13						4.8	
B-4	SM	29	16				NP	NP	27.2	
B-4	SM	29	19			NP	NP	NP	20	
B-4	SP-SM	29	25			NP	NP	NP	11.5	
B-4	SM	29	36				NP	NP	12.3	
B-4	SM	29	40			37	26	11	45.5	
B-4	CH	29	46	89.2	33	60	25	35		6065
B-4	SM	29	55			NP	NP	NP	18.2	
B-4	SM	29	70			NP	NP	NP	23.2	
B-5	SM	31	2						19.6	
B-5	SM	31	5						16.7	
B-5	SM	31	6.5						24.1	
B-5	SM	31	8			21	20	1	26.2	
B-5	SP-SM	31	11						7.8	
B-5	SP-SM	31	15.5						8.6	
B-5	SP-SM	31	18.5						9.6	
B-5	SP-SM	31	25				NP	NP	10.8	
B-5	SP-SM	31	35						11.3	
B-5	SM	31	40						12.1	
B-5	SP-SM	31	45			NP	NP	NP	11.6	
B-6	SP-SM	70	3						5	
B-6	SP-SM	70	5						10.5	
B-6	SM	70	11						15.8	
B-6	SP	70	13						3.5	
B-6	SP-SM	70	17						10.1	
B-6	SP-SM	70	25			NP	NP	NP	8.1	

4. Seismic Design Considerations

4.1 Seismic Design Criteria

According to the study objectives, the seismic criteria of the South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection Project in San Francisco need to be developed for a major level of seismic activity, equivalent to a maximum earthquake on the San Andreas and other major active faults in the San Francisco area.

In accordance with the guideline outlined in General Seismic Requirements for Design of New Facilities and Upgrade of Existing Facilities published by SFPUC GSR, South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection Project needs to be designed based on a magnitude 8.05M_w event on the San Andreas, representing a repeat of the great San Francisco earthquake of 1906. This characterization is consistent with the latest consensus of a significant sector of the earth sciences community as expressed by the Working Group on Northern California Earthquake Probabilities (WGCEP, 2008).

4.2 Development of Earthquake Ground Motion for SSSI

This Section describes the criteria for seismic design of the South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection Project. Since this technical memorandum is focused on the SSSI study of the project, only development of the time-history acceleration of the at the base of the model is discussed. More details can be found in the GIR prepared by AGS.

The target spectrum was developed according to SFPUC 2014 GSR for the bottom level of the analyses. First, seismic sources were identified and characterized. Next a set of applicable ground motion prediction equations were selected to estimate the horizontal ground motion measures at the bedrock level. Since depth to bedrock is very deep, the MCE_R response spectrum and acceleration time histories of the base motion developed in accordance with Section 21.1.1 of ASCE 7-16 were adjusted upward using site coefficients in accordance with Section 11.4.3 of ASCE 7-16 consistent with the classification of the soils at the profile base. Then, over a range of periods (0.01 to 10.0 sec.), spectral acceleration hazard curves were developed. Five (5) percent damping response spectrum for the 5 percent chance of exceedance in 50 years (return period of 975 years) was calculated using the hazard curves. Target spectrum was calculated as the envelope of the developed response spectrum and minimum of SFPUC 2014 GSR. Controlling source of the seismic hazard at the site location is the San Andreas Fault. This fault can produce an earthquake of 8.05 M_w at 2.6 kilometers from the project site. Three pairs of ground motions were selected and matched to the target spectrum developed which were used for SSSI analysis.

4.2.1 Ground Motion Parameters at the Bedrock Level

To develop the design ground motion parameters, Probabilistic Seismic Hazard Analysis (PSHA) was performed using the OpenSHA "Hazard Spectrum Application" version 1.7.0 (2018) to estimate the site accelerations associated with the 975-year return period seismic events at the site. A 975-year return period ground motion corresponds to a 5 percent probability that the ground motion will be exceeded over a 50-year period. In accordance with the SFPUC 2014 GSR and a Performance Class III, a 975-year return period spectrum has been developed for the site by using applicable, equally weighted five attenuation relationships of the NGA West2. The UCERF3 fault source model (2015) was selected for the analyses. Based on our analyses, site class-D response spectra with 5 percent probability of being exceeded over a 50-year period were computed and rotated to ROTD100 maximum direction using correction factors published in Shahi and Baker (2014).

For development of the deterministic horizontal spectral accelerations, AGS equally weighted results from all five NGA West2 attenuation relationships to estimate 84th percentile ground motions for the design earthquake event (8.05M_w and distance of 2.6 kilometers based on guidance of the SFPUC 2014 GSR), values were held to a minimum of those shown on Figure 21.1-1 of ASCE 7-16 for Site Class D (for the development of the target spectrum). In addition to the probabilistic 5-percent response spectrum, an 84th percentile deterministic horizontal acceleration response spectrum at the base of the numerical model was developed based on an arithmetic average of these NGA West2 correlations. The accelerations were rotated to ROTD100 maximum direction using correction factors published in Shahi and Baker (2014). Probabilistic, deterministic, minimum spectra, and design spectra for the project are presented on Table 5 in accordance with the ASCE7-16 (ASCE-SEI, 2016).

TABLE 5 - TARGET RESPONSE SPECTRA

Structural Period (sec)	975-year Probabilistic (g)	84th-percentile Deterministic (g)	Lower of Probabilistic or Deterministic (g)	Deterministic Lower Limit (g)	Design Response Spectrum (g)
0.01	0.96	0.99	0.96	0.65	0.96
0.02	0.97	0.99	0.97	0.69	0.97
0.03	1.01	1.01	1.01	0.74	1.01
0.05	1.05	1.04	1.04	0.83	1.04
0.08	1.23	1.23	1.23	0.96	1.23
0.10	1.55	1.41	1.41	1.05	1.41
0.15	1.90	1.68	1.68	1.28	1.68
0.20	2.03	1.86	1.86	1.50	1.86
0.25	2.07	2.03	2.03	1.50	2.03
0.30	2.05	2.13	2.05	1.50	2.05
0.40	2.00	2.25	2.00	1.50	2.00
0.50	1.92	2.23	1.92	1.50	1.92
0.75	1.52	1.84	1.52	1.50	1.52
1.00	1.24	1.51	1.24	1.50	1.50
1.50	0.85	1.06	0.85	1.00	1.00
2.00	0.65	0.78	0.65	0.75	0.75
3.00	0.44	0.53	0.44	0.50	0.50
4.00	0.31	0.38	0.31	0.38	0.38
5.00	0.25	0.29	0.25	0.30	0.30
7.50	0.13	0.15	0.13	0.20	0.20
10.00	0.08	0.09	0.08	0.15	0.15

4.2.2 Ground Motion Development for Seismic Soil Structure Interaction (SSSI) Analysis

Three pairs of acceleration time histories were developed for the SSSI analysis. These pairs represent the design Earthquake Scenario for the project site at base of the numerical model level. To develop the time histories, first AGS selected earthquake records which relatively match the properties of the design earthquake at the site location. We have considered the earthquake magnitude, distance to the site, site conditions, and mechanism of source to choose the appropriate set of earthquake time history records. Summary of the chosen ground motions is shown in Table 6. Acceleration time history records were obtained from PEER NGA-West2 Database (Ancheta, et al., 2013). Next, we scaled the original ground motions (obtained from

PEER NGA-West2 Database) to the target response spectrum developed in Section 4.2.1. Scaling was performed using the square root mean squares (SRSS) spectrum for each horizontal component set. Both horizontal components of each earthquake record were scaled using the same scale factor. More details of selecting of the ground motions and matching to the target spectrum can be found in the GIR (AGS, 2020). Scaled acceleration time histories (along and perpendicular to the tunnel) for the chosen ground motions are presented on Plate A-5.

TABLE 6 - GROUND MOTIONS USED IN THE SSSI ANALYSIS

Scenario	Record Sequence Number in PEER Database	Seed Earthquake Name	Year	Station Name	M_w	Mechanism	R_{rup} (km)	V_{s30} (m/sec)
A	RSN68	San Fernando	1971	LA - Hollywood Stor FF	6.61	Reverse	22.77	316
B	RSN292	Irpinia_ Italy-01	1980	Sturno (STN)	6.9	Normal	10.84	382
C	RSN725	Superstition Hills-02	1987	Poe Road (temp)	6.54	Strike Slip	11.16	316

5. Seismic Deformation Evaluation

5.1 Summary

AGS developed five (3-D) dynamic SSSI numerical analyses to simulate the response of the wastewater tunnel, retaining secant pile wall, and supporting tie-back cables to the design Earthquake Scenarios. Dynamic analyses were performed for the condition in which the front soil of the wall is eroded to minimum Elevation of +2 feet. Maximum height of the retained soil above the eroded ground surface is estimated to be about 19 feet. Numerical analyses were developed using FLAC3D (Itasca Consulting Group Inc, 2019) software. Tunnel, wall, piles, tie-back cables, and surrounding geomaterial are included in the analyses. To achieve proper mesh size capable of capturing waves up to a frequency of 10 Hz and focus on response of the model in the most critical segments, five (5) segments along the alignment were selected which represent the stations identified in Table 1. A summary of the information for each segment which includes geometry and number of elements is presented in Table 8 and Table 9. FLAC3D computer program was used to calculate the response of each segment to the design base acceleration time histories. Ground motions developed in Section 4.2 were applied concurrently in two horizontal orthogonal directions. The main purpose of the numerical dynamic analyses is to calculate the seismic demand on the secant pile wall, supporting tie-back cables and on the existing tunnel.

Mohr-Coulomb constitutive model was used for all the non-liquefiable materials. Potentially liquefiable layers including dune sand and existing fill were modeled with Finn-Bryne (Byrne, 1991) material constitutive model to simulate the liquefaction conditions in the subsurface materials. Tunnel lining, secant pile wall, CLSM, and tie-back cables were modeled using elastic material.

Tunnel lining was modeled using FLAC3D's liner elements. This type of element was selected since it could consider the interaction of the interface of the tunnel and surrounding media.

Subsurface soils were numerically modeled with hexahedral and tetrahedral elements. Element sizes were chosen in a way that waves up to a frequency of 10 Hz could propagate through the soil model.

In addition to the damping created by the nonlinear behavior of the material, hysteretic damping was applied to the materials. Hysteretic damping parameters were chosen to represent the corresponding materials.

5.2 Engineering Properties of the Materials

The main engineering properties of soils required for the site response analyses consist of unit weights, drained and undrained shear strengths, residual shear strengths of liquefied soils, elastic modulus, Poisson ratio, and modulus reduction and damping relationships for the subsurface soils in the project site area. In the following subsections, the data on these properties are presented and discussed.

5.2.1 Elastic Properties of Geomaterials

Two elastic parameters (modulus of elasticity, E, and Poisson ratio, ν) for various earth materials were estimated based on available literature review and field geotechnical data. These values were used to compute shear and bulk modulus inputs used in the FLAC3D analysis.

5.2.2 Soil Strength Parameters

Table 7 summarizes unit weight and shear strength parameters for all the subsurface materials used in the 3-D SSSI analyses.

5.2.1 Properties and Constitutive Models of Geomaterials

Prior to dynamic analysis of each model, a static effective-stress analysis was performed for each segment. Drained material properties are used to perform this analysis. This analysis is performed to initialize the stresses and bring models to the expected conditions prior to the Earthquake Scenarios.

Mohr-Coulomb and Finn-Byrne constitutive models (Byrne, 1991) were used in FLAC3D to model the geomaterials in the analyses. The Mohr-Coulomb model was used in static analyses to establish equilibrium prior to dynamic analyses. Geostatic equilibrium was reached prior to performing further analysis. Based on our liquefaction analysis only loose to medium dense (fill and dune sand) below groundwater level material is susceptible to liquefaction. The Finn-Byrne constitutive model, which can capture pore pressure rise and liquefaction induced by earthquake loading, was used to model the liquefiable materials below the water table (Byrne, 1991). The Mohr-Coulomb model was used for the fill, dune sand above the water table, and underlying Colma and Merced formations as they are not liquefiable.

Table 7 summarizes the material parameters used in the FLAC3D analyses. The hysteretic model was used to simulate the dynamic characteristics as defined by the modulus reduction factor (G/G_{max}) and damping ratio curves. Dynamic modulus of elasticity for each geomaterial was determined based on the shear wave velocity and density of the material. Hysteretic damping was used to represent the damping in the geomaterials during the earthquake shakings. Damping parameters were determined according to the FLAC3D manual recommendation and summarized in Table 7.

5.2.2 Structural Materials

Elastic material parameters were used to model the behavior of structures (tunnel's lining, piles, and tie-back cables, and secant wall). Elastic materials were defined by two parameters. Elastic modulus and Poisson ratio were used to define the materials parameters. Elastic material parameters for the materials are summarized in Table 7.

TABLE 7 - SUMMARY OF MATERIAL PROPERTIES USED IN FLAC3D ANALYSES

Material	Total Unit Weight [Υ] (pcf)	Friction Angle [φ] (Degrees)	Residual Shear Strength [S_u] (psf)	Poisson Ratio [v]	Elastic Modulus [E] (ksf X 1000)	Shear Wave Velocity [V_s] (ft/sec)	Hysteretic Damping Parameters	
							L_1^*	L_2^*
Fill	120	33	800	0.27	2866.08	550	-3.325	0.823
Dune Sand	120	34	800	0.27	2866.08	550	-3.325	0.823
Colma Formation	125	36	NA	0.33	7644.29	860	-3.325	0.823
Merced Formation	125	27	NA	0.35	5901.26	750	-3.325	0.823
Tunnel Lining	150	NA	NA	0.2	58003.26	NA	NA	NA
CLSM	150	NA	NA	0.2	82080.86	NA	NA	NA
Wall	150	NA	NA	0.2	51912.05	NA	NA	NA
Tie-Back	150	NA	NA	0.2	432000.46	NA	NA	NA

NA: Not Applicable

- L_1 and L_2 are the extreme values of logarithmic strain. $L = log_{10}(\gamma)$, $s = \frac{L_2-L}{L_2-L_1}$, and $M_s = s^2(3-2s)$. M_s and γ are secant modulus and strain, respectively.

5.3 Numerical Analyses

Five comprehensive numerical models were developed to examine the response of the numerical analyses to the design earthquake level. We included the tunnel, secant pile wall, and supporting tie-back cables in the developed numerical models. Numerical 3-D models extended to the elevation of -131feet. Limits of each model are summarized in Table 8.

Geometry of the models were developed using the building blocks module of the FLAC3D. Separate "groups" were assigned to the zones[2] for the different subsurface layers to distinguish between the different geomaterials. Developed geometry was discretized into a mesh for each segment. Responses of the developed models to the design Earthquake Scenarios were evaluated using FLAC3D software (Itasca Consulting Group Inc, 2019). FLAC3D uses finite difference method (FDM) to solve the governing motion equations. Ground motions were applied as stresses at the base of the model and appropriate boundary conditions were applied depending on the stage of the analyses. In the following sections, details of the numerical modeling are presented. Following stages were considered in the response evaluation of the tunnel, secant wall, and supporting tie-back cables:

1. Initial condition (geostatic equilibrium under gravity loads);
2. Adding the hydrostatic water pressure to the model water to the model;
3. Excavation to place existing tunnel and adding the liner of the tunnel;
4. Installation of the piles;
5. Adding the tie-Back cables;
6. Adding the CLSM;
7. Excavation to model the erosion;
8. Seismic response Including:
    a. Transient response to the earthquake excitation; and
    b. Response at post liquefaction condition.

[2] "The finite volume zone is the smallest geometric domain within which the change in a phenomenon (e.g., stress versus strain) is evaluated". (FLAC3D Manual)

5.3.1 Geometry

As mentioned earlier, the numerical analyses were performed to examine the seismic response of the proposed structure including the secant pile wall to protect the existing wastewater tunnel against the coastal erosion. In addition to the protecting structure, seismic response of the existing sewer tunnel is also calculated. Five (5) segments, representing various subsurface conditions were selected from the study zone. Numerical models were developed for each segment to evaluate the response of the above-mentioned components to the design Earthquake Scenarios. Summary of the geometry of the 5 segments is presented in Table 8.

TABLE 8 - SUMMARY OF THE GEOMETRY FOR THE SEGMENTS OF THE NUMERICAL ANALYSIS

Segment	X_{min}[ft]	X_{max}[ft]	Y_{min}[ft]	Y_{max}[ft]	Z_{min}[ft]
North Reach	+355	+415	-157	+162	
EQR	+1140	+1190	-262	+57	
Rubble Reach	+1722	+1781	-175	+144	-131
Bluff Reach	+2481	+2536	-105	+214	
South Reach	+3148	+3223	+19	+339	

Geometry of the secant pile wall, tunnel lining, and supporting tie-back cables are presented on Plates B-2, C-2, D-2, E-2, and F-2 for the North Reach, EQR, Rubble Reach, Bluff Reach, and South Reach, respectively. Models were created in the Cartesian coordinates system. Positive direction for the X axis points to the North and parallel to the proposed retaining wall and for the Y axis positive direction is toward the West and transverse to the proposed retaining wall. Direction of the Z axis is vertical and is defined according to the right-hand rule.

Details of the subsurface conditions are presented in Section 2.

5.3.2 Finite Difference Model

The numerical models were developed to provide responses to the seismic waves up to 10 Hz frequencies. Therefore, sizes of the zones were selected to assure the meshes were capable of propagating at least 10 Hz waves. Mesh of the segments are shown on Plates B-1(a and b), C-1(a and b), D-1(a and b), E-1(a and b), and F-1 (a and b) for the North Reach, EQR, Rubble

Reach, Bluff Reach, and South Reach, respectively. Meshes were gradually refined from bottom to top of the model for the following reasons:

- Most of the interaction between the tunnel, secant pile wall and surrounding soil takes place relatively close to the surface.
- Dune sand below the water table is susceptible to liquefaction. Therefore, finer mesh was required to appropriately capture the physics and material behavior.

Tunnel, secant pile wall, and tie-back cables (shown on Plates B-2, C-2, D-2, E-2, and F-2.), were modeled using FLAC3D's Liner, Structural Pile, and Structural Cable elements, respectively. The proposed primary piles are made of concrete and the secondary piles have a composite section. Since the pile element in FLAC3D cannot consider a composite section. Therefore, an equivalent section was defined for the secondary pile elements. Physical and mechanical parameters of the material (density and modulus of elasticity) and the pile section properties (area and moment of inertia) for secondary piles were adjusted to assure the defined section in FLAC3D is representative of the equivalent cross section of the piles.

Numbers of the numerical elements for each segment are summarized in Table 9.

TABLE 9 - NUMBER OF ELEMENTS IN FDM

Segment	Number of Zones	Number of Structural Elements		
		Liners[1]	Piles[2]	Cables[2]
North Reach	92868	1650	244	210
Bluff Reach	146028	1840	159	150
Rubble Reach	120717	2208	206	150
South Reach	180356	1920	301	240
EQR	115281	1710	187	150

1- Liner Elements are used to model the interface between the tunnel lining and surrounding soil.

2- Each pile or cable include multiple pile or cable elements in the FLAC3D model.

5.3.3 Boundary Conditions

Boundary conditions for all the segments were set to be identical. For each segment, static and dynamic analyses required different boundary condition assumptions. For the static analysis which numerical model is analyzed under gravity loads following boundary conditions are applied:

- At the bottom of the numerical model, vertical direction degrees of freedom (normal to the boundary plane) were fixed. Other degrees of freedom (parallel to the boundary plane) were free.
- At the sides of the numerical model, degrees of freedom in horizontal direction and normal to the boundary were fixed and vertical degrees of freedom were free.

For the dynamic analysis (response of the model to the Earthquake Scenarios), following boundary conditions were imposed:

- At the bottom of the numerical model, absorbing boundary conditions were assigned, and earthquake excitation was applied as a stress-time history.
- Along the sides of the numerical model free field boundary conditions were applied.

5.3.4 Dynamic Response Analyses

This Section describes the procedure of how the analyses were performed in FLAC3D. Analysis procedure for all the segments was identical. Analyses included two general steps and multiple sub steps. At the first step, the model was brought to geostatic equilibrium and conditions in which an earthquake is expected to excite the project site. Next, the response of the developed numerical model was analyzed to the design Earthquake Scenarios. Detailed procedure in numerical analyses is presented in Section 5.3.

5.3.4.1 Geostatic Equilibrium Analysis

The initial static shear stress can affect the dynamic behavior of the numerical model. Therefore, prior to seismic excitation, appropriate constitutive material models were applied to each geomaterial. Water table was assigned, the excavated tunnel was included, and linings were added to the model, secant pile wall elements were created, and tie-back cables were added. Next, grouting was added, and pretension forces were applied to the cables. Controlled-low Strength material (CLSM) was added to the model afterwards. Eventually, an eroded portion of the soil was removed from the model. Plates B-2, C-2, D-2, E-2, and F-2 show the initial pore water pressure for each segment. At the end of each step, the numerical model of each segment was analyzed under gravity loads and brought to the geostatic equilibrium in which the sum of forces in all directions was zero.

5.3.4.2 Dynamic Analysis

After the numerical model was brought to geostatic equilibrium, displacements throughout the mesh and Structural Elements reset to zero and boundary conditions changed to free field along the sides and quiet boundary condition was applied along the bottom of the numerical model. In addition, the constitutive model for the potentially liquefiable soils was changed to "*Finn-Byrne*" model (Byrne, 1991) to capture the porewater pressure generation and proper non-linear behavior of such material during the earthquake excitation. Dynamic hysteretic damping was applied to the soil layers. Damping parameters were assigned to materials to simulate the appropriate hysteretic damping during the earthquake shaking for the soils.

Three (3) pairs of acceleration time histories were used in the analyses. Shaking was applied at the bottom of the numerical model as a stress boundary condition. Input motions (free field) are presented on Plates 5.

5.3.5 Dynamic Analysis Results

Response of the soil layers, tunnel, secant pile wall, and tie-back cables to Earthquake Scenarios A, B and C are presented in this Section. Earthquake Scenarios represent the design earthquake as discussed in Section 4. Displacement response at each point was computed as a time history. Results were presented at elapsed time snapshots. Snapshots represent the critical shaking soil behavior to examine the structural elements during the critical stages of the Earthquake Scenarios.

Results of SSSI analyses indicate that seismically-induced pore water pressures developed during each Earthquake Scenarios and increased significantly to a value close to effective stresses in loose to medium dense fill materials and dune sand. Therefore, liquefaction triggered in loose to medium dense fill materials and dune sand below groundwater level. As expected, seismically-induced excess pore water pressure did not reach to a value in which liquefaction triggering occurs in Colma and Merced Formation. The results of the FLAC3D analyses are in general agreement with liquefaction potential analyses presented in the GIR.

5.3.5.1 Secant Pile Walls

Displacement responses along longitudinal and transverse directions for the secondary pile approximately located at the middle of each segment for Earthquake Scenarios A, B, and C are presented at several time steps of the dynamic analyses on Plates B-8, B-17, B-25, C-8, C-17, C-25, D-8, D-17, D-25, E-8, E-17, E-25), F-8, F-17, and F-25. In addition, for the entire course of the dynamic analyses, time history of the transverse direction displacements for the exposed portion of the wall are depicted on Plates B-6, B-15, B-23, C-6, C-15, C-23, D-6, D-15, D-23, E-6, E-15, E-23, F-6, F-15, and F-23. Results are shown in several elapsed times and at different elevations. Displacement of each elevation is presented relative to the bottom end of the pile. In all the models, Y-Direction is transverse to the proposed wall alignment. Positive direction of the transverse direction axis is toward the slope. X-Direction axis is longitudinal to the proposed wall alignment. Top portion of the wall which includes both primary and secondary piles is continuous. Maximum (Maximum Transient) and permanent displacement are presented in Table 10.

TABLE 10 - MAXIMUM AND PERMANENT DISPLACEMENTS AT TOP OF THE SECANT WALL

			Earthquake Scenario		
			A	B	C
North Reach	Max. Disp. [in]	longitudinal	-1.3	3.6	-4.2
		transverse	-3.8	+3.0	-2.8
	Perm. Disp. [in]	longitudinal	-1.3	3.5	-4.2
		transverse	-3.8	+3.0	-1.5
EQR	Max. Disp. [in]	longitudinal	-3.6	-2.0	+1.9
		transverse	+9.2	+7.0	+0.5*
	Perm. Disp. [in]	longitudinal	-3.6	-2.0	+0.8
		transverse	+9.2	+7.0	+0.1[1]
Rubble Reach	Max. Disp. [in]	longitudinal	+2.5	+1.9	-1.8
		transverse	-7.1	+5.2	+0.9
	Perm. Disp. [in]	longitudinal	+1.2	+0.8	-1.7
		transverse	+1.8[2]	+5.2	+0.7[3]
Bluff Reach	Max. Disp. [in]	longitudinal	+0.5	-1.4	+1.2
		transverse	-3.5	-1.5[4]	-3.5
	Perm. Disp. [in]	longitudinal	+0.2	-1.4	+1.2
		transverse	-3.5	+0.1[4]	-3.5
South Reach	Max. Disp. [in]	longitudinal	+1.9	+3.2	+0.7
		transverse	-8.1	-3.6	-7.2
	Perm. Disp. [in]	longitudinal	+0.1	+2.3	+0.7
		transverse	-7.2	-3.4	-5.0

Note: All the displacement are calculated relative to the bottom of the pile.

1- Middle of the pile moves about 1 inch in the reverse direction.
2- Middle of the pile moves about 1.5 inches in the reverse direction.
3- Middle of the pile moves about 1 inch in the reverse direction.
4- Middle of the pile moves about 2.5 inches in the reverse direction.

North Reach:

Maximum and permanent displacements are presented in Table 10. In longitudinal direction, for the Earthquake Scenarios A and B, displacement in top portion of the pile (embedded inside the secant wall) is negligible. However, for Earthquake Scenario C, the pile moves monotonically from bottom to top toward the ocean. Displacement pattern in the pile implies tilting the entire pile about the bottom.

In the transverse direction, top of the piles (relative to the bottom) moves toward the ocean for the Earthquake Scenarios A and C. However, in Earthquake Scenario B, the lower one-third of the pile moves monotonically toward the ocean and the remaining two-third of the pile deforms toward the slope.

EQR:

In the longitudinal direction, for Earthquake Scenarios A and C, the deformation pattern in the piles implies tilting the entire piles about the bottom. However, in Earthquake Scenario B, the top portion of the piles, which are embedded inside the secant wall, deform less than the bottom. Scenario B will put higher demand on the pile in longitudinal direction.

In the transverse direction, for both Earthquake Scenarios A and C, from bottom to top, the pile deforms monotonically toward the ocean. However, in response to Earthquake Scenario B, the pile (relative to the bottom) deforms monotonically toward the ocean in the lower one-third of the pile. From this point upward, monotonic deformation of the pile is observed toward the slope. This deformation pattern indicates a lower potential failure plane which is consistent with calculated displacement of the soil.

Rubble Reach:

In the longitudinal-direction, displacement from bottom to top of the piles monotonically increases in response to Earthquake Scenario A. However, displacement pattern is different in response to Earthquake Scenarios B and C. The portion of the piles which are embedded inside the secant wall deforms negligibly and most of the deformation in longitudinal-direction takes place below the secant wall.

In the transverse direction, piles respond differently to each Earthquake Scenario. In response to Earthquake Scenario A, deformed shape which creates the maximum displacement at top of the piles monotonically moves toward the ocean from bottom to top. However, residual deformation of the piles from bottom to the middle of the piles is toward the ocean. From the middle to top of the piles, both maximum and permanent displacement is toward the slope. In response to Earthquake Scenarios B and C, displacement in the lower one-third of the piles monotonically increases toward the ocean and the remaining portion of the pile deforms toward the slope.

Bluff Reach:

In the longitudinal-direction, for Earthquake Scenarios A and C, piles monotonically deform toward the slope (from bottom to top the pile). However, a reverse deformation takes place in response to Earthquake Scenario B.

In the transverse direction, for Earthquake Scenarios A and C, piles deform monotonically toward the ocean from bottom to top. However, in response to Earthquake Scenario B, the bottom half portion of the piles deforms toward the ocean and the upper half portion of the piles deforms toward the slope.

South Reach:

In the longitudinal-direction and for the Earthquake Scenarios A and C, piles displacements increase monotonically (from bottom of the piles to the lower one-third) to the north. However, the upper two-thirds of the piles move monotonically toward the south in longitudinal-direction. Displacement of the piles to Earthquake Scenario B monotonically increases toward south from bottom to top.

In the transverse direction, the pile deforms monotonically toward the ocean (from bottom to top of the pile).

5.3.5.2 Tie-Back Cables

Maximum and residual forces in the tie-back cables were computed for the Earthquake Scenarios A, B and C and are presented in Table 11. As observed, maximum axial force in the cables does not exceed 66 kips which is below the capacity of the proposed cables. Maximum and residual axial force in the tie-backs are also presented on Plates B-9, B-10, B-18, B-19, B-26, B-27, C-9, C-10, C-18, C-19, C-26, C-27, D-9, D-10, D-18, D-19, D-26, D-27, E-9, E-10, E-18, E-19, E-26, E-27, F-9, F-10, F-18, F-19, F-26, and F-27.

TABLE 11 - MAXIMIM AND RESIDUAL AXIAL FORCE IN THE TIE-BACK CABLES[1]

	Earthquake Scenario A		Earthquake Scenario B		Earthquake Scenario C	
	Max Tension Force [kips]	Residual Tension Force [kips]	Max Tension Force [kips]	Residual Tension Force [kips]	Max Tension Force [kips]	Residual Tension Force [kips]
North Reach	48	46	64	52	**68**	**63**
EQR	48	34	**52**	**36**	**52**	28
Rubble Reach	56	23	**58**	**42**	55	21
Bluff Reach	58	42	52	**46**	**62**	0
South Reach	58	**56**	58	35	**62**	**56**

1- Bold numbers indicate maximum values for each segment.

5.3.5.3 Tunnel

In the previous Sections, response of the secant pile wall and tie-back cables to the Earthquake Scenarios were presented. To examine the adequacy of the existing tunnel structure to tolerate the displacement demand by the Earthquake Scenarios, displacements were evaluated at several points around the tunnel at the middle of each section. We have presented the relative displacement of the tunnel lining (Y-Z Plane) relative to the bottom of the pile as shown on Plates B-12, B-20, B--28, C-12, C-20, C-28, D-12, D-20, D-28, E-12, E-20, E-28, F-12, F-20, and F-28. All the presented displacements were measured relative to the bottom of the pile. Displacements at the tunnel nodes are presented in a section which makes a slight angle with the perpendicular section of the tunnel.

In addition to the displacement response at the middle section, we calculated the torsion (degrees per feet) for the middle one-third of the tunnel of each segment. We selected the middle one-third to avoid boundary effects (see Plates B-12b, B-20b, B-28b, C-12b, C-20b, C-28b, D-12b, D-20b, D-28b, E-12b, E-20b, E-28b, F-12b, F-20b, and F-28b). The shear strains generated by torsion might not be distributed uniformly in the tunnel cross section. Therefore, we selected four points on the tunnel cross section and calculated torsion at each points. The selected points are: 1) the farthest location on the tunnel wall from the ocean, 2) the closest location on the tunnel wall to the ocean, 3) the crown of the tunnel, and 4) the invert of the tunnel. All selected points were specified on the outer edge of the tunnel.

For all the segments and the Earthquake Scenarios, both maximum and permanent deformations of the tunnel lining were computed to be less than 0.5 inches.

North Reach:

For the Earthquake Scenario A, as presented on Plate B-12, tunnel permanent displacement in transverse direction is less than 3 inches and the tunnel settles less than 3 inches relative to the bottom of the pile.

As shown on Plates B-20 and B-28, less than 2 inches and 7 inches settlement was computed for the Earthquake Scenarios B and C, respectively.

EQR:

As presented on Plates C-12, tunnel permanent displacement in transverse direction is less than 9 inches (away from the pile) and the tunnel settles about 11 inches relative to the bottom of the piles.

As shown on Plates C-20, and C-28, tunnel permanent displacement in transverse direction is estimated to be about 1 inch toward the pile for the Earthquake Scenario B. Less than 8 inches and 3 inches of settlement was estimated for the Earthquake Scenarios B and C, respectively.

Rubble Reach:

For the Earthquake Scenario A, as presented on Plates D-12, permanent displacement of the tunnel (relative to bottom of the pile) is 5 inches. Settlement of the tunnel (relative to bottom of the pile) is estimated to be about 6 inches.

As shown on Plates D-20 and D-28, settlement of the tunnel is estimated to be less than 6 inches and 3 inches for the Earthquake Scenarios B and C, respectively.

Bluff Reach:

Displacement responses of the tunnel to the Earthquake Scenarios A, B, and C are shown on Plates E-12, E-20, and E-28, respectively. Permanent displacement of the tunnel (relative to the bottom of the pile) is estimated to be less than 4 inches, 1.5 inches, and 3 inches for the Earthquake Scenarios A, B and C, respectively. Settlement of the tunnel for the Earthquake Scenarios A, B, and C is estimated to be 2 inches, 4 inches, and 1.5 inches, respectively.

South Reach:

As presented on Plate F-12, for the Earthquake Scenario A, permanent displacement in transverse direction is estimated to be less than 6 inches and the tunnel settles less than 3 inches relative to the bottom of the pile.

As shown on Plate F-20, for the Earthquake Scenario B, permanent displacement of the tunnel (relative to the bottom of the pile) is estimated to be about 2.5 inches. Less than 2 inches of settlement is observed for Earthquake Scenario B.

As shown on Plate F-28, for the Earthquake Scenario C, permanent displacement of the tunnel (relative to the bottom of the pile) is estimated to be about 6 inches and 4 inches, respectively. Less than 1 inch settlement is calculated for the Earthquake Scenario C.

6. Conclusions and Recommendations

The results of the SSSI analyses provide realistic estimates and extent of liquefaction potential and consequences for the materials along the project alignment. The SSSI analyses also provide seismically-induced lateral deformations and settlements of the proposed secant wall and tie-backs and LMT. The results are summarized below:

- **Liquefaction Potential and Consequences -** Loose to medium dense fill and Dune sands are susceptible to liquefaction. The secant wall and tie-backs are extended below the potentially liquefiable soils. The tunnel invert is also below the liquefiable materials. Therefore, no liquefaction-induced settlement is expected for the tunnel. Liquefaction-induced excess pore water pressure is expected to influence the performance of LMT and the proposed secant wall. The final deformations of LMT and the secant wall presented in this report include the effect of liquefaction-induced pore water pressure.

  The SWOO backfill has high liquefaction potential. Since secant piles cannot be constructed deep enough at this location to provide sufficient lateral resistance, soil improvement such as deep soil mixing is expected to be performed which will results in densification of the existing backfill between LMT and 15 feet west of the secant wall alignment. Therefore, we expect that this section of the project will perform better that the remaining part of the project under seismic loading condition.
- **Secant Wall and Supporting Piles -** Maximum transient and permanent displacement at top of the secant wall (relative to bottom of the piles) in the majority of the dynamic analyses are less than 1 percent of the pile length. However, only for Earthquake Scenario A in EQR segment, South Reach, and Rubble Reach; and for Earthquake Scenario C South Reach, the calculated displacements exceeded 1 percent and remained below 1.5 percent of the pile length.
- **Lake Merced Transport (LMT) -** Dynamic analyses results show that LMT mostly moves as a rigid body and is not subjected to significant deformation. For most of the segments and in most of the Earthquake Scenarios, settlement of the tunnel lining is predicted to be less than 7 inches. However, for the EQR segment, permanent displacements of up to 11 inches were predicted. For all the Earthquake Scenarios and all the segments, imposed torsion due to Earthquake Scenarios is not significant.

- **Tie-Back Cables -** Tie-Back cables will be prestressed to 130 kips at the end of the construction. Dynamic analyses performed by AGS presents the tension forces in tie-backs during the Earthquake Scenarios. After completion of shaking, except for one Earthquake Scenario in Bluff Reach in which tie-backs loses the tension force, the post-earthquake tension forces remains above 20 kips for all Earthquake Scenarios.

Detailed summary of the conclusions and recommendations are as following:

North Reach:

Developed numerical model and dynamic analyses results for the design Earthquake Scenarios are presented in Appendix B.

- **Secant Pile Wall:** In longitudinal-direction, displacement of the secondary piles is less than 0.2 percent of the pile length. In transverse direction, maximum and permanent displacements are less than 0.75 percent of the pile length.
- **Tie-Back Cables**: Maximum axial force in the tie-back cables will be less than 64 kips which is significantly less than the capacity of the proposed cables.
- **Tunnel**: For the studied Scenarios, the worst Earthquake Scenario shows that tunnel moves 3 inches (permanently) in horizontal direction and settlement of the tunnel was estimated to be about 8 inches. Most of the displacement in tunnel lining manifests as rigid body movement.

EQR:

Developed numerical model and dynamic analyses results for the Earthquake Scenario A are presented in Appendix C.

- **Secant Pile Wall:** In longitudinal-direction, maximum and permanent displacement (relative to the bottom end of the pile) was estimated to be less than 0.65 percent of the pile length. However, the top portion of the secant pile wall which is continuous and includes both primary and secondary piles moves like a rigid body relative to the bottom portion which only includes secondary piles. In transverse direction, the top of the pile deforms less than 2 percent (maximum and residual) relative to the bottom end of the secondary pile.
- **Tie-Back Cables**: Maximum axial force in the tie-back cables during the earthquake excitation will be below 65 kips. It is significantly less than the capacity of the cables.

- **Tunnel:** Tunnel will displace 5 inches (permanently) in horizontal direction and settlement of the tunnel will be about 6 inches (worst case Scenario of three). Displacement is mostly in the form of rigid body motion.

Rubble Reach:

Developed numerical model and dynamic analyses results for the design Earthquake Scenarios are presented in Appendix D.

- **Secant Pile Wall:** Permanent displacements in both longitudinal and transverse directions are less than the maximum displacements (relative to the bottom end of the secondary pile). In longitudinal-direction, maximum and residual deformation of the wall is less than 0.5 percent and 0.25 percent, respectively. In transverse direction, maximum displacement at top of the pile is less than 1.5 percent. Permanent displacement of the wall is less than 0.2 percent in transverse direction.
- **Tie-Back Cables:** The numerical analyses show that the maximum axial forces along the tie-back cables do not exceed 66 kips. It is considerably less than the cable structural capacity.
- **Tunnel:** The numerical analyses indicate that the tunnel moves 5 inches (permanently) in horizontal direction and settlement of the tunnel is about 6 inches. Most of the displacement is in the form of rigid body movement.

Bluff Reach:

Developed numerical model and dynamic analyses results for the design Earthquake Scenarios are presented in Appendix E.

- **Secant Pile Wall:** Maximum and permanent displacements in longitudinal-direction for this segment are less than 0.1 percent. However, in transverse direction maximum and permanent displacements (relative to the bottom end of the secondary pile) are less than 0.6 percent.
- **Tie-Back Cables:** The numerical analyses indicate that for the design Earthquake Scenario A, the maximum axial tension force remains below 61 kips which is less than the allowable force for the proposed cables.

- **Tunnel:** Permanent movement of the tunnel in horizontal direction is estimated to be 4 inches and settlement of the tunnel is estimated to be about 3 inches. Tunnel lining displaces mostly as a rigid body.

South Reach:

Developed numerical model and dynamic analyses results for the design Earthquake Scenarios are presented in Appendix F.

- **Secant Pile Wall:** In longitudinal-direction, piles deform less than 0.3 percent. However, residual deformation is less than 0.1 percent. In the transverse direction the top of the secondary piles moves less than 1.5 percent relative to the bottom end.
- **Tie-Back Cables:** The numerical analyses indicate that, for the design Earthquake Scenario A, maximum axial tension force in the tie-back cables is less than 64 kips. Therefore, proposed cables can carry the load caused by the design earthquake.
- **Tunnel:** Tunnel moves 1 inch (permanently) in horizontal direction and settlement of the tunnel is about 2.5 inches. Similar to the other segments., the tunnel mostly moves as a rigid body.

Even though dynamic analyses results do not predict more than 0.5 percent deformation in the tunnel lining, we recommend that adequacy of the tunnel to withstand the displacement demands be evaluated.

7. Closure

This report has been prepared in accordance with generally accepted professional geotechnical engineering practice for the exclusive use of San Francisco Public Utilities Commission for the proposed South Ocean Beach Coastal Erosion and Wastewater Infrastructure Protection project in San Francisco, California. No other warranty, express or implied, is made.

It is the responsibility of the owner or its representative to ensure that the applicable provisions of this report are incorporated into the plans and specifications and that the necessary steps are taken to see that the contractor carry out such provisions.

Respectfully submitted,

AGS, Inc.

Kamran Ghiassi, Ph.D.	Bahram Khamenehpour, Ph.D.
Geotechnical Engineer, 2792	Geotechnical Engineer, 2104

8. References

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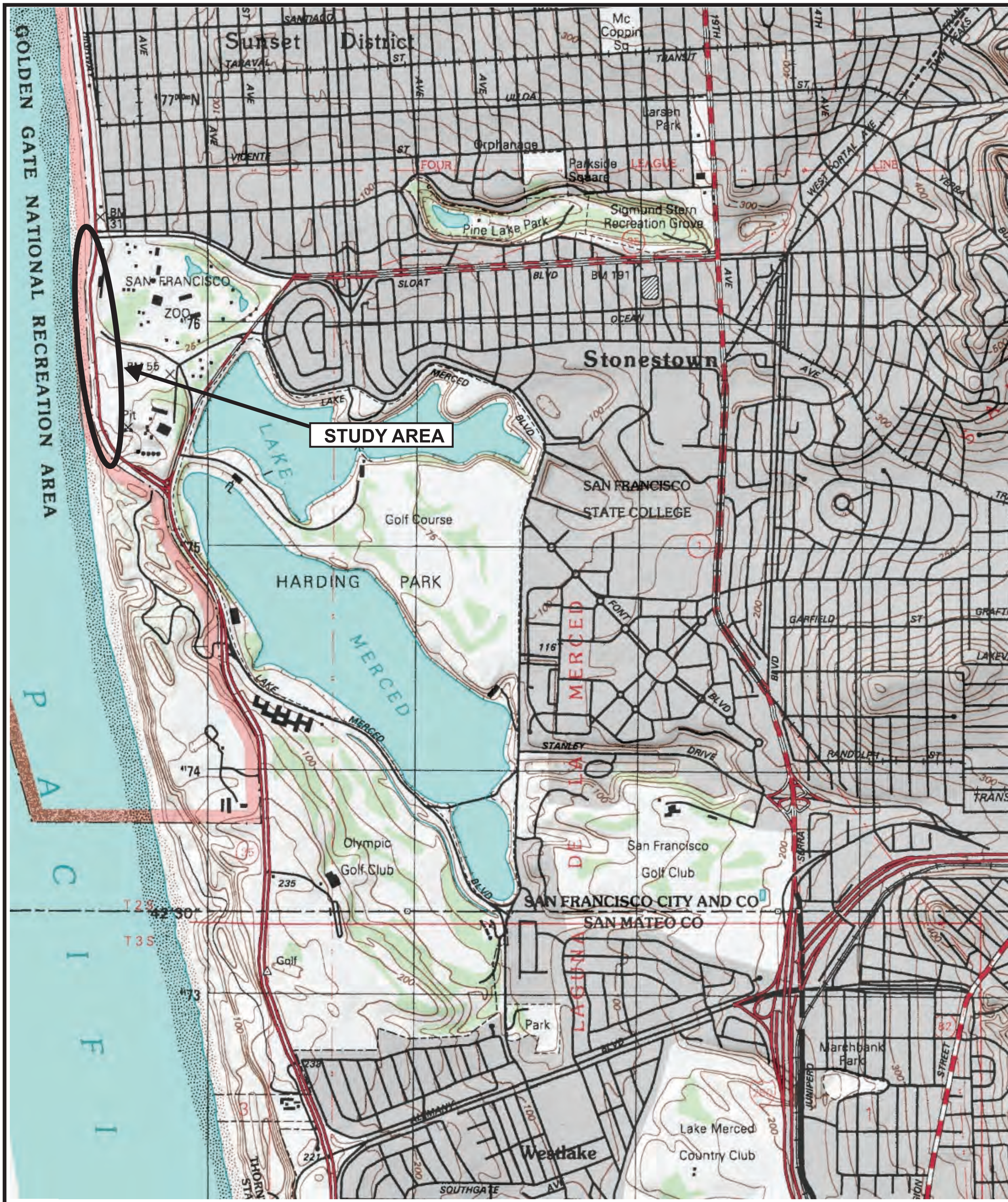
Appendices

Appendix A

BACKGROUND PLATES



Source: USGS, 7.5 Minute Topographic Map of the San Francisco South (2018) Quadrangle.

N

1000 0 1000 2000 ft

Scale

SITE VICINITY MAP

SOUTH OCEAN BEACH COASTAL EROSION AND WW INFRASTRUCTURE PROTECTION PROJECT SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE A-1

LEGEND

N

SCALE

0 150 300 feet

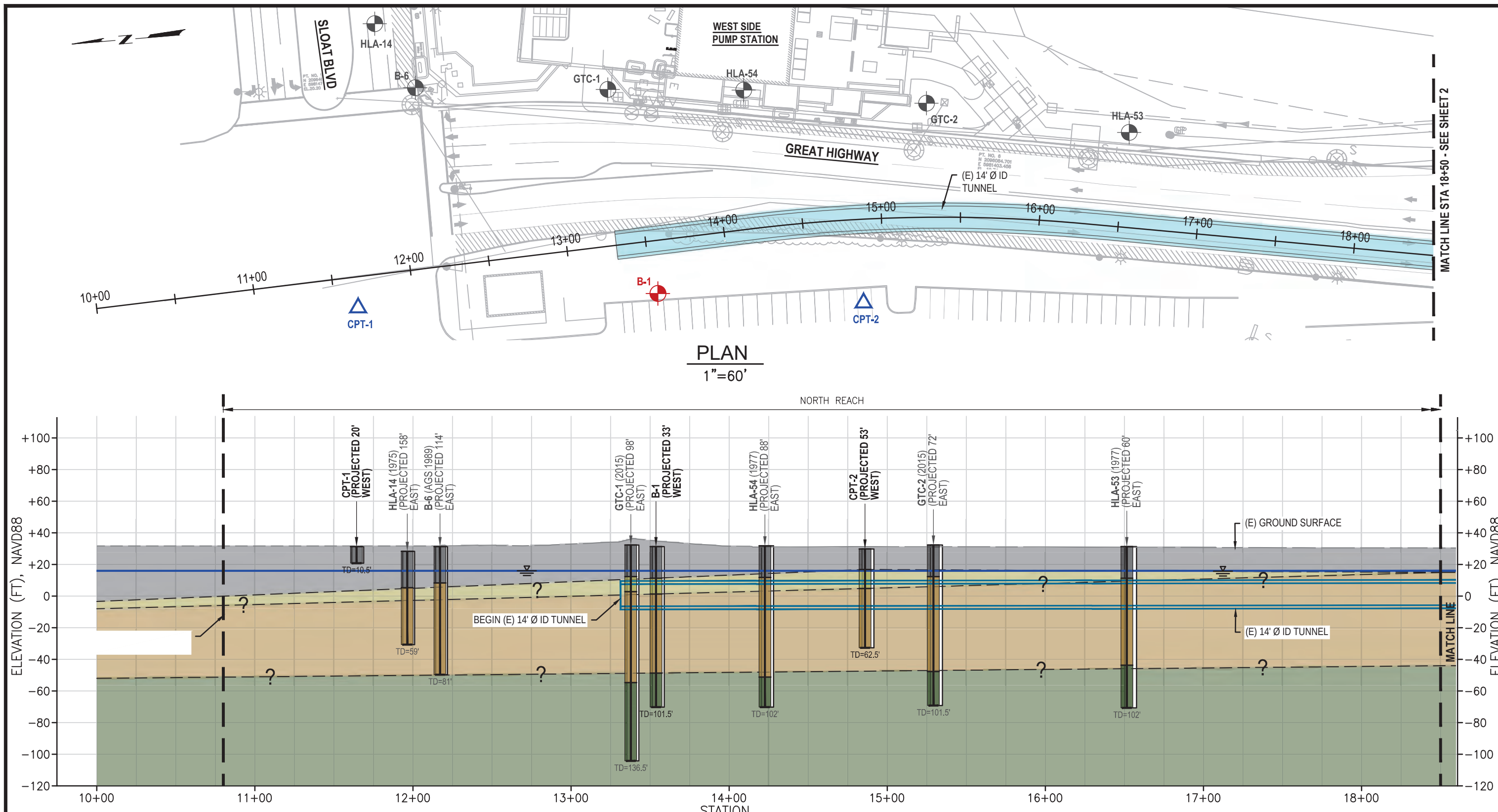
B-6/EB-6 Geo/Env Boring Location
CPT-13 CPT Location
PH-2B Potholing Location
MW-3 Monitoring Well Location

TP-1 Test Pit Location
ET-1 Environmental Direct Push Location

ML-4 Approximate location of Seismic Survey Array

Approximate location of Southwest Ocean Outfall (SWOO) pipeline

SITE LOCATION PLAN & EXPLORATION PROGRAM MAP SOUTH OCEAN BEACH COASTAL EROSION AND WW INFRASTRUCTURE PROTECTION PROJECT SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JULY 2020	PLATE A-2



LEGEND

- ARTIFICIAL FILL
- DUNE SAND
- COLMA FORMATION
- MERCED FORMATION
- LMT ALIGNMENT
- ESTIMATED DESIGN GROUNDWATER
- B-1 TD=TOTAL DEPTH (FT)
- 0+00 LMT ALIGNMENT
- SOIL BORING (AGS 2019)
- CONE PENETROMETER TEST SOUNDING (AGS 2019)
- PREVIOUS SOIL BORINGS
- POTHOLES (AGS 2019)

PROFILE

HOR.: 1"=60'
VERT.: 1"=60'

NOTES

1. SFCD = NAV88−11.35 FEET.
2. ORIGINAL PLOT SIZE OF THIS SHEET IS 11"X17".

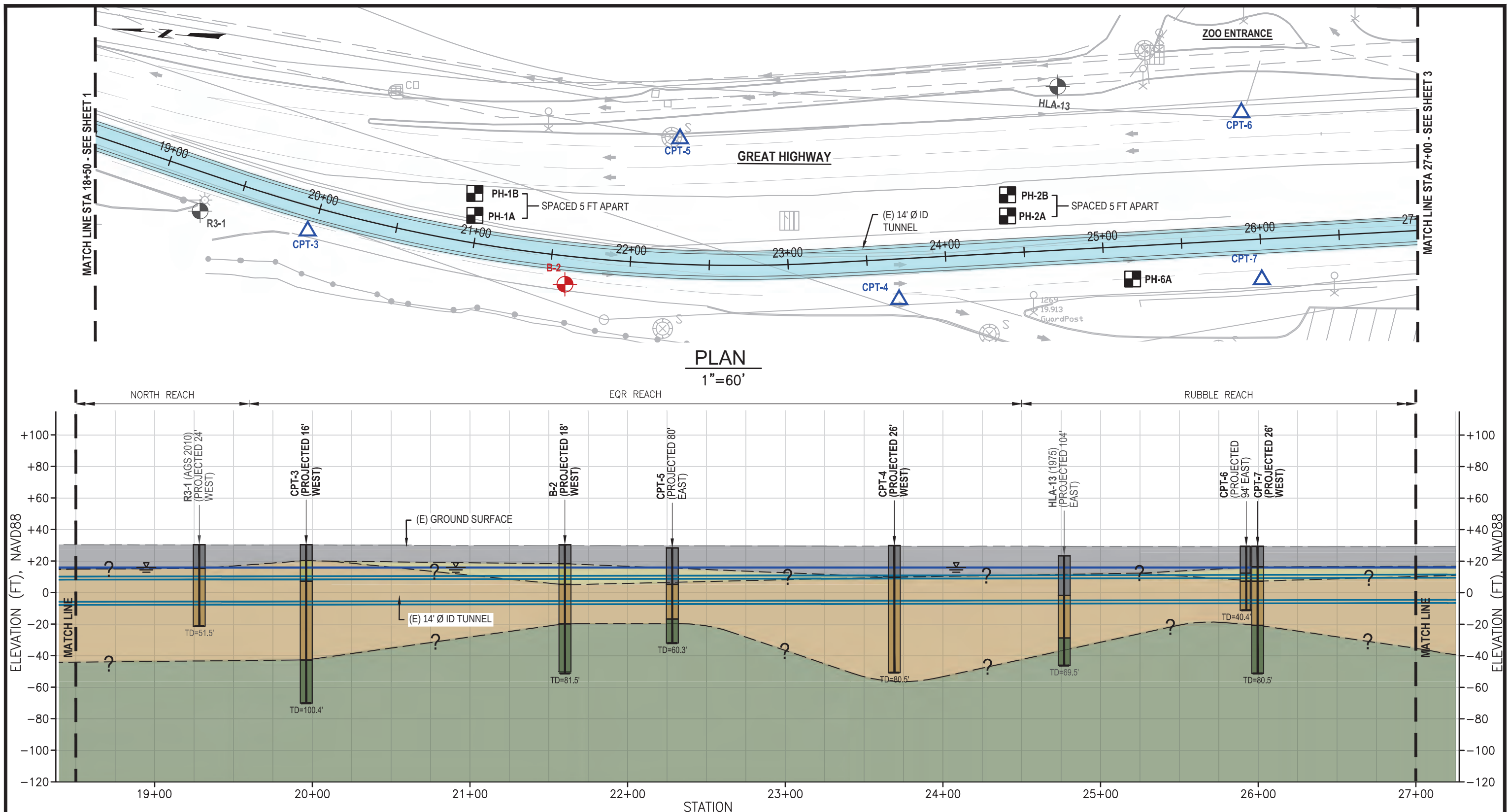
DISCLAMER: THE SUBSURFACE CONDITIONS WERE INTERPOLATED. THE INTERPOLATION AND EXTRAPOLATION OF THE SUBSURFACE SOIL AND GROUNDWATER CONDITIONS ARE BASED ON WIDELY SPACED BORINGS. VARIATION IN THE ACTUAL CONDITIONS FROM THOSE DEPICTED SHOULD BE ANTICIPATED. NEITHER BORING LOGS NOR THE GENERALIZED SOIL PROFILES ARE STAND-ALONE DOCUMENTS, AND THEY SHOULD ONLY BE USED WITHIN THE FULL CONTEXT OF THE SOILS REPORT. THIS PROFILE WAS PREPARED FOR THE SOLE PURPOSE OF DEVELOPING GEOTECHNICAL RECOMMENDATIONS. IT IS NOT INTENDED FOR USE IN ESTIMATING QUANTITY OF VARIOUS SOIL TYPES.

GENERALIZED PLAN AND PROFILE ALONG LMT ALIGNMENT

SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO: AGS-18-003	DATE: JUNE 2021	PLATE A-3A



LEGEND

- ARTIFICIAL FILL
- DUNE SAND
- COLMA FORMATION
- MERCED FORMATION
- LMT ALIGNMENT
- ESTIMATED DESIGN GROUNDWATER
- B-1 — TD=TOTAL DEPTH (FT)
- 0+00 LMT ALIGNMENT
- SOIL BORING (AGS 2019)
- CONE PENETROMETER TEST SOUNDING (AGS 2019)
- PREVIOUS SOIL BORINGS
- POTHOLES (AGS 2019)

PROFILE

HOR.: 1"=60'
VERT.: 1"=60'

NOTES

1. SFCD = NAV88−11.35 FEET.
2. ORIGINAL PLOT SIZE OF THIS SHEET IS 11"X17".

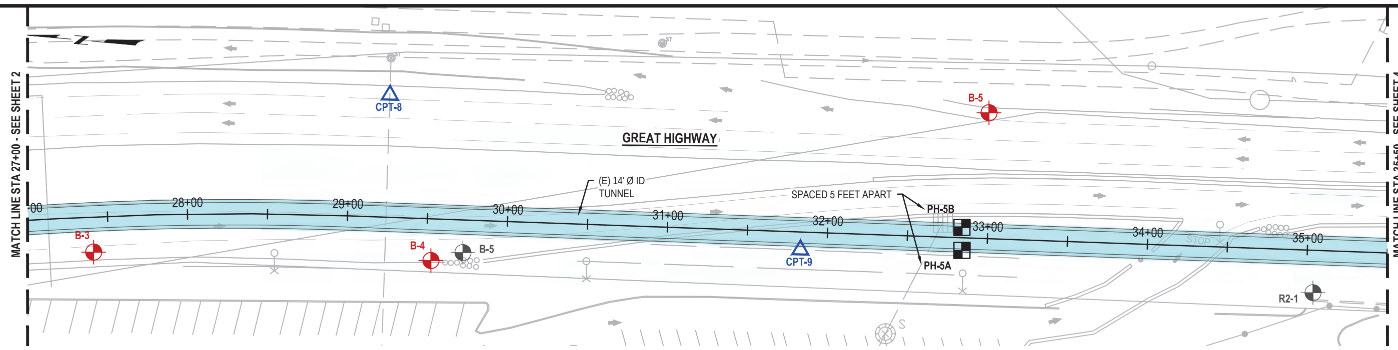
DISCLAMER: THE SUBSURFACE CONDITIONS WERE INTERPOLATED. THE INTERPOLATION AND EXTRAPOLATION OF THE SUBSURFACE SOIL AND GROUNDWATER CONDITIONS ARE BASED ON WIDELY SPACED BORINGS. VARIATION IN THE ACTUAL CONDITIONS FROM THOSE DEPICTED SHOULD BE ANTICIPATED. NEITHER BORING LOGS NOR THE GENERALIZED SOIL PROFILES ARE STAND-ALONE DOCUMENTS, AND THEY SHOULD ONLY BE USED WITHIN THE FULL CONTEXT OF THE SOILS REPORT. THIS PROFILE WAS PREPARED FOR THE SOLE PURPOSE OF DEVELOPING GEOTECHNICAL RECOMMENDATIONS. IT IS NOT INTENDED FOR USE IN ESTIMATING QUANTITY OF VARIOUS SOIL TYPES.

GENERALIZED PLAN AND PROFILE ALONG LMT ALIGNMENT

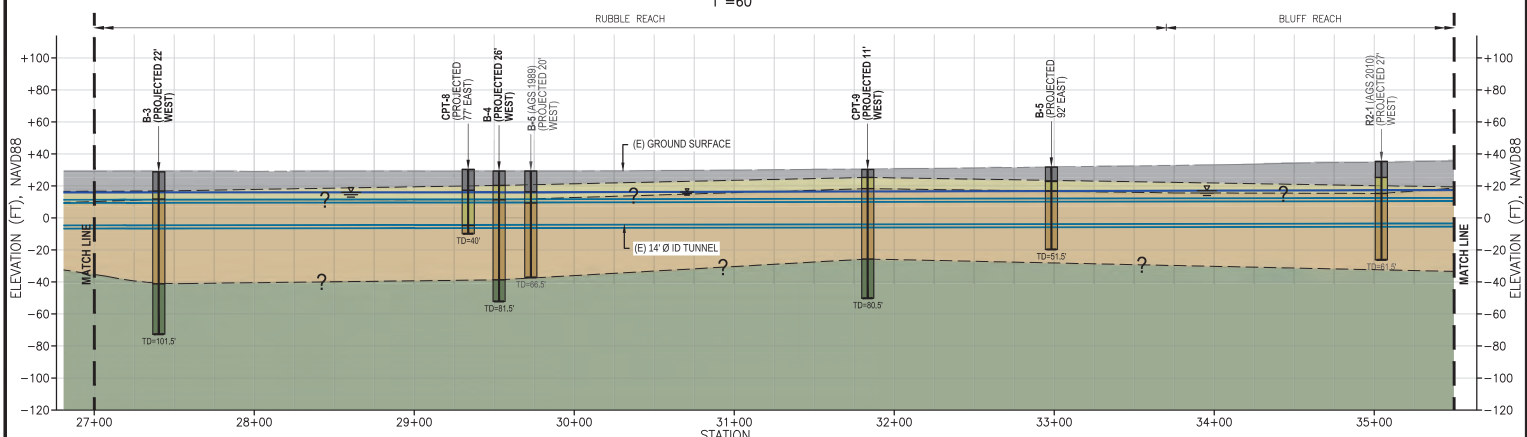
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO: AGS-18-003	DATE: JUNE 2021	PLATE A-3B



PLAN
1"=60'



LEGEND

- ARTIFICIAL FILL
- DUNE SAND
- COLMA FORMATION
- MERCED FORMATION
- LMT ALIGNMENT
- ESTIMATED DESIGN GROUNDWATER
- B-1 TD=TOTAL DEPTH (FT)
- 0+00 LMT ALIGNMENT
- SOIL BORING (AGS 2019)
- CONE PENETROMETER TEST SOUNDING (AGS 2019)
- PREVIOUS SOIL BORINGS
- POTHOLES (AGS 2019)

PROFILE

HOR.: 1"=60'
VERT.: 1"=60'

NOTES

1. SFCD = NAV88−11.35 FEET.
2. ORIGINAL PLOT SIZE OF THIS SHEET IS 11"X17".

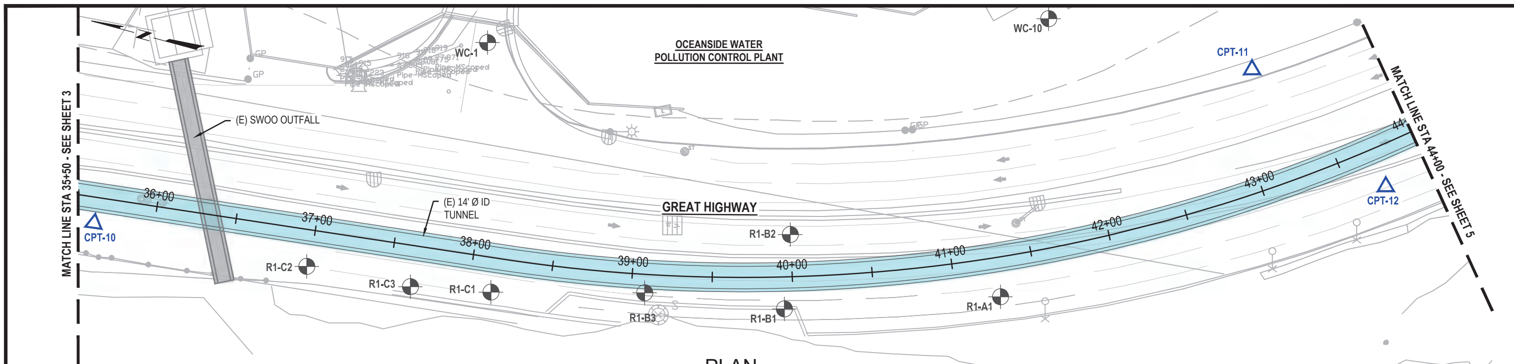
DISCLAMER: THE SUBSURFACE CONDITIONS WERE INTERPOLATED. THE INTERPOLATION AND EXTRAPOLATION OF THE SUBSURFACE SOIL AND GROUNDWATER CONDITIONS ARE BASED ON WIDELY SPACED BORINGS. VARIATION IN THE ACTUAL CONDITIONS FROM THOSE DEPICTED SHOULD BE ANTICIPATED. NEITHER BORING LOGS NOR THE GENERALIZED SOIL PROFILES ARE STAND-ALONE DOCUMENTS, AND THEY SHOULD ONLY BE USED WITHIN THE FULL CONTEXT OF THE SOILS REPORT. THIS PROFILE WAS PREPARED FOR THE SOLE PURPOSE OF DEVELOPING GEOTECHNICAL RECOMMENDATIONS. IT IS NOT INTENDED FOR USE IN ESTIMATING QUANTITY OF VARIOUS SOIL TYPES.

GENERALIZED PLAN AND PROFILE ALONG LMT ALIGNMENT

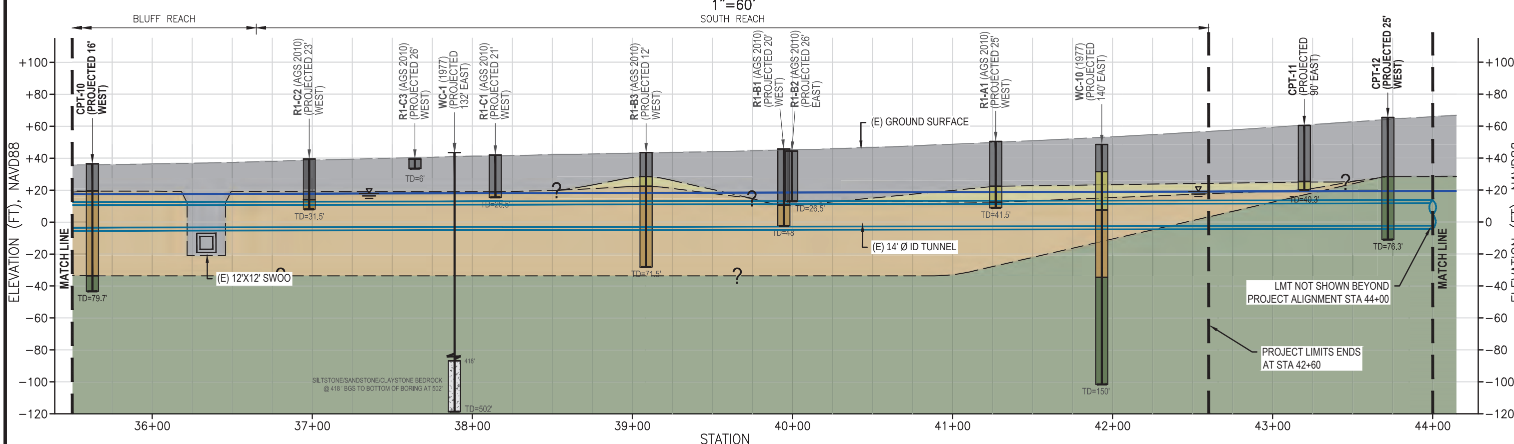
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO: AGS-18-003	DATE: JUNE 2021	PLATE A-3C



PLAN
1"=60'



LEGEND

- ARTIFICIAL FILL
- DUNE SAND
- COLMA FORMATION
- MERCED FORMATION
- LMT ALIGNMENT
- ESTIMATED DESIGN GROUNDWATER
- B-1 TD=TOTAL DEPTH (FT)
- 0+00 LMT ALIGNMENT
- SOIL BORING (AGS 2019)
- CONE PENETROMETER TEST SOUNDING (AGS 2019)
- PREVIOUS SOIL BORINGS
- POTHOLES (AGS 2019)

PROFILE
HOR.: 1"=60'
VERT.: 1"=60'

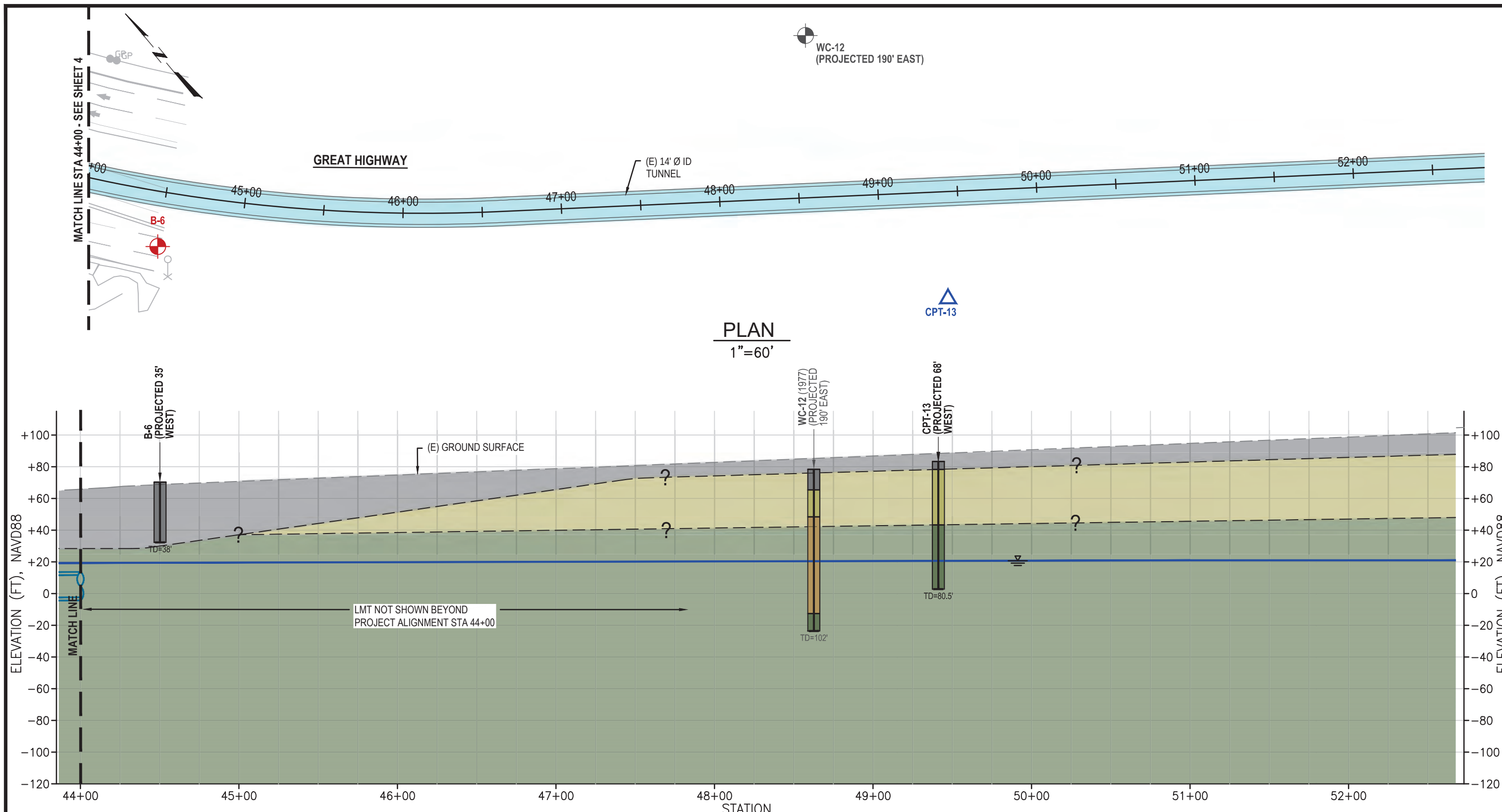
NOTES
1. SFCD = NAV88−11.35 FEET.
2. ORIGINAL PLOT SIZE OF THIS SHEET IS 11"X17".

DISCLAMER: THE SUBSURFACE CONDITIONS WERE INTERPOLATED. THE INTERPOLATION AND EXTRAPOLATION OF THE SUBSURFACE SOIL AND GROUNDWATER CONDITIONS ARE BASED ON WIDELY SPACED BORINGS. VARIATION IN THE ACTUAL CONDITIONS FROM THOSE DEPICTED SHOULD BE ANTICIPATED. NEITHER BORING LOGS NOR THE GENERALIZED SOIL PROFILES ARE STAND-ALONE DOCUMENTS, AND THEY SHOULD ONLY BE USED WITHIN THE FULL CONTEXT OF THE SOILS REPORT. THIS PROFILE WAS PREPARED FOR THE SOLE PURPOSE OF DEVELOPING GEOTECHNICAL RECOMMENDATIONS. IT IS NOT INTENDED FOR USE IN ESTIMATING QUANTITY OF VARIOUS SOIL TYPES.

GENERALIZED PLAN AND PROFILE ALONG LMT ALIGNMENT
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO: AGS-18-003	DATE: JUNE 2021	PLATE A-3D



LEGEND

- ARTIFICIAL FILL
- DUNE SAND
- COLMA FORMATION
- MERCED FORMATION
- LMT ALIGNMENT
- ESTIMATED DESIGN GROUNDWATER
- B-1 TD=TOTAL DEPTH (FT)
- 0+00 LMT ALIGNMENT
- SOIL BORING (AGS 2019)
- CONE PENETROMETER TEST SOUNDING (AGS 2019)
- PREVIOUS SOIL BORINGS
- POTHOLES (AGS 2019)

PROFILE

HOR.: 1"=60'
VERT.: 1"=60'

NOTES

1. SFCD = NAV88−11.35 FEET.
2. ORIGINAL PLOT SIZE OF THIS SHEET IS 11"X17".

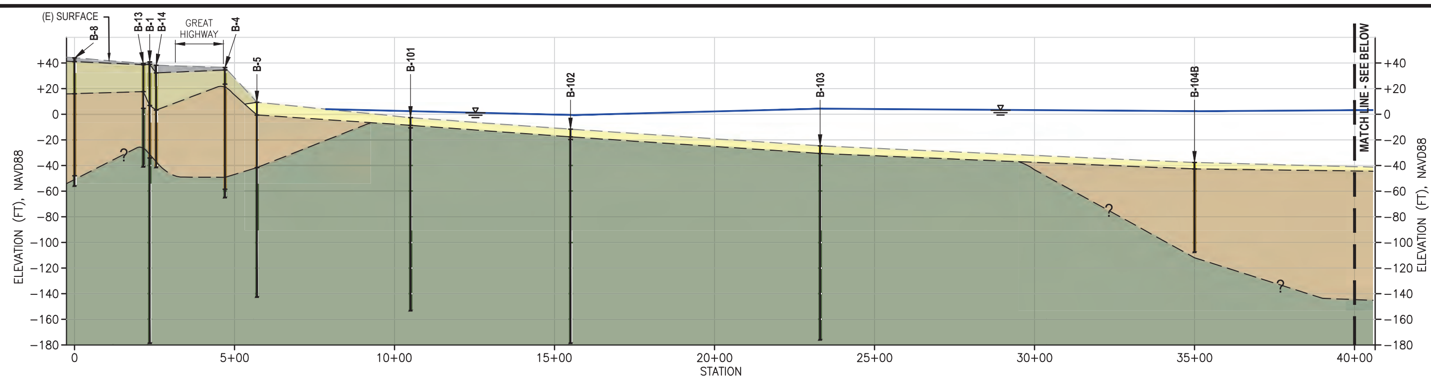
DISCLAMER: THE SUBSURFACE CONDITIONS WERE INTERPOLATED. THE INTERPOLATION AND EXTRAPOLATION OF THE SUBSURFACE SOIL AND GROUNDWATER CONDITIONS ARE BASED ON WIDELY SPACED BORINGS. VARIATION IN THE ACTUAL CONDITIONS FROM THOSE DEPICTED SHOULD BE ANTICIPATED. NEITHER BORING LOGS NOR THE GENERALIZED SOIL PROFILES ARE STAND-ALONE DOCUMENTS, AND THEY SHOULD ONLY BE USED WITHIN THE FULL CONTEXT OF THE SOILS REPORT. THIS PROFILE WAS PREPARED FOR THE SOLE PURPOSE OF DEVELOPING GEOTECHNICAL RECOMMENDATIONS. IT IS NOT INTENDED FOR USE IN ESTIMATING QUANTITY OF VARIOUS SOIL TYPES.

GENERALIZED PLAN AND PROFILE ALONG LMT ALIGNMENT

SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT
SAN FRANCISCO, CALIFORNIA

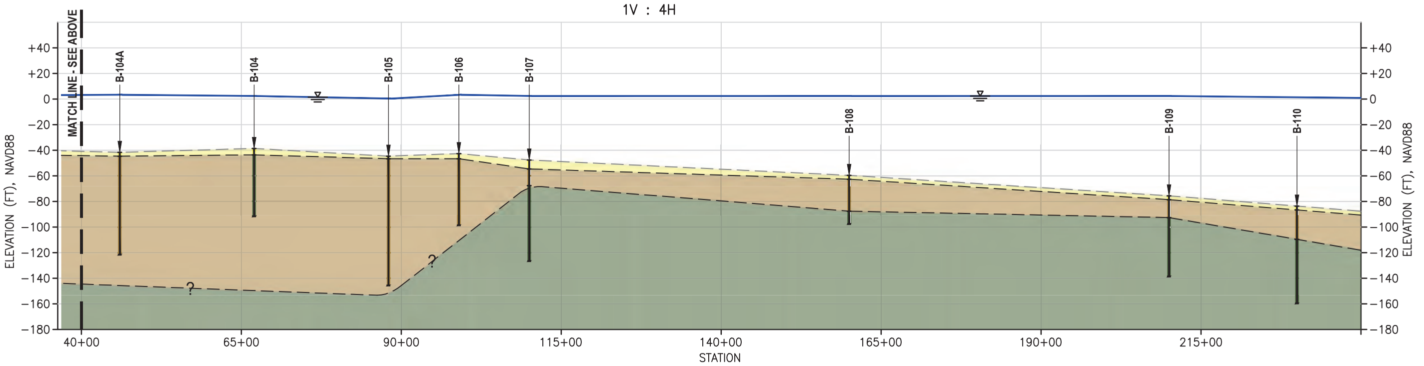
AGS

JOB NO: AGS-18-003	DATE: JUNE 2021	PLATE A-3E



PROFILE

HOR.: 1"=300'
VERT.: 1"=75'
1V : 4H



PROFILE

HOR.: 1"=1500'
VERT.: 1"=75'
1V : 20H

LEGEND

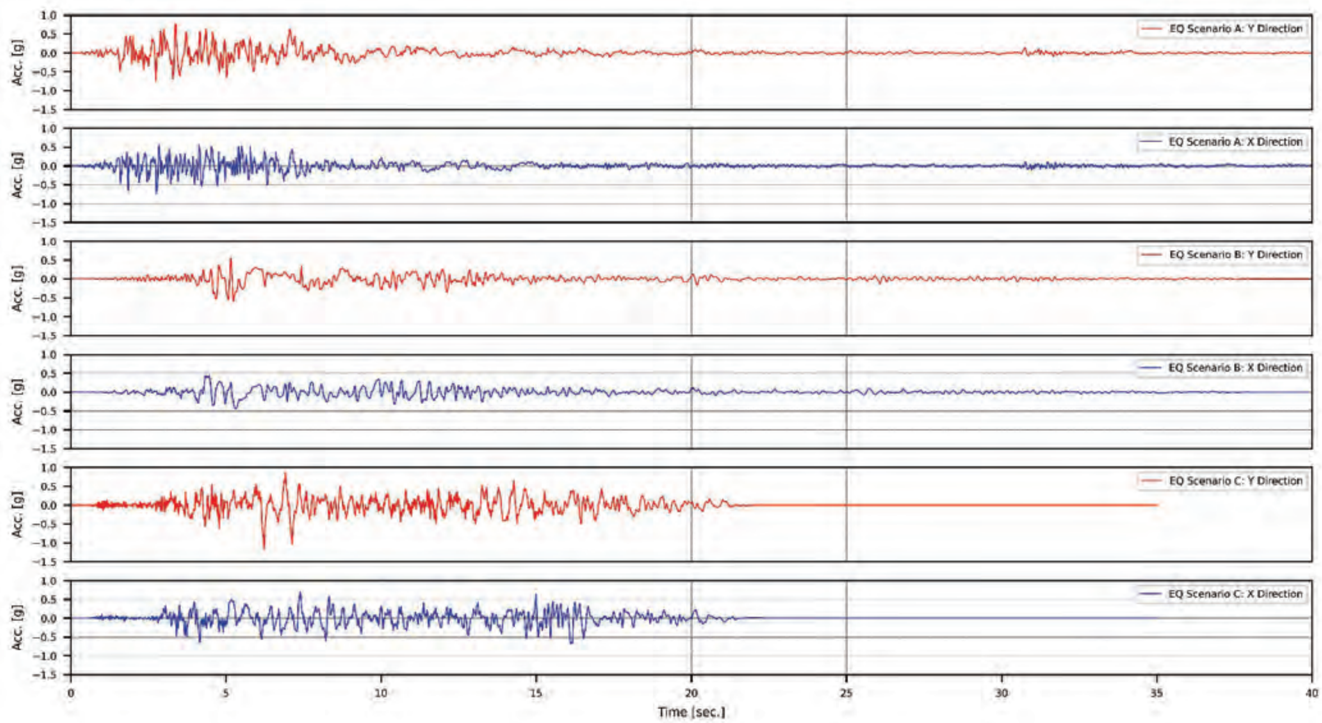
- ARTIFICIAL FILL
- DUNE SAND
- COLMA FORMATION
- MERCED FORMATION
- LMT ALIGNMENT
- ESTIMATED DESIGN GROUNDWATER
- B-1 TD=TOTAL DEPTH (FT)

DISCLAMER: THE SUBSURFACE CONDITIONS WERE INTERPOLATED. THE INTERPOLATION AND EXTRAPOLATION OF THE SUBSURFACE SOIL AND GROUNDWATER CONDITIONS ARE BASED ON WIDELY SPACED BORINGS. VARIATION IN THE ACTUAL CONDITIONS FROM THOSE DEPICTED SHOULD BE ANTICIPATED. NEITHER BORING LOGS NOR THE GENERALIZED SOIL PROFILES ARE STAND-ALONE DOCUMENTS, AND THEY SHOULD ONLY BE USED WITHIN THE FULL CONTEXT OF THE SOILS REPORT. THIS PROFILE WAS PREPARED FOR THE SOLE PURPOSE OF DEVELOPING GEOTECHNICAL RECOMMENDATIONS. IT IS NOT INTENDED FOR USE IN ESTIMATING QUANTITY OF VARIOUS SOIL TYPES.

NOTES

1. NAV88 = SFCD+11.35 FEET
2. PROFILE STATIONING BEGINS AT B-8.
3. SUBSURFACE SHOWN IS BASED ON WOODWARD AND CLYDE BORINGS PRIOR TO CONSTRUCTION OF SWOO.

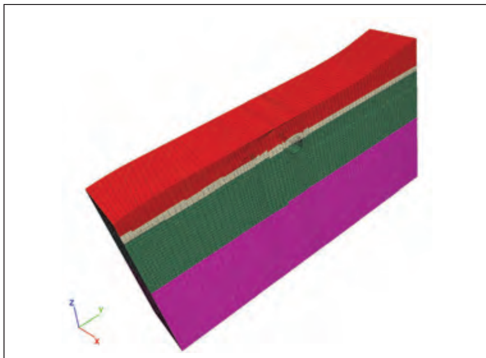
GENERALIZED SUBSURFACE PROFILE - SWOO SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION PROJECT SAN FRANCISCO, CALIFORNIA		AGS
JOB NO: AGS-18-003	DATE: MAY 2021	PLATE A-4



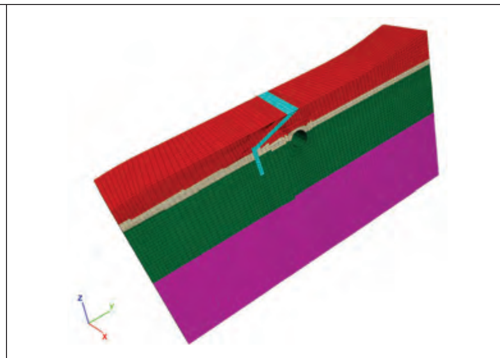
HORIZONTAL ACCELERATION TIME HISTORIES APPLIED AT THE BASE OF THE MODELS SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE A-5

Appendix B

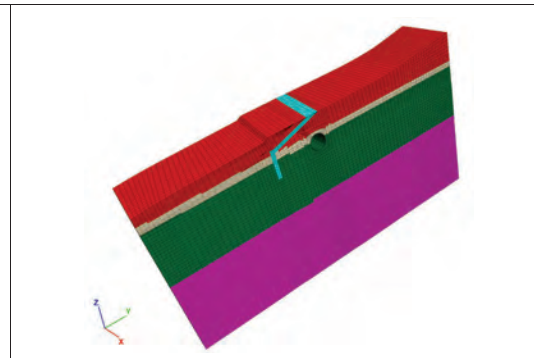
FLAC3D NUMERICAL SSSI SIMULATION RESULTS FOR NORTH REACH



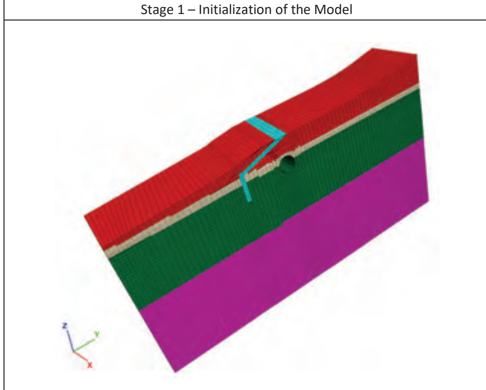
Stage 1 – Initialization of the Model



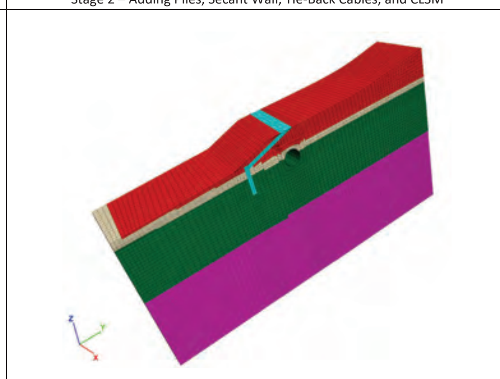
Stage 2 – Adding Piles, Secant Wall, Tie-Back Cables, and CLSM



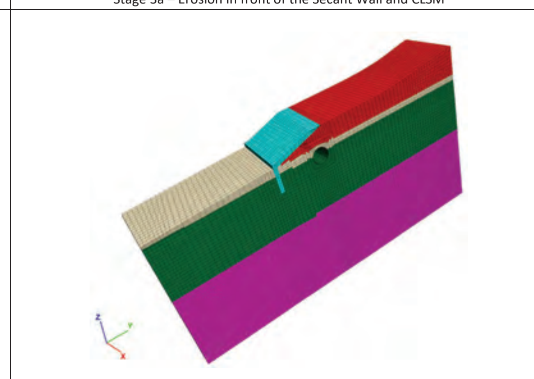
Stage 3a – Erosion in front of the Secant Wall and CLSM



Stage 3b – Erosion in front of the Secant Wall and CLSM



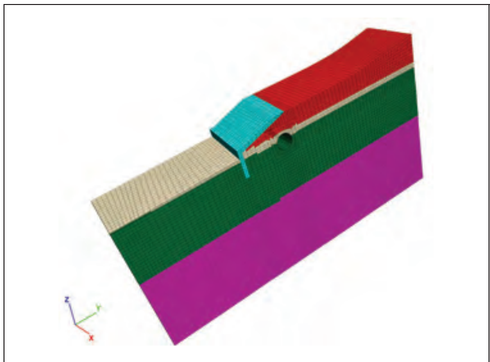
Stage 3c – Erosion in front of the Secant Wall and CLSM



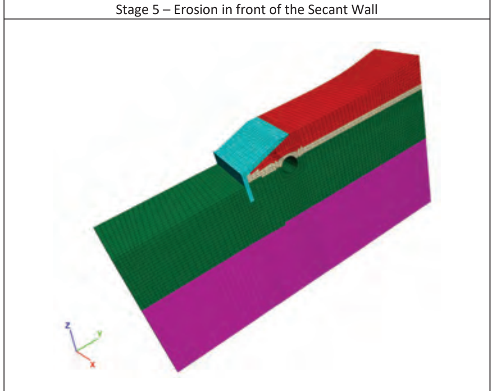
Stage 4 – Erosion in front of the Secant Wall

CLSM and Wall	Liquefiable Layer
	Colma Formation
Fill	Merced Formation

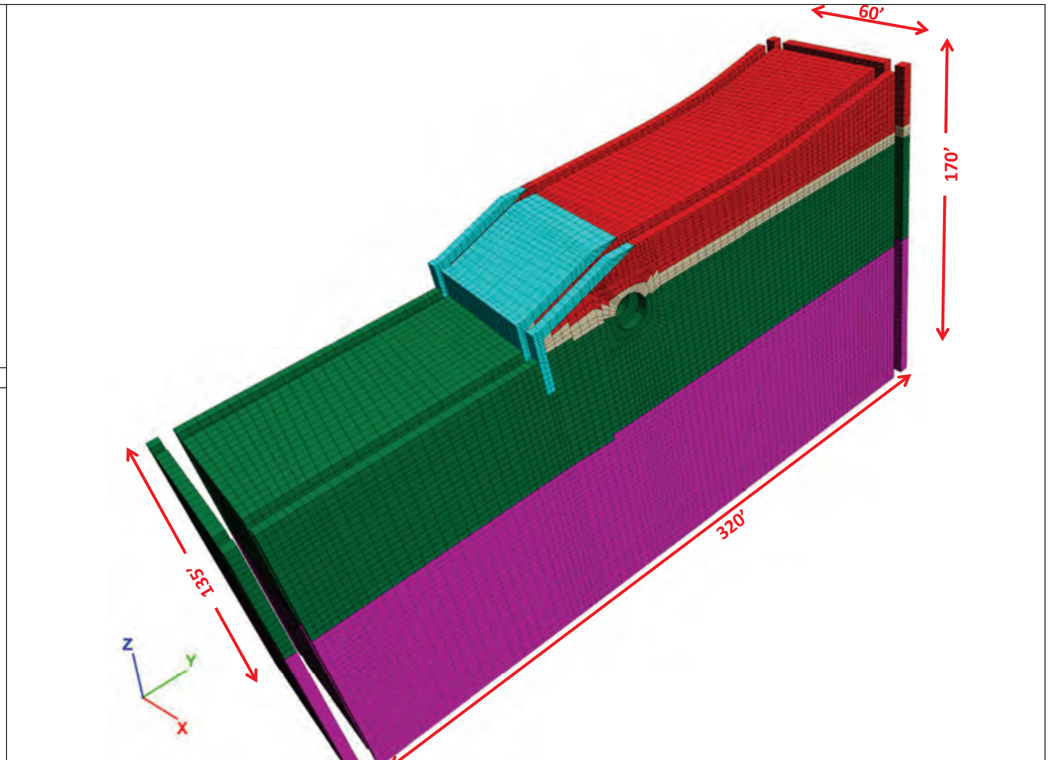
EQR MESH – DIFFERENT STAGES OF THE NUMERICAL SIMULATION SOUTH OCEAN BEACH C OASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-1 a



Stage 5 – Erosion in front of the Secant Wall



Stage 6 – Erosion in front of the Secant Wall



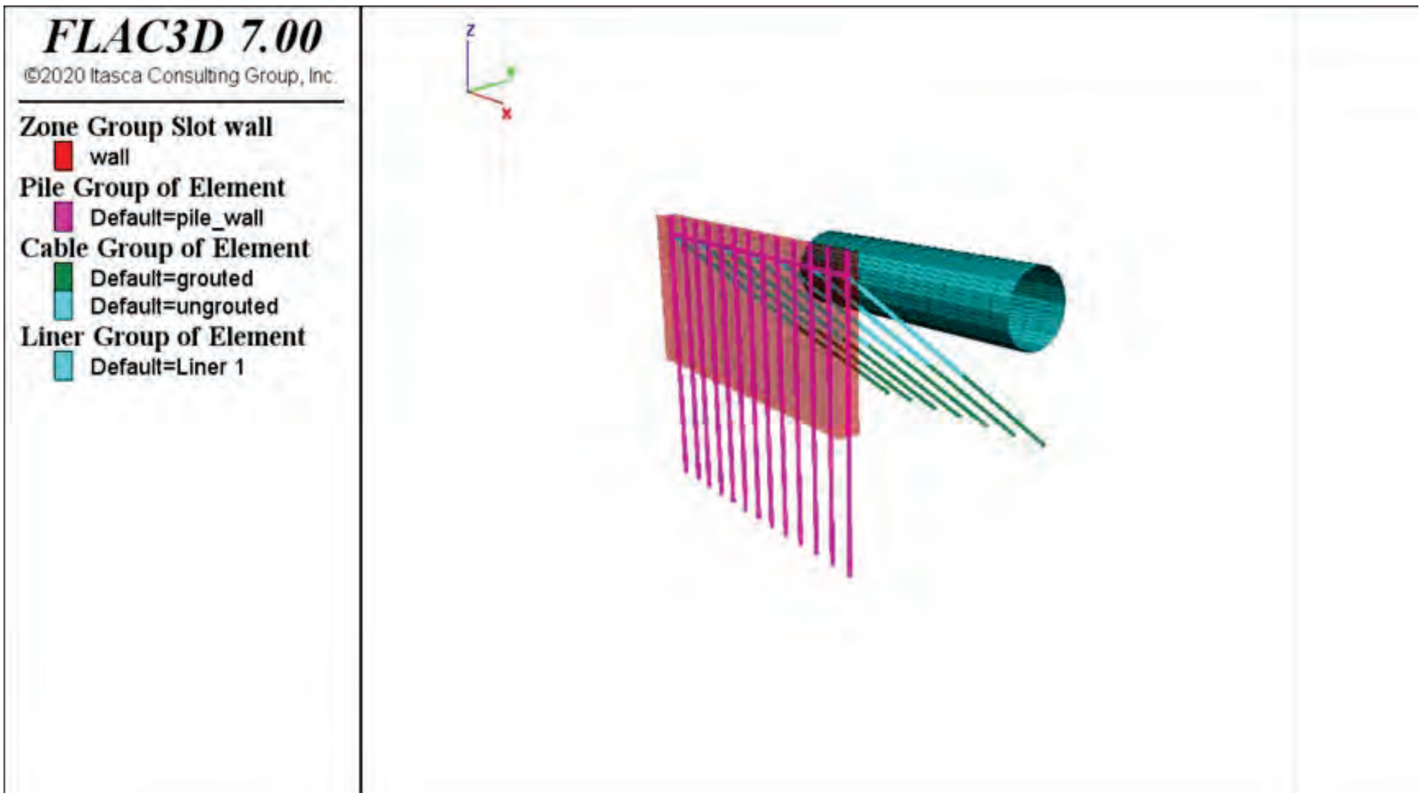
Mesh Used to Seismic Analysis – Includes Free Field Boundary Conditions

CLSM and Wall	Liquefiable Layer
Fill	Colma Formation
	Merced Formation

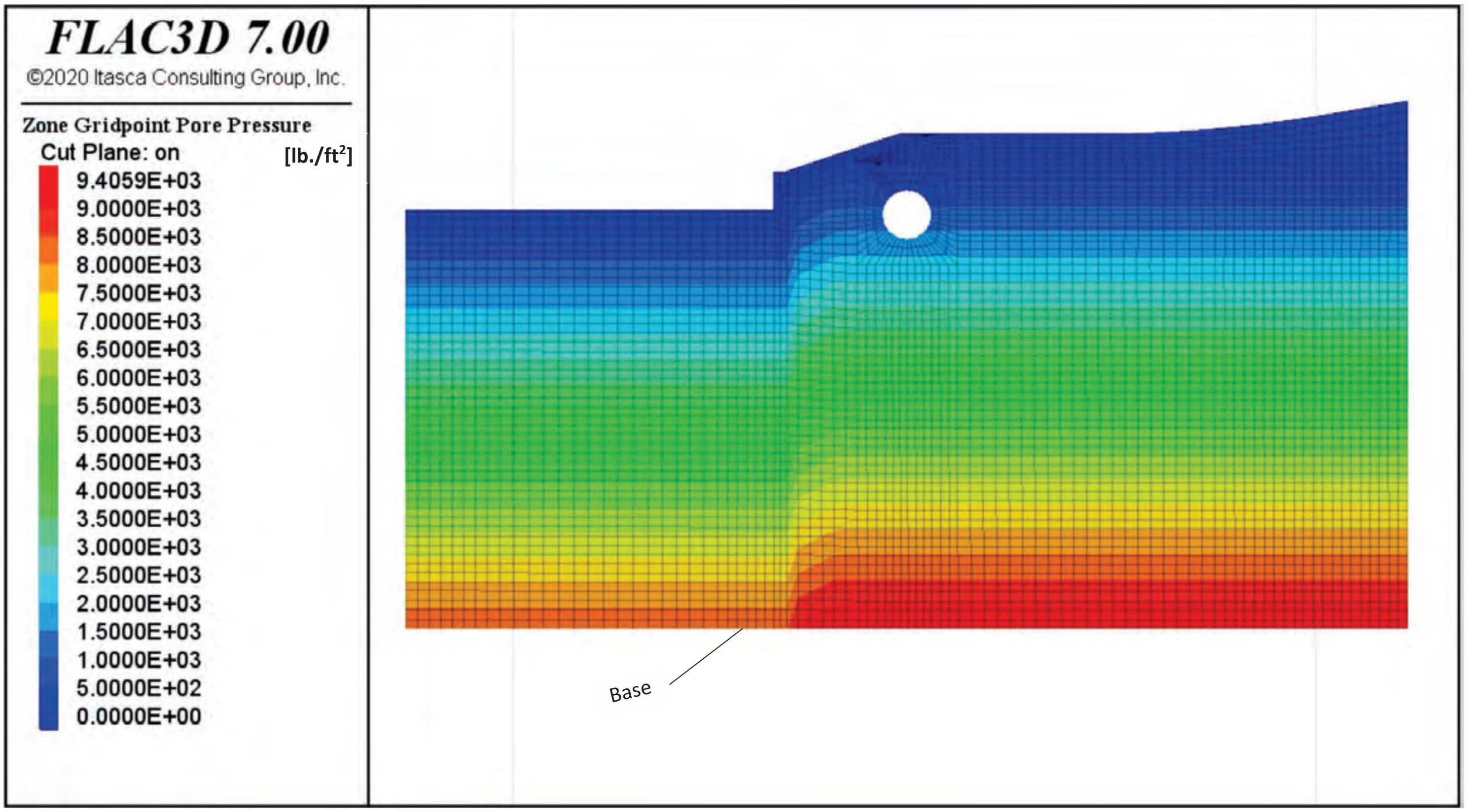
EQR MESH – DIFFERENT STAGES OF THE NUMERICAL SIMULATION
SOUTH OCEAN BEACH C OASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO, CALIFORNIA

AGS

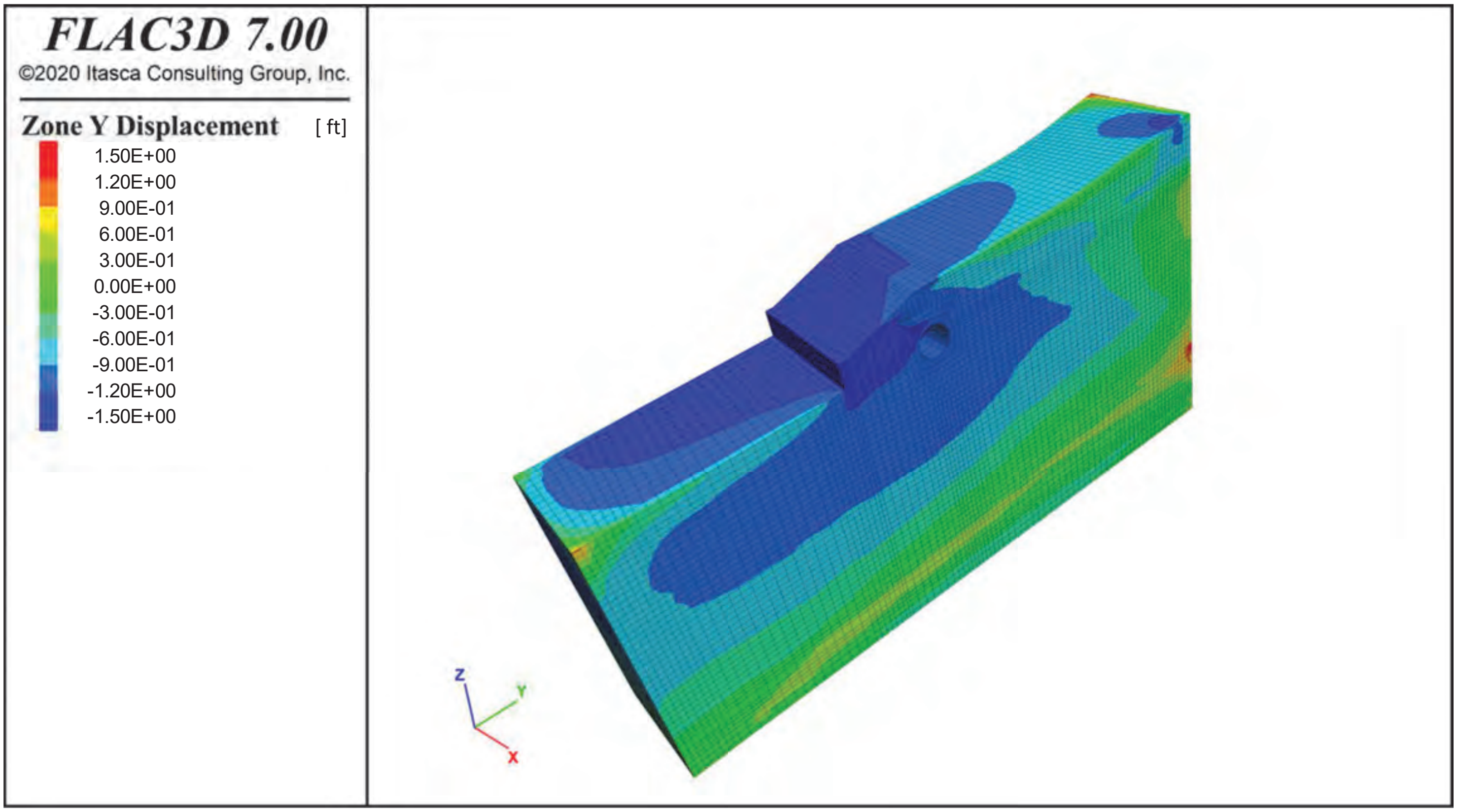
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-1 b



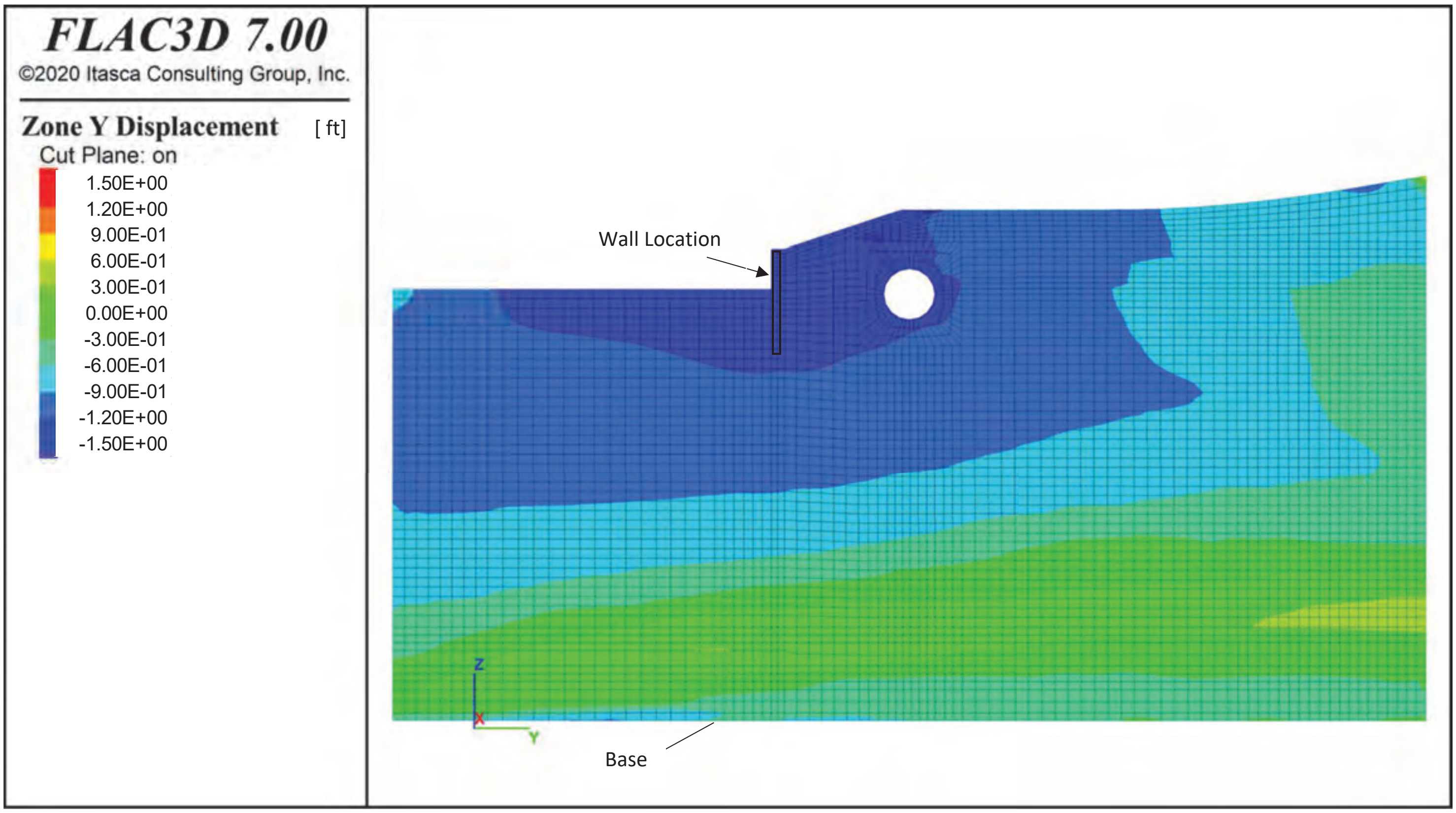
STRUCTURAL ELEMENTS OF THE MODEL **(EARTHQUAKE SCENARIO A - NORTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS
JOB NO. AGS-18-003 · DATE: JUNE 2021	PLATE B-2



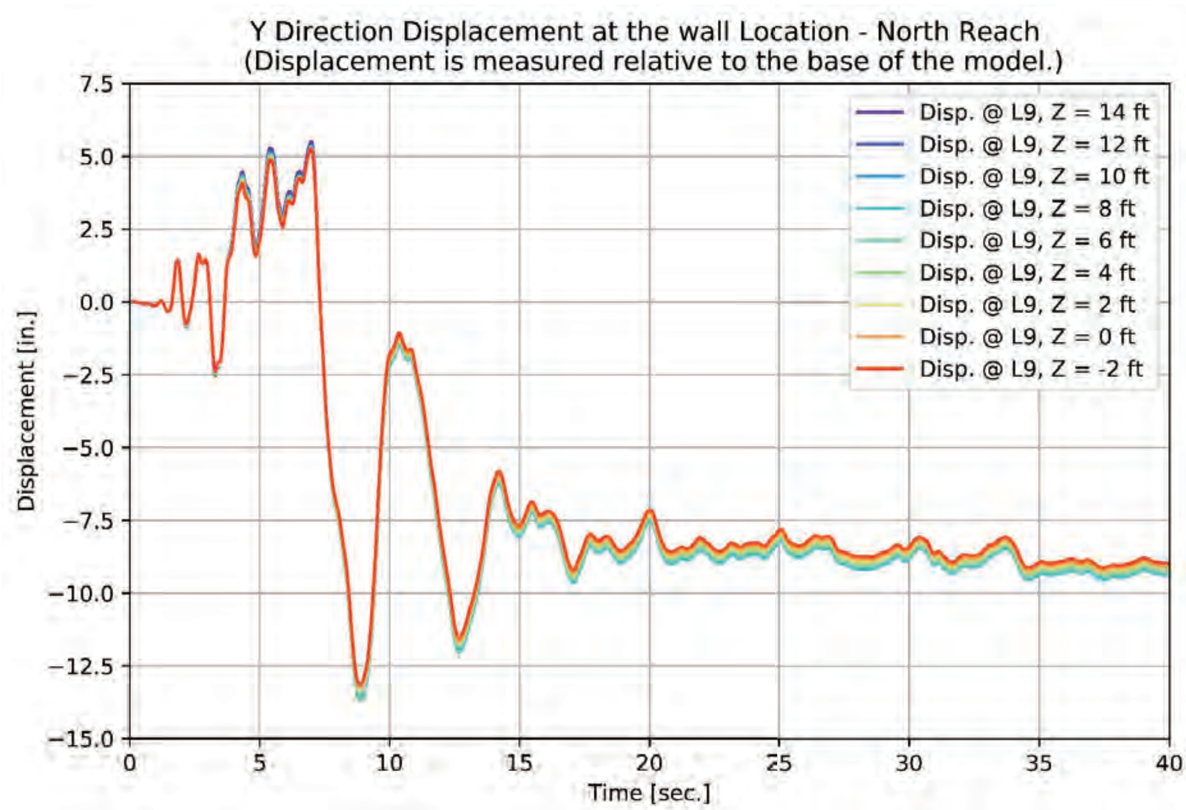
HYDROSTATIC PORE-WATER PRESSURE PRIOR TO SHAKING (NORTH REACH) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-3



RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO A - NORTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-4

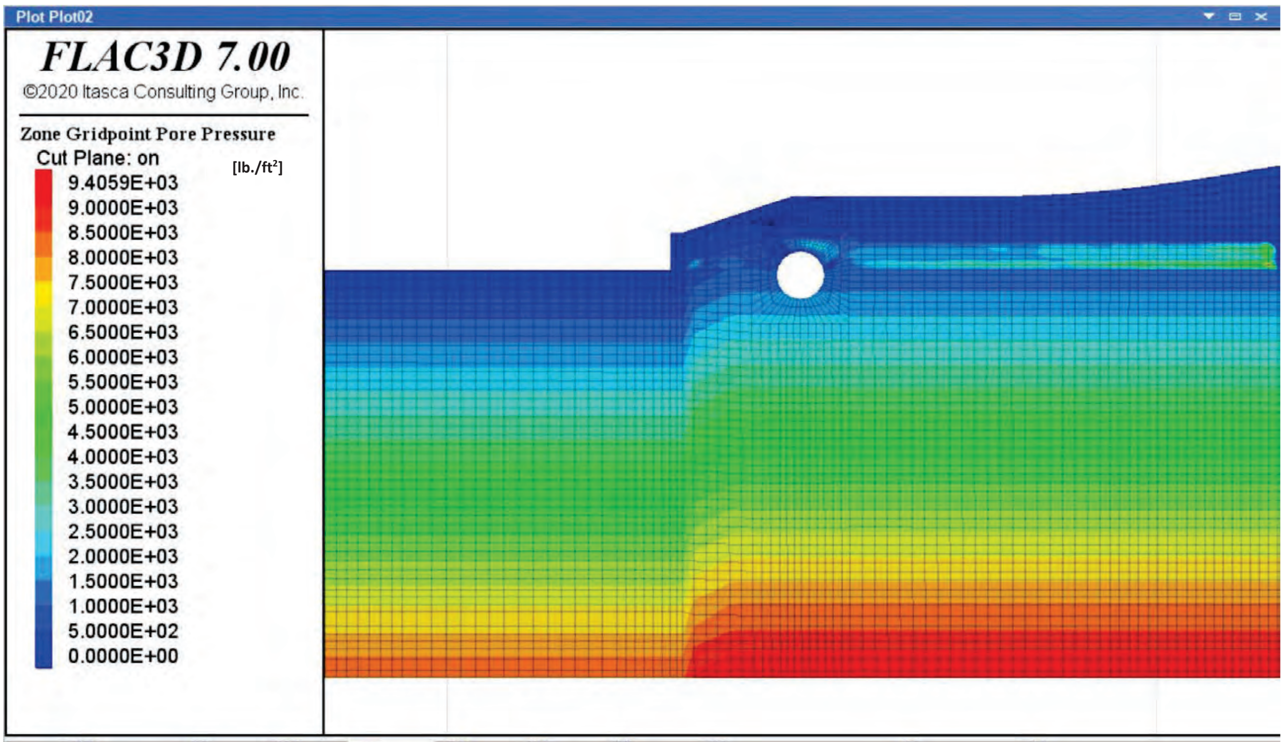


RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 40 SEC. - EARTHQUAKE SCENARIO A - NORTH REACH) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-5

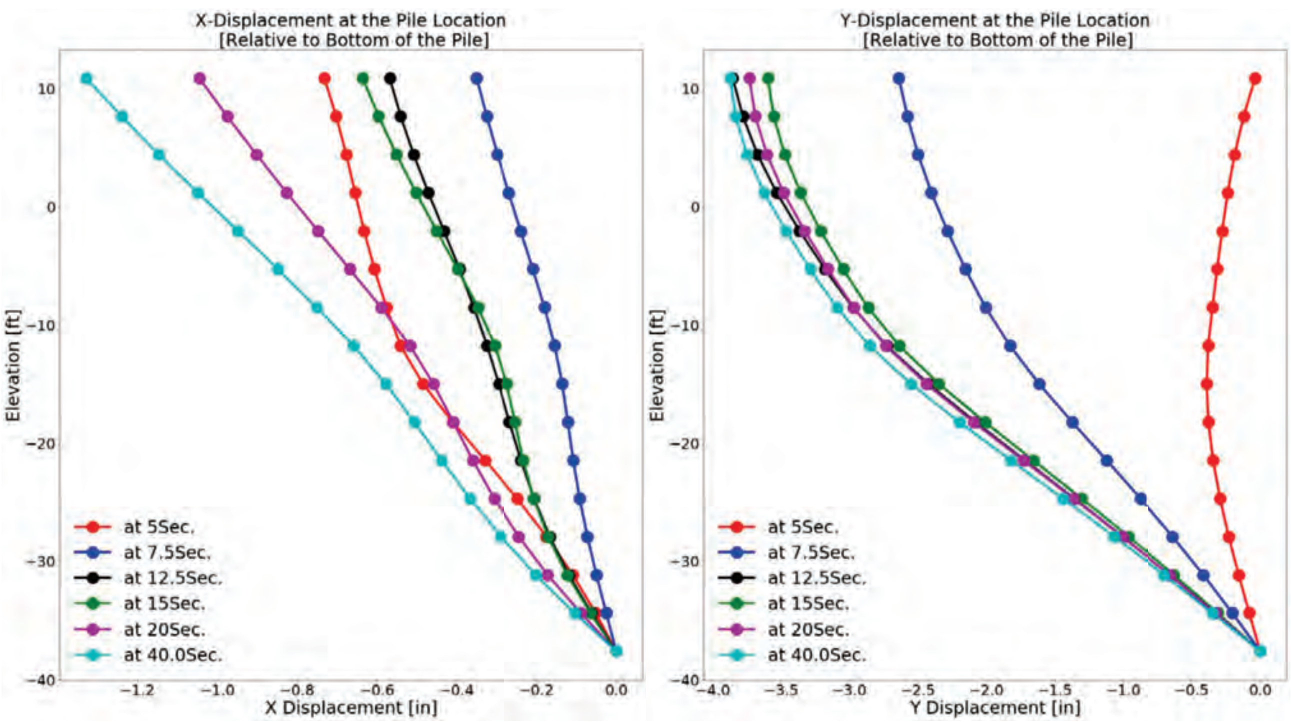


Note: Positive direction in Y direction is toward the slope (away from the ocean).

Y-DIRECTION DISPLACEMENT AT DIFFERENT ELEVATIONS OF THE WALL – MIDDLE SECTION **(EARTHQUAKE SCENARIO A - NORTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-6

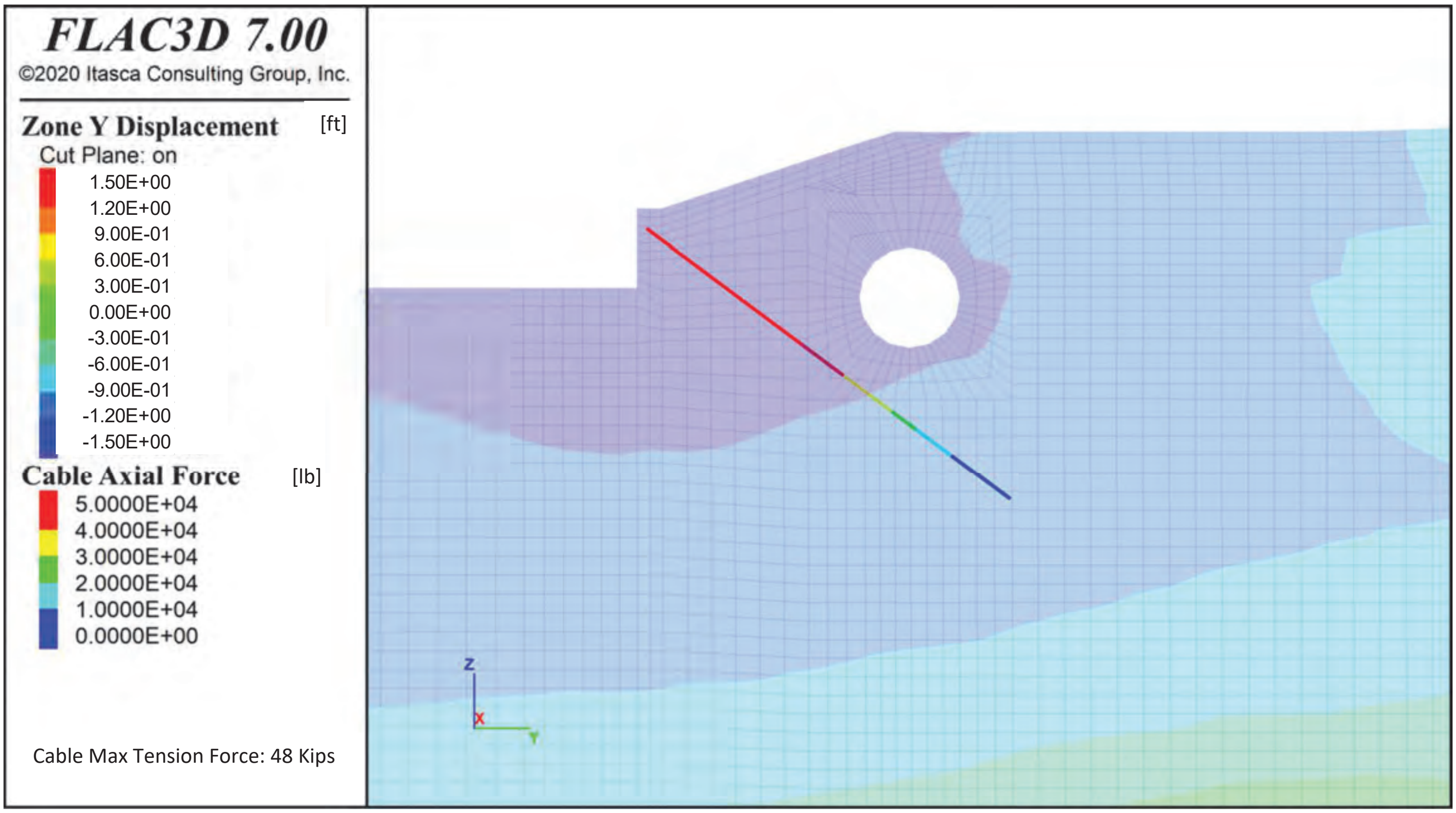


	PORE WATER PRESSURE **(EARTHQUAKE SCENARIO-A AT t= 7.4 SEC- NORTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS
	JOB NO. AGS-18-003 DATE: JUNE 2021	PLATE B-7



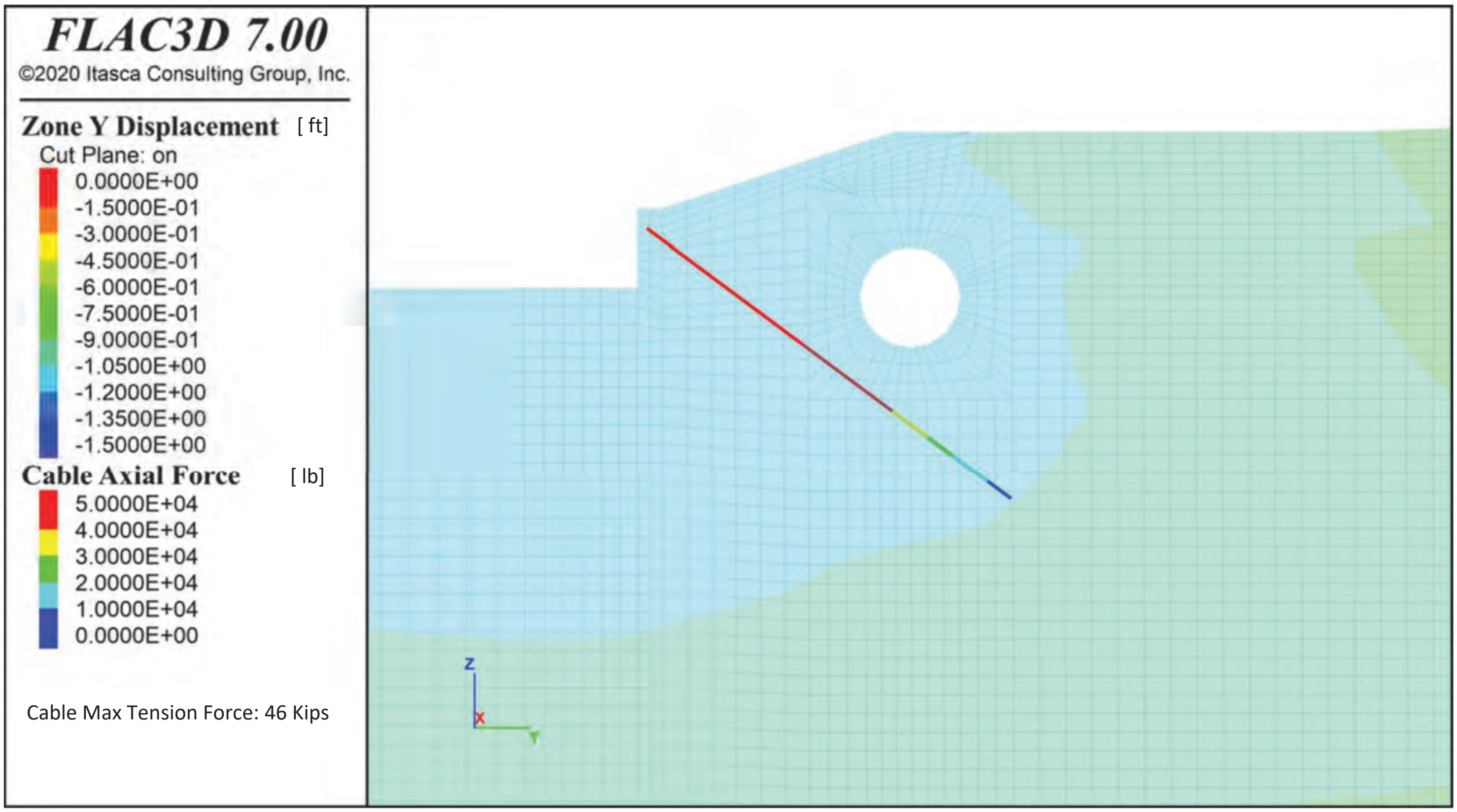
Note 1: Positive direction in Y direction is toward the slope (away from the ocean).
Note 2: Displacements are shown up to fixity depth of the pile.

DISPLACEMENT IN THE PILE OF THE WALL (MIDDLE PILE) **(NORTH REACH - EARTHQUAKE SCENARIO A - NORTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-8



Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 7.8 SEC. - EARTHQUAKE SCENARIO A - NORTH REACH) AND TENSILE FORCE IN THE TIE-BACK SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-9

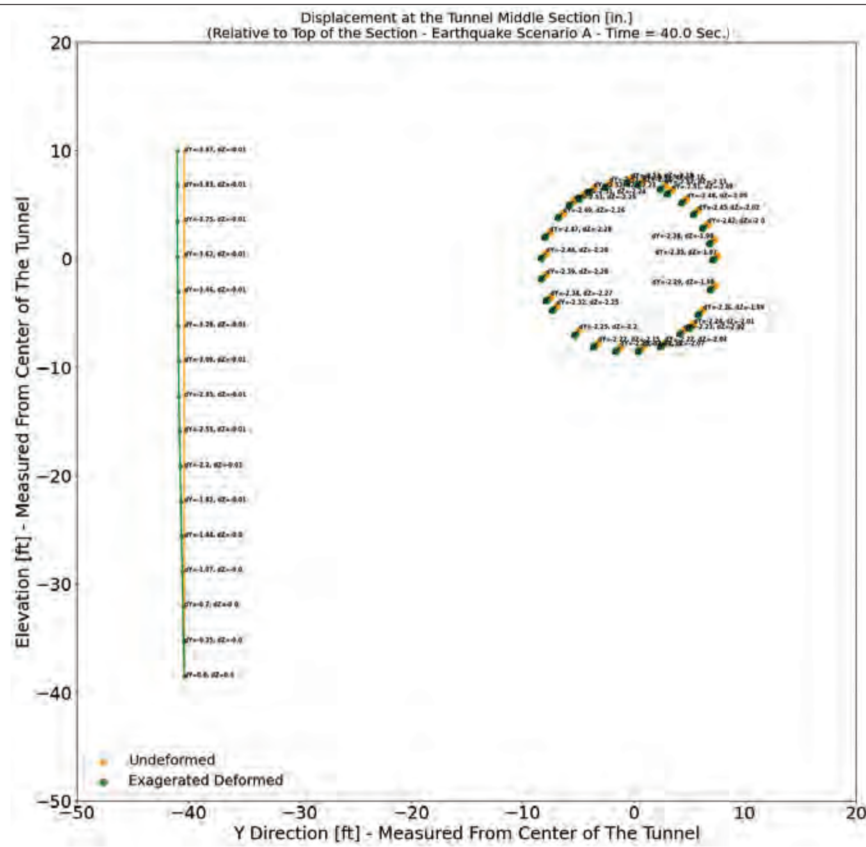


Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION
(AT t = 40.0 SEC. - EARTHQUAKE SCENARIO A - NORTH REACH)
AND TENSILE FORCE IN THE TIE-BACK
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-10

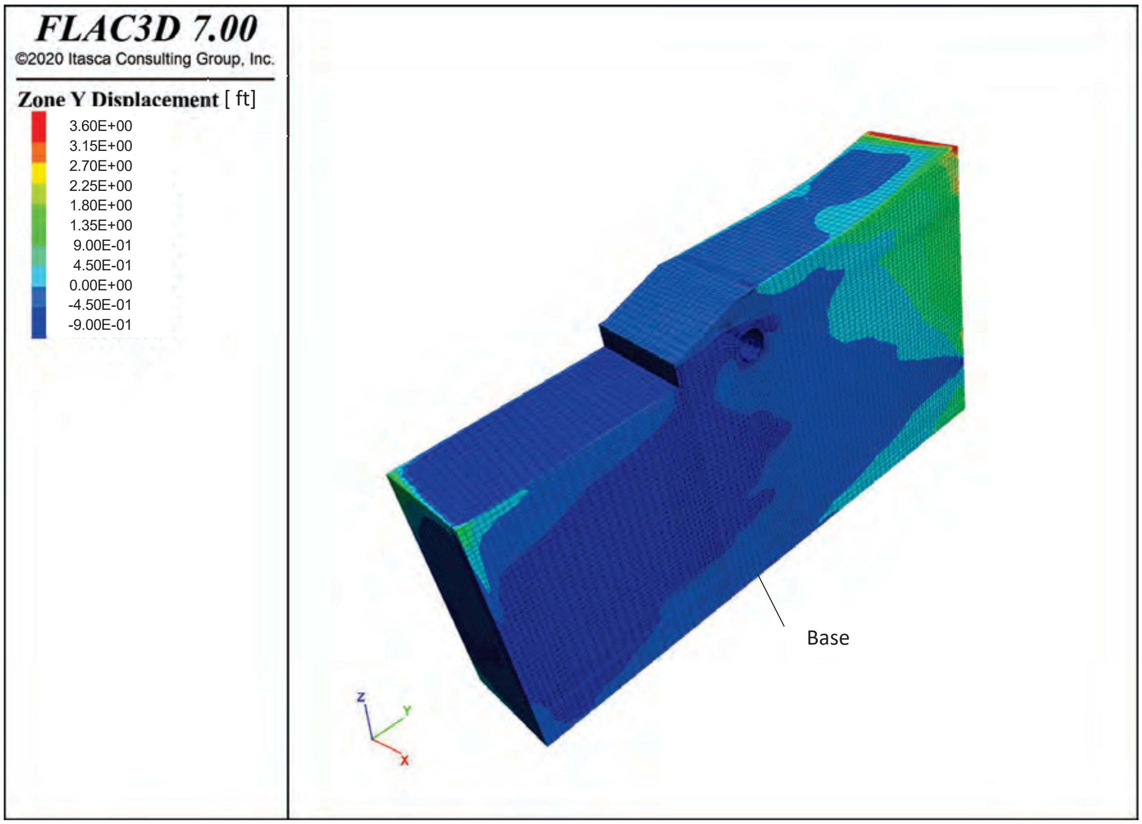


Note 1: Depicted displacements are only due to seismic simulation for the subject SCENARIO and are in addition to the gravitational loading.
Note 2: dY and dZ are relative displacement (relative to bottom of the pile) in Y and Z (elevation) directions, respectively.
Note 3: Displacements are exaggerated. Exaggeration factor in both Z and Y direction is 2.
Note 4: Positive direction in Y direction is toward the slope (away from the ocean).

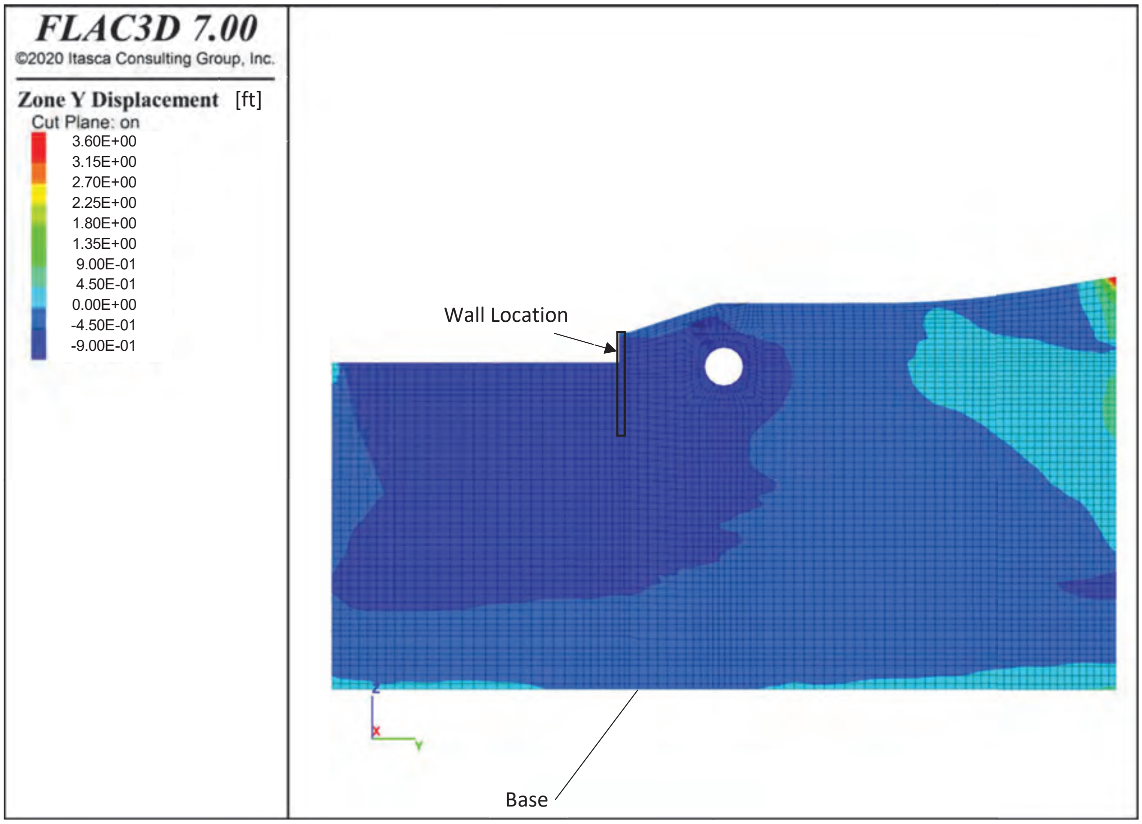
TUNNEL AND PILE DISPLACEMENT
(AT t = 40.0 SEC. - EARTHQUAKE SCENARIO A NORTH REACH)
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA

AGS

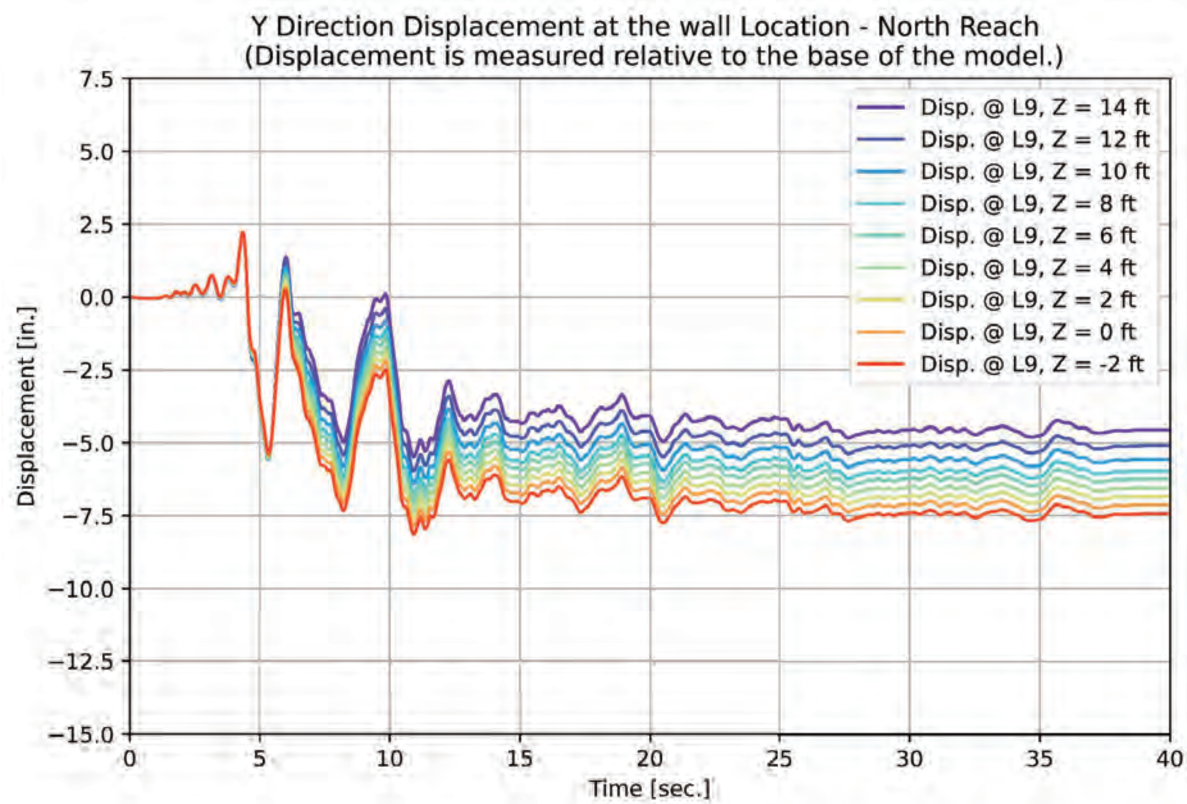
JOB NO. AGS-18-003	DATE: JUNE 2021	Plate B-12



RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO B - NORTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-13

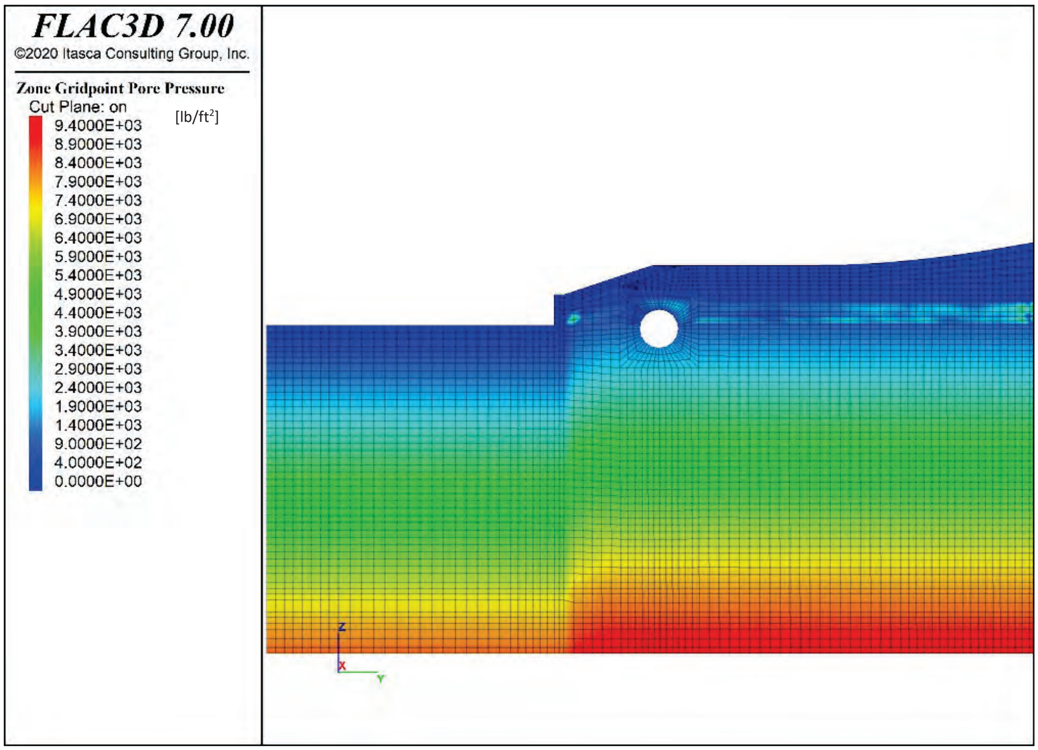


RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO B - NORTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-14

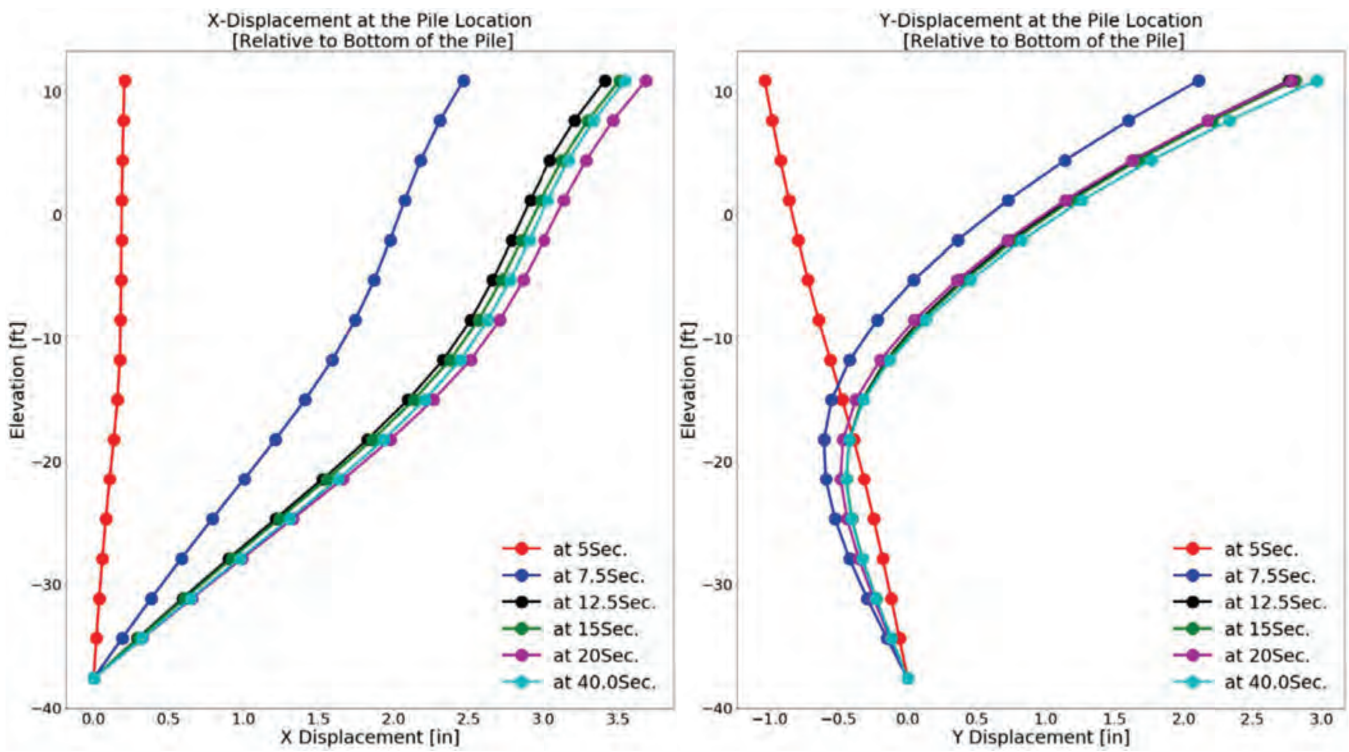


Note: Positive direction in Y direction is toward the slope (away from the ocean).

Y-DIRECTION DISPLACEMENT AT DIFFERENT ELEVATIONS OF THE WALL – MIDDLE SECTION **(EARTHQUAKE SCENARIO B - NORTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-15

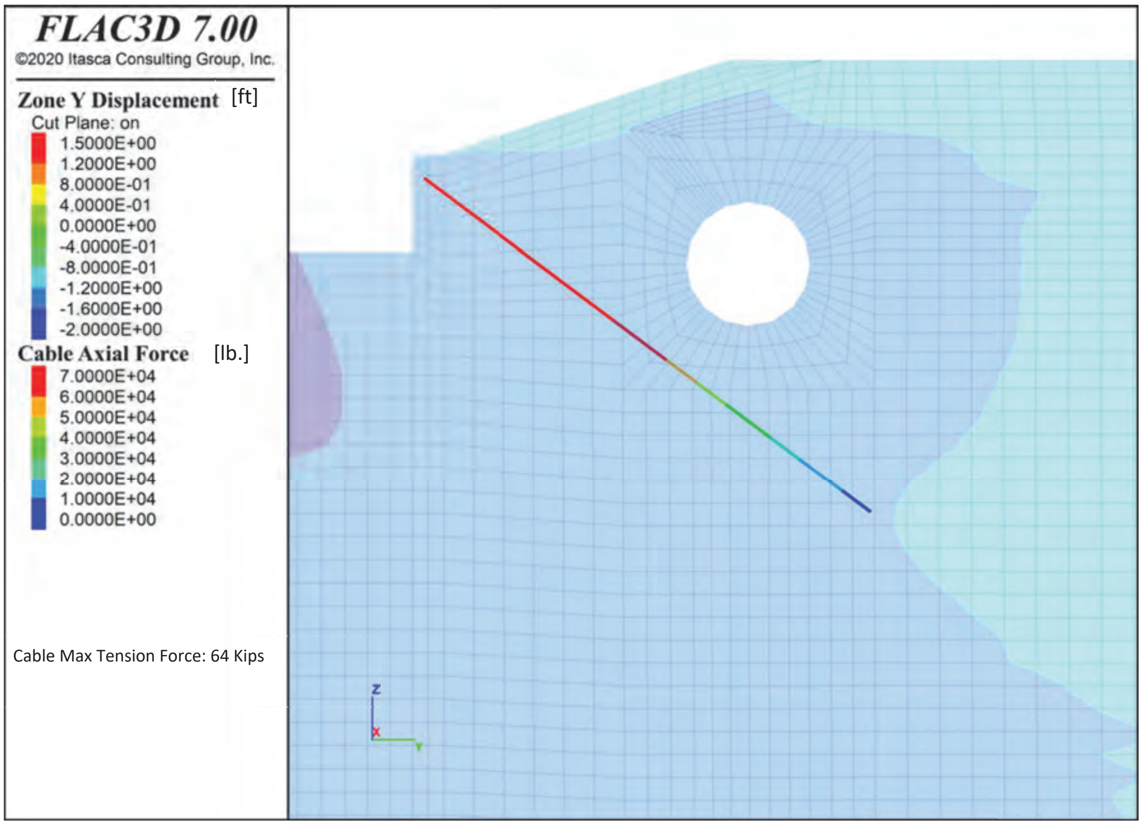


	PORE WATER PRESSURE **(EARTHQUAKE SCENARIO-B AT t= 7.5 SEC. - NORTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA		AGS
	JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-16



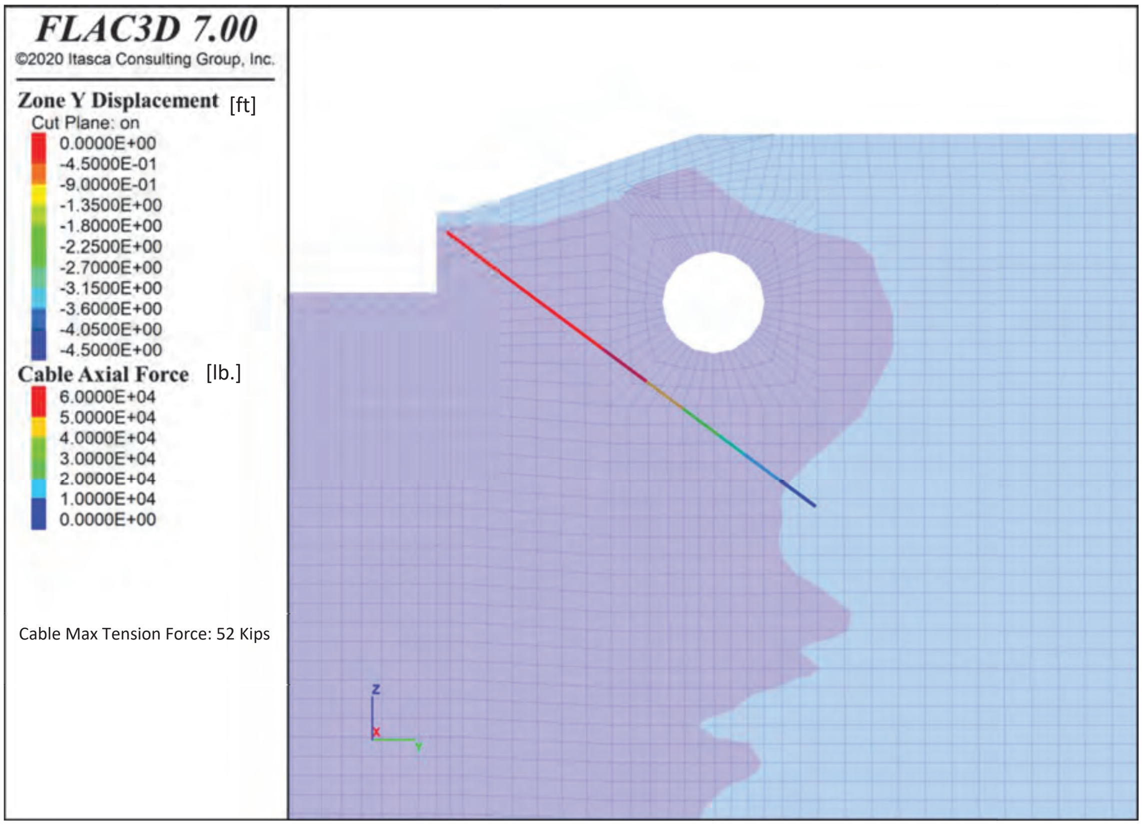
Note 1: Positive direction in Y direction is toward the slope (away from the ocean).
Note 2: Displacements are shown up to fixity depth of the pile.

DISPLACEMENT IN THE PILE OF THE WALL (MIDDLE PILE) **(NORTH REACH - EARTHQUAKE SCENARIO B - NORTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-17



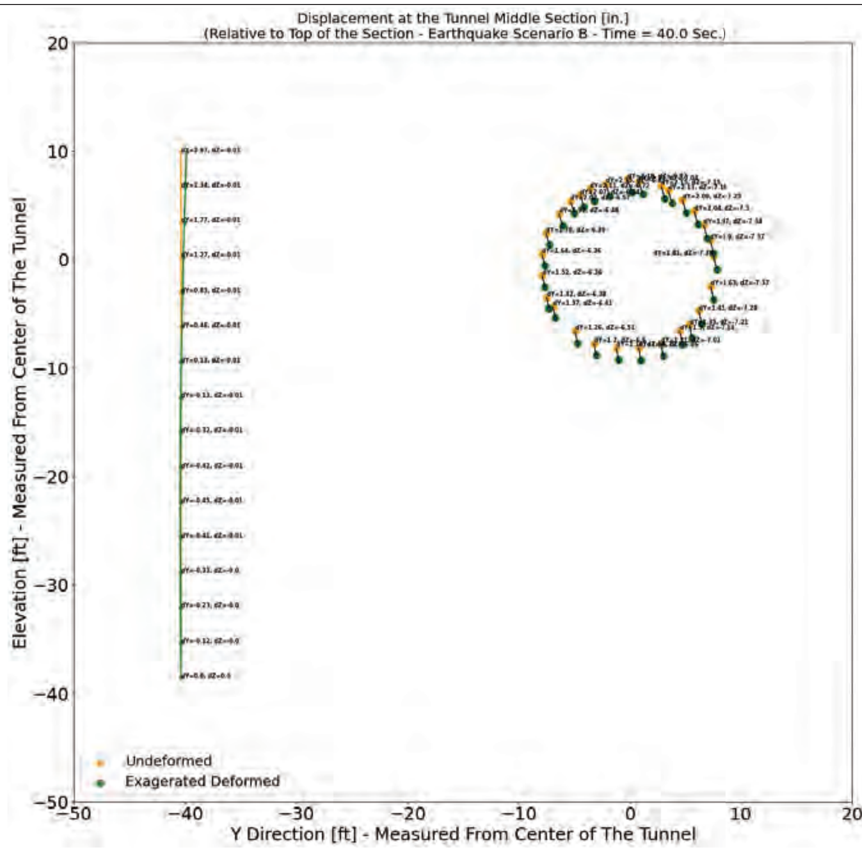
Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 11 SEC. - EARTHQUAKE SCENARIO B - NORTH REACH) AND TENSILE FORCE IN THE TIE-BACK SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-18



Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 40 SEC. - EARTHQUAKE SCENARIO B - NORTH REACH) AND TENSILE FORCE IN THE TIE-BACK SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-19

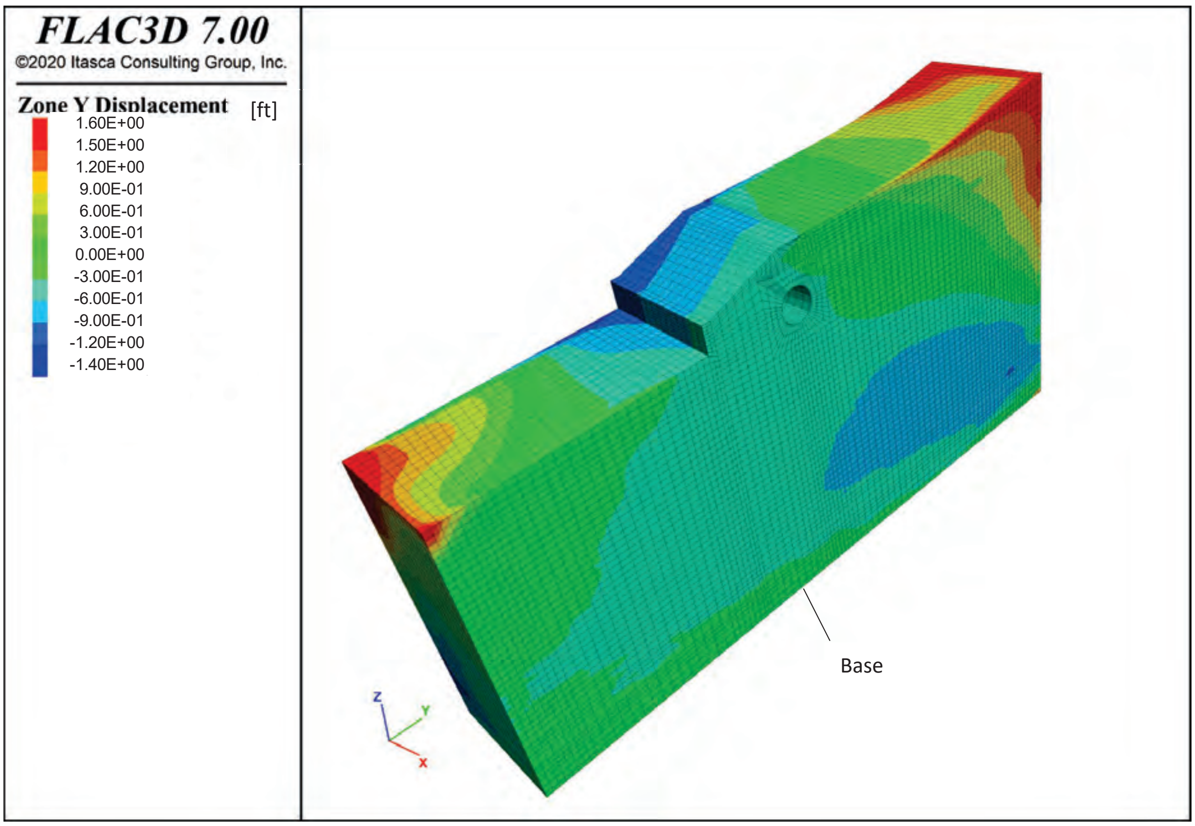


Note 1: Depicted displacements are only due to seismic simulation for the subject SCENARIO and are in addition to the gravitational loading.
Note 2: dY and dZ are relative displacement (relative to bottom of the pile) in Y and Z (elevation) directions, respectively.
Note 3: Displacements are exaggerated. Exaggeration factor in both Z and Y direction is 2.
Note 4: Positive direction in Y direction is toward the slope (away from the ocean).

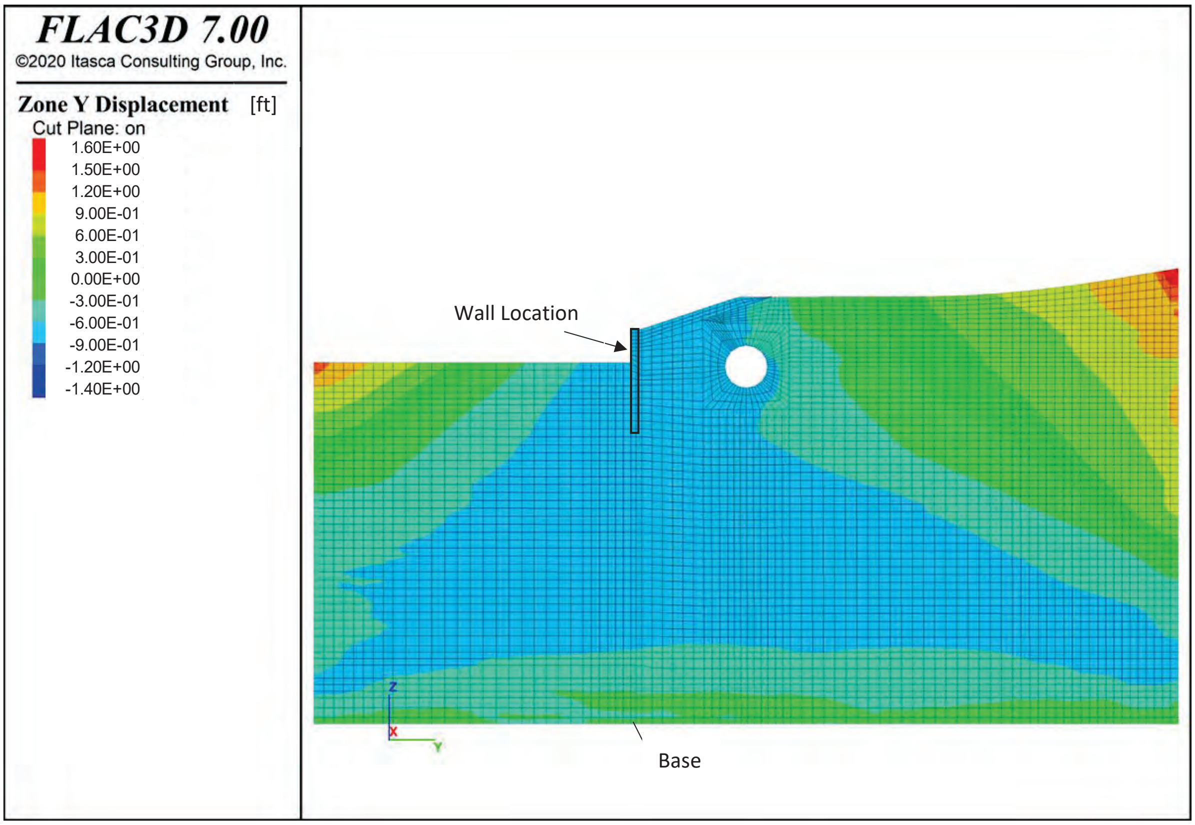
TUNNEL AND PILE DISPLACEMENT
(AT t = 40.0 SEC. - EARTHQUAKE SCENARIO B NORTH REACH)
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA

AGS

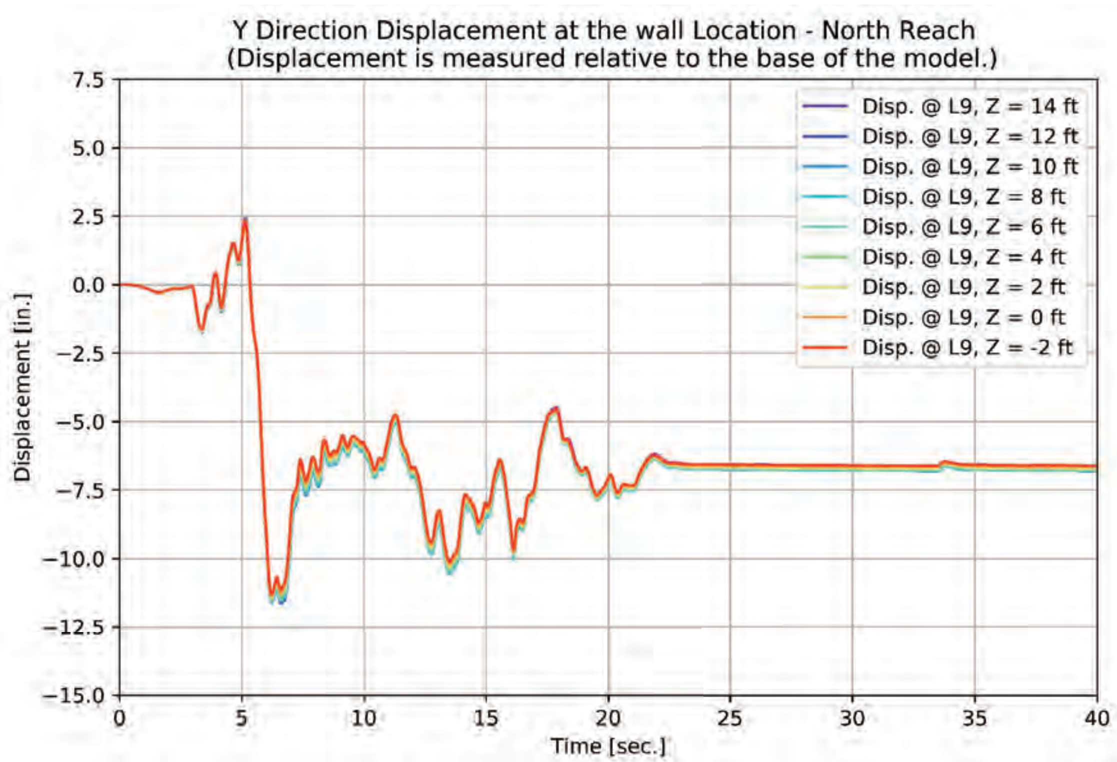
JOB NO. AGS-18-003	DATE: JUNE 2021	Plate B-20



RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO C - NORTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA	AGS
JOB NO. AGS-18-003 / DATE: JUNE 2021	PLATE B-21

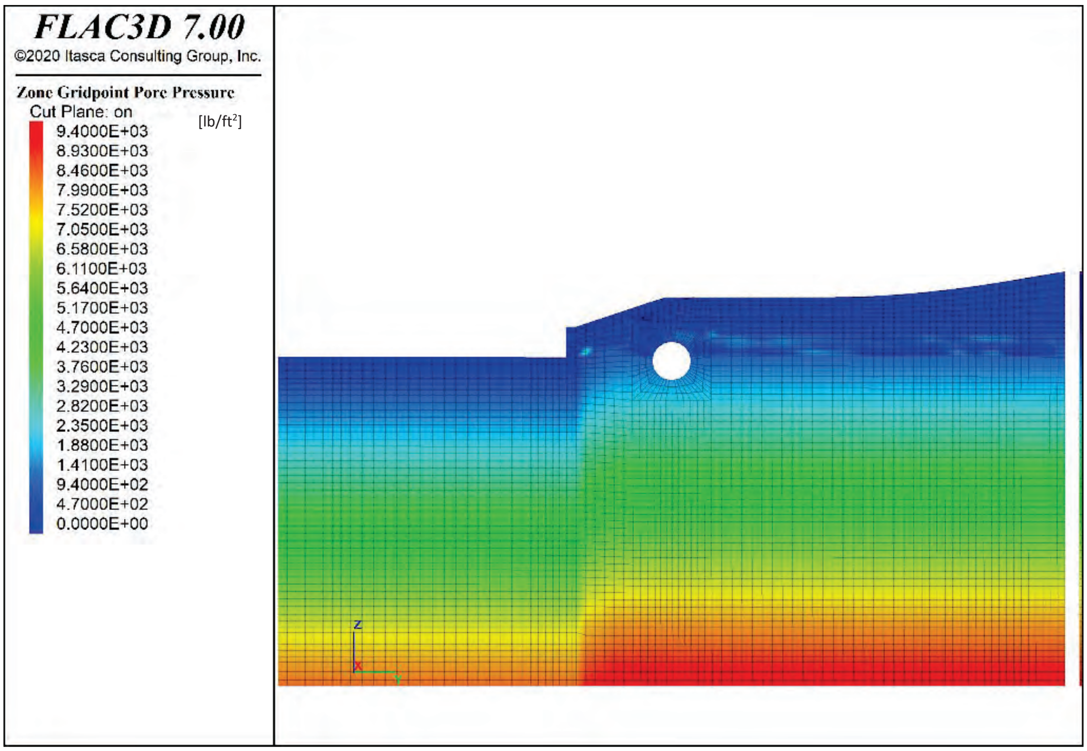


RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 40 SEC. - EARTHQUAKE SCENARIO C - NORTH REACH) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-22

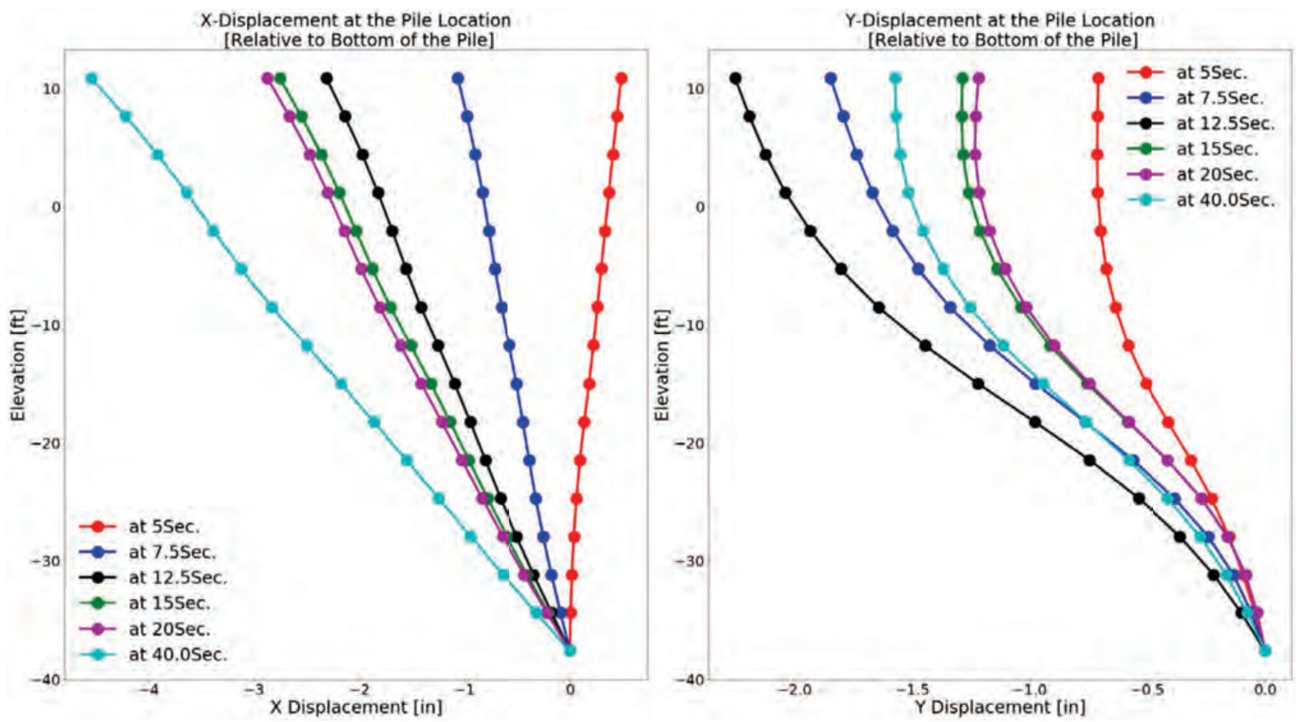


Note: Positive direction in Y direction is toward the slope (away from the ocean).

Y-DIRECTION DISPLACEMENT AT DIFFERENT ELEVATIONS OF THE WALL – MIDDLE SECTION **(EARTHQUAKE SCENARIO C - NORTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-23

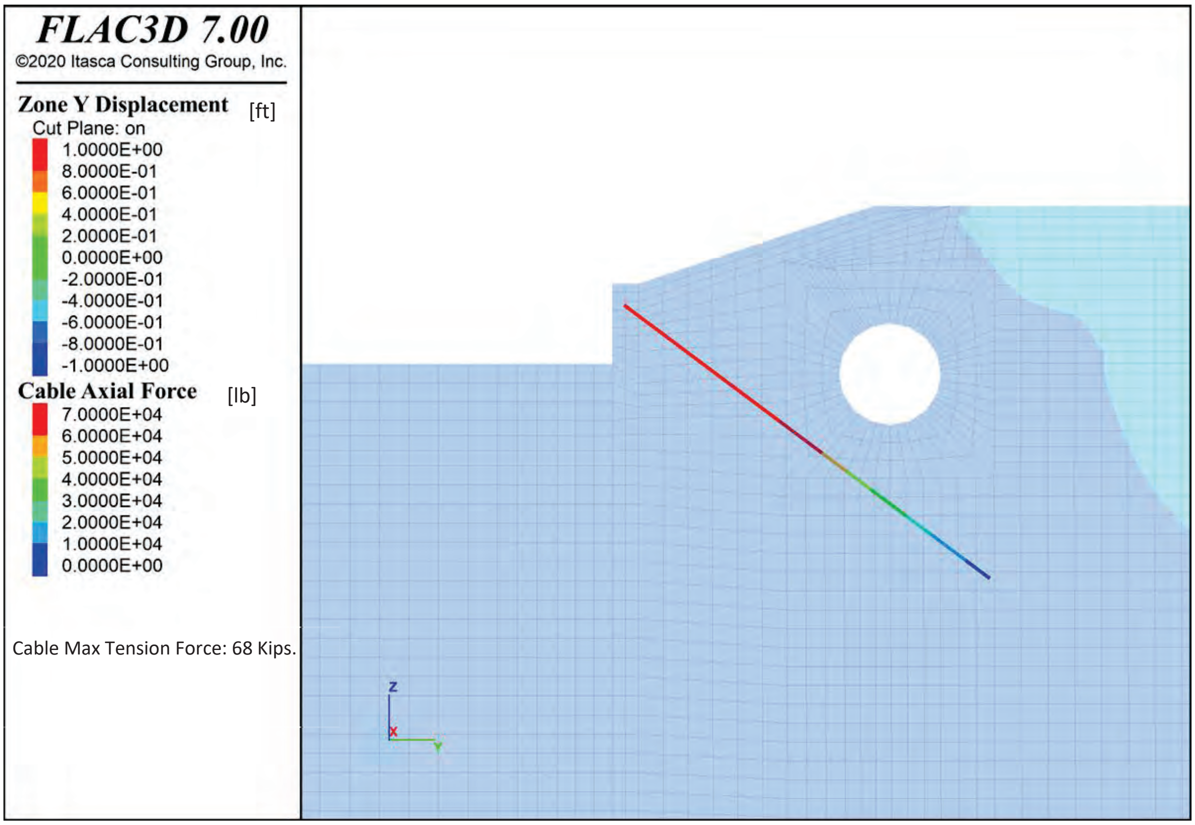


PORE WATER PRESSURE **(EARTHQUAKE SCENARIO-C AT t= 7.5 SEC- NORTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-24

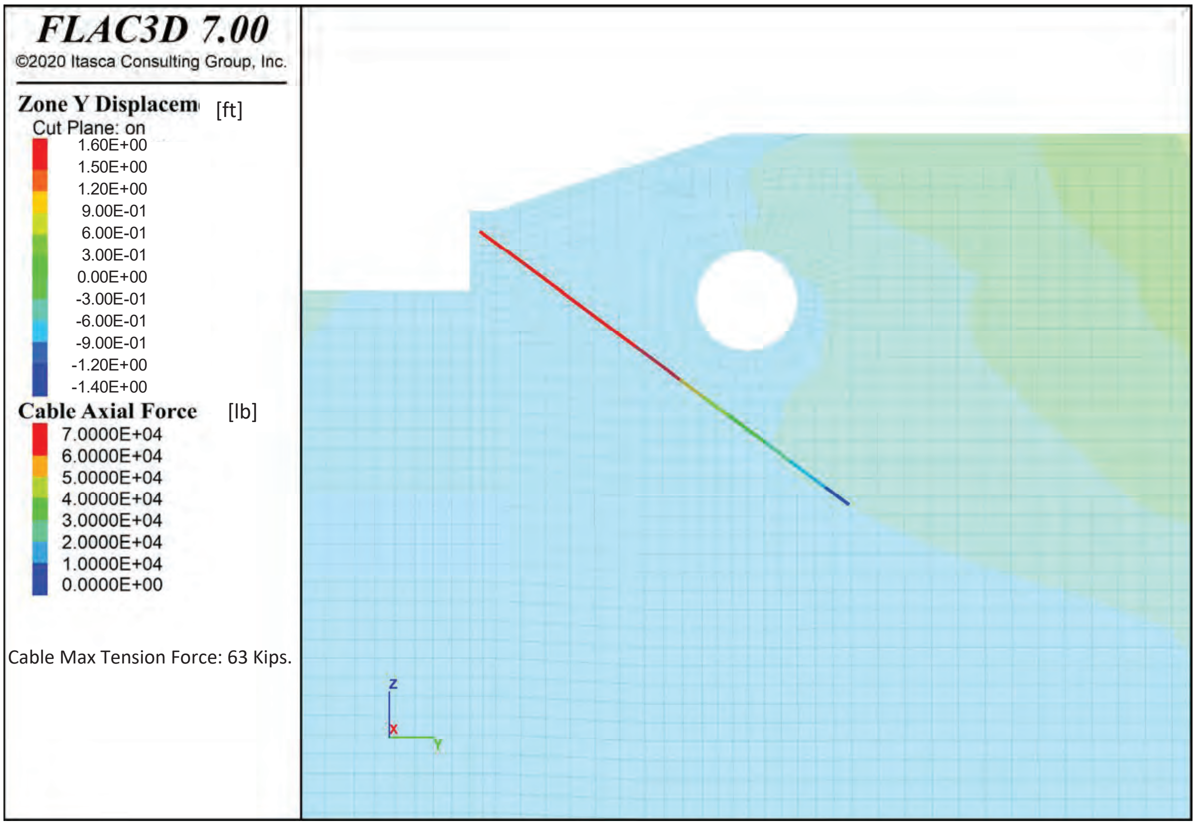


Note 1: Positive direction in Y direction is toward the slope (away from the ocean).
Note 2: Displacements are shown up to fixity depth of the pile.

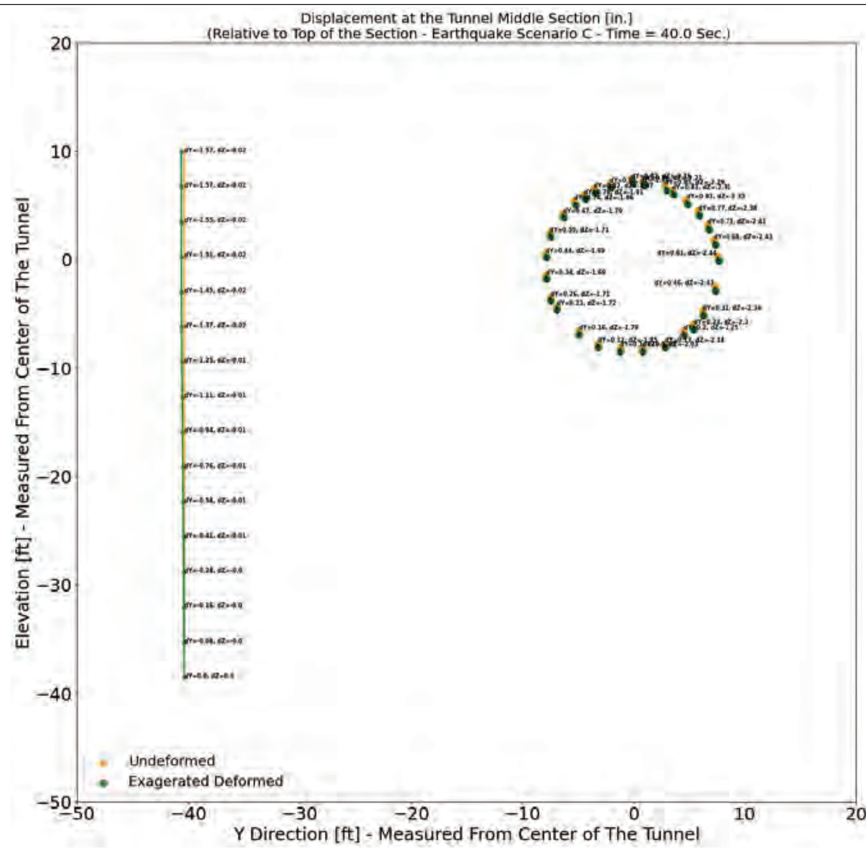
DISPLACEMENT IN THE PILE OF THE WALL (MIDDLE PILE) **(NORTH REACH - EARTHQUAKE SCENARIO C - NORTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-25



RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 6.0 SEC. - EARTHQUAKE SCENARIO C - NORTH REACH) AND TENSILE FORCE IN THE TIE-BACK SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE B-26



RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40. SEC. - EARTHQUAKE SCENARIO C - NORTH REACH)** **AND TENSILE FORCE IN THE TIE-BACK** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS
JOB NO. AGS-18-003 / DATE: JUNE 2021	PLATE B-27



Note 1: Depicted displacements are only due to seismic simulation for the subject SCENARIO and are in addition to the gravitational loading.
Note 2: dY and dZ are relative displacement (relative to bottom of the pile) in Y and Z (elevation) directions, respectively.
Note 3: Displacements are exaggerated. Exaggeration factor in both Z and Y direction is 2.
Note 4: Positive direction in Y direction is toward the slope (away from the ocean).

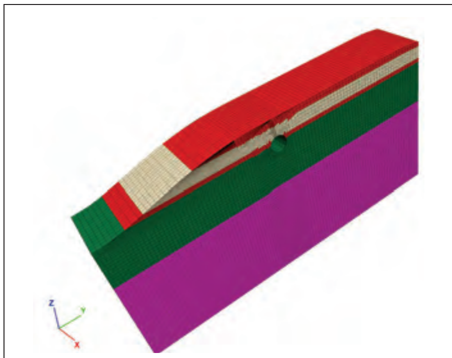
TUNNEL AND PILE DISPLACEMENT
(AT t = 40.0 SEC. - EARTHQUAKE SCENARIO C NORTH REACH)
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA

AGS

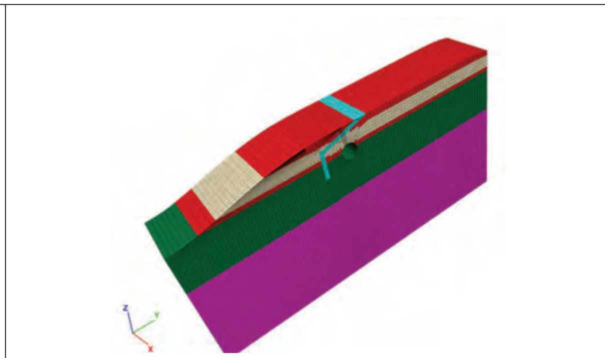
JOB NO. AGS-18-003	DATE: JUNE 2021	Plate B-28

Appendix C

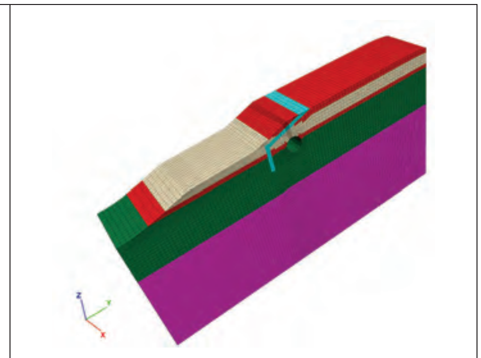
FLAC3D NUMERICAL SSSI SIMULATION RESULTS FOR EQR



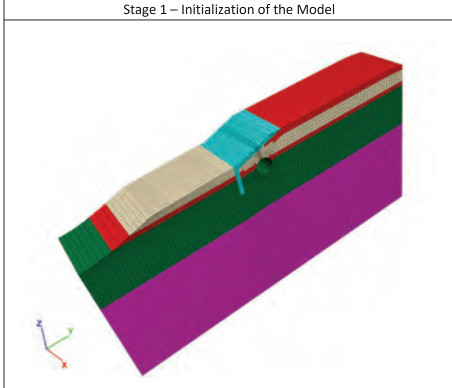
Stage 1 – Initialization of the Model



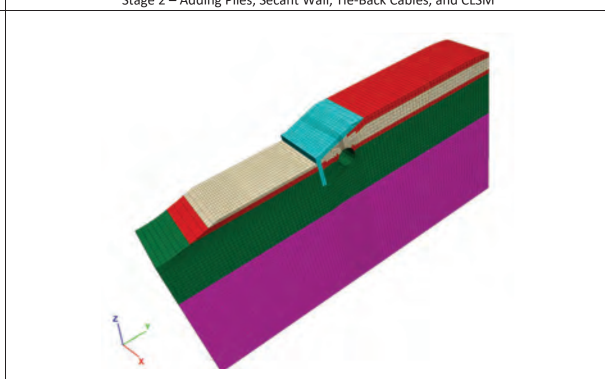
Stage 2 – Adding Piles, Secant Wall, Tie-Back Cables, and CLSM



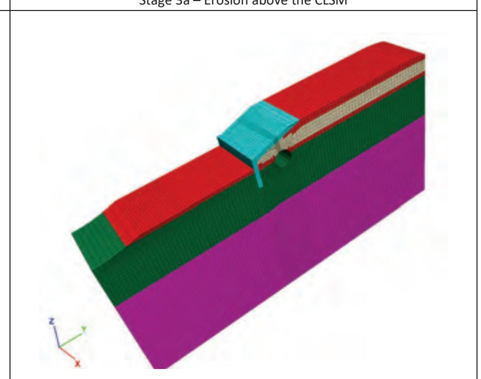
Stage 3a – Erosion above the CLSM



Stage 3b – Erosion above the CLSM



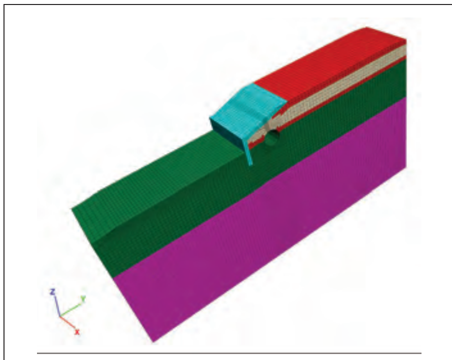
Stage 3c – Erosion above the CLSM



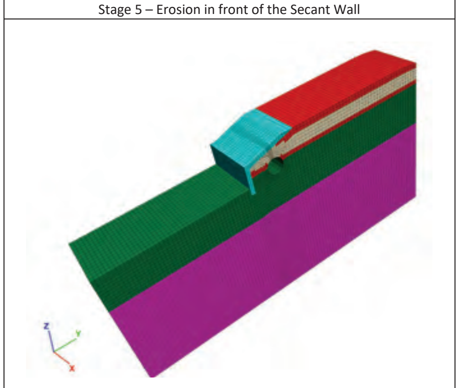
Stage 4 – in front of the Secant Wall

CLSM and Wall	Liquefiable Layer
Fill	Colma Formation
	Merced Formation

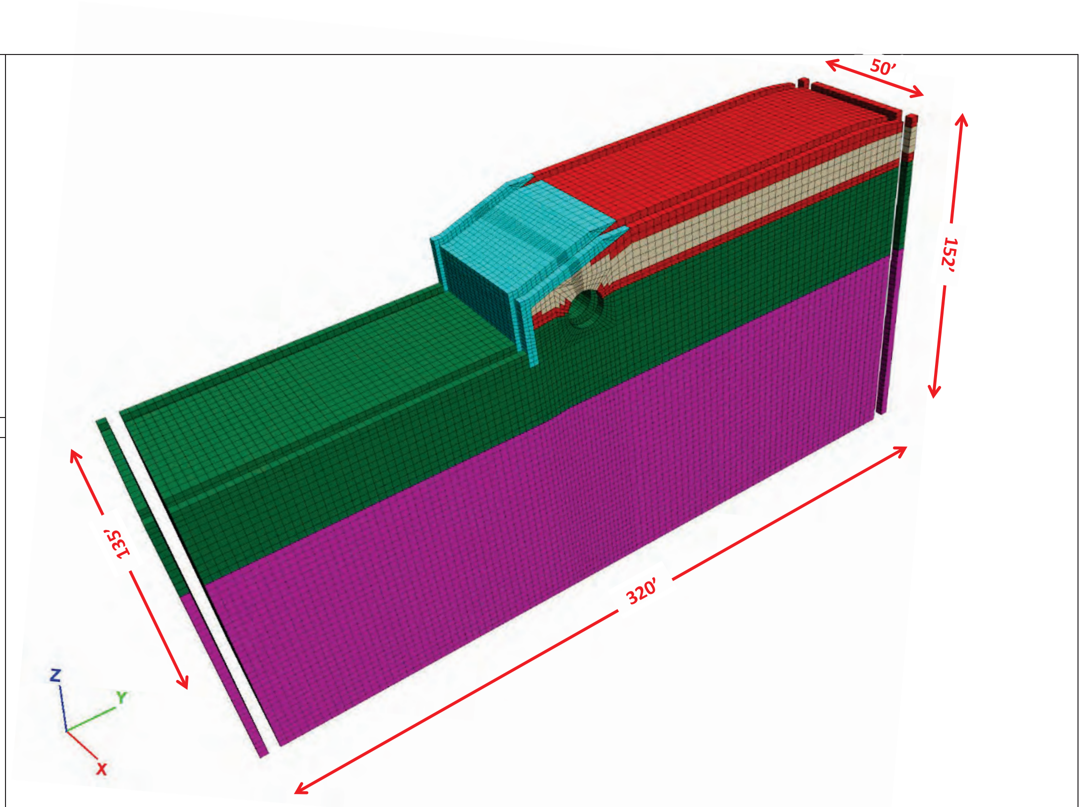
EQR MESH – DIFFERENT STAGES OF THE NUMERICAL SIMULATION SOUTH OCEAN BEACH C OASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-1 a



Stage 5 – Erosion in front of the Secant Wall



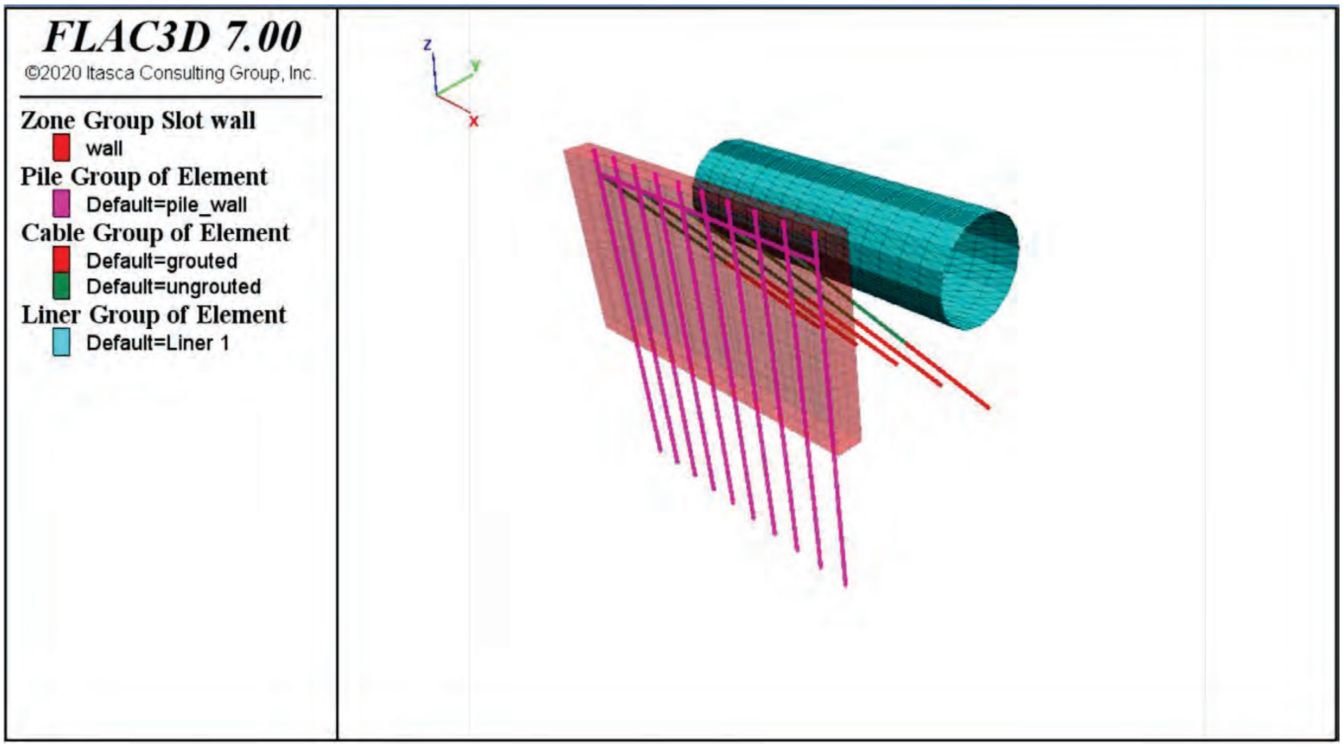
Stage 6 – Erosion in front of the Secant Wall



Mesh Used to Seismic Analysis – Includes Free Field Boundary Conditions

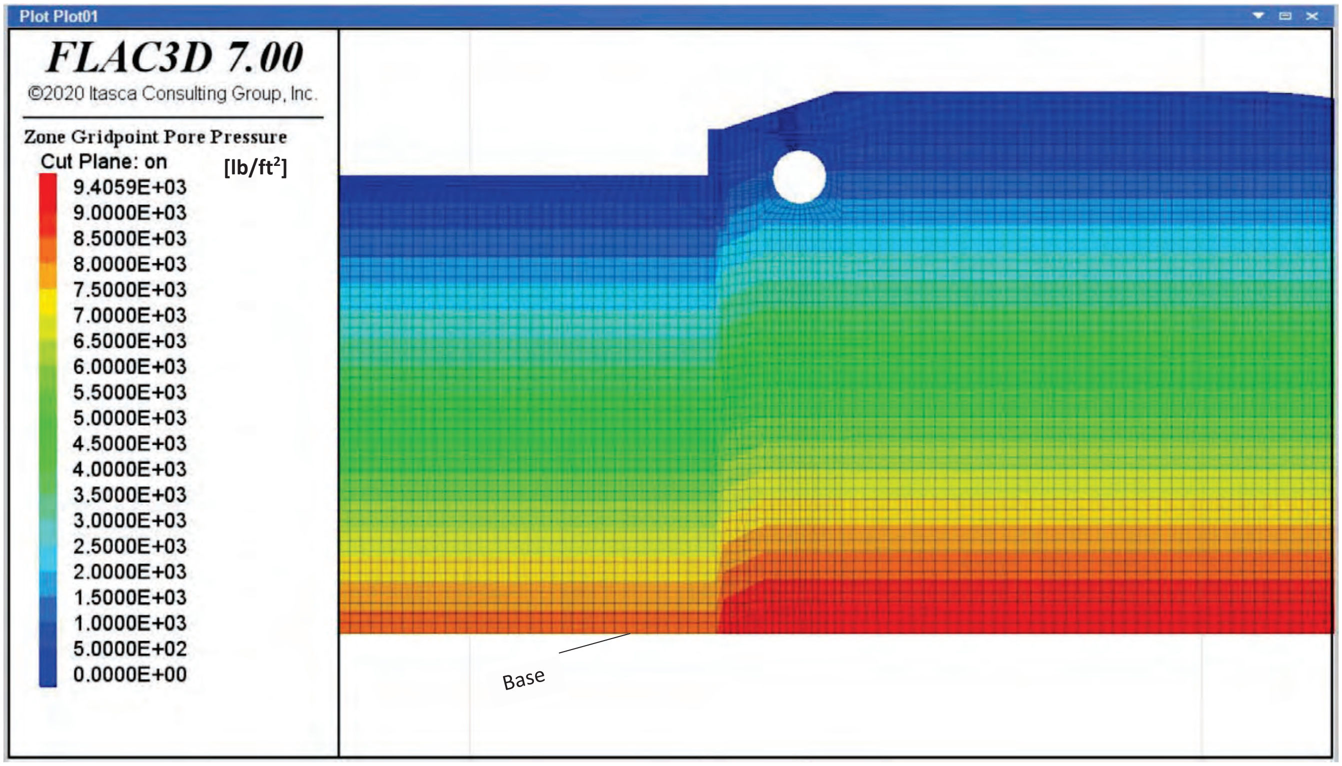
CLSM and Wall	Liquefiable Layer
Fill	Colma Formation
	Merced Formation

EQR MESH – DIFFERENT STAGES OF THE NUMERICAL SIMULATION SOUTH OCEAN BEACH C OASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-1 b

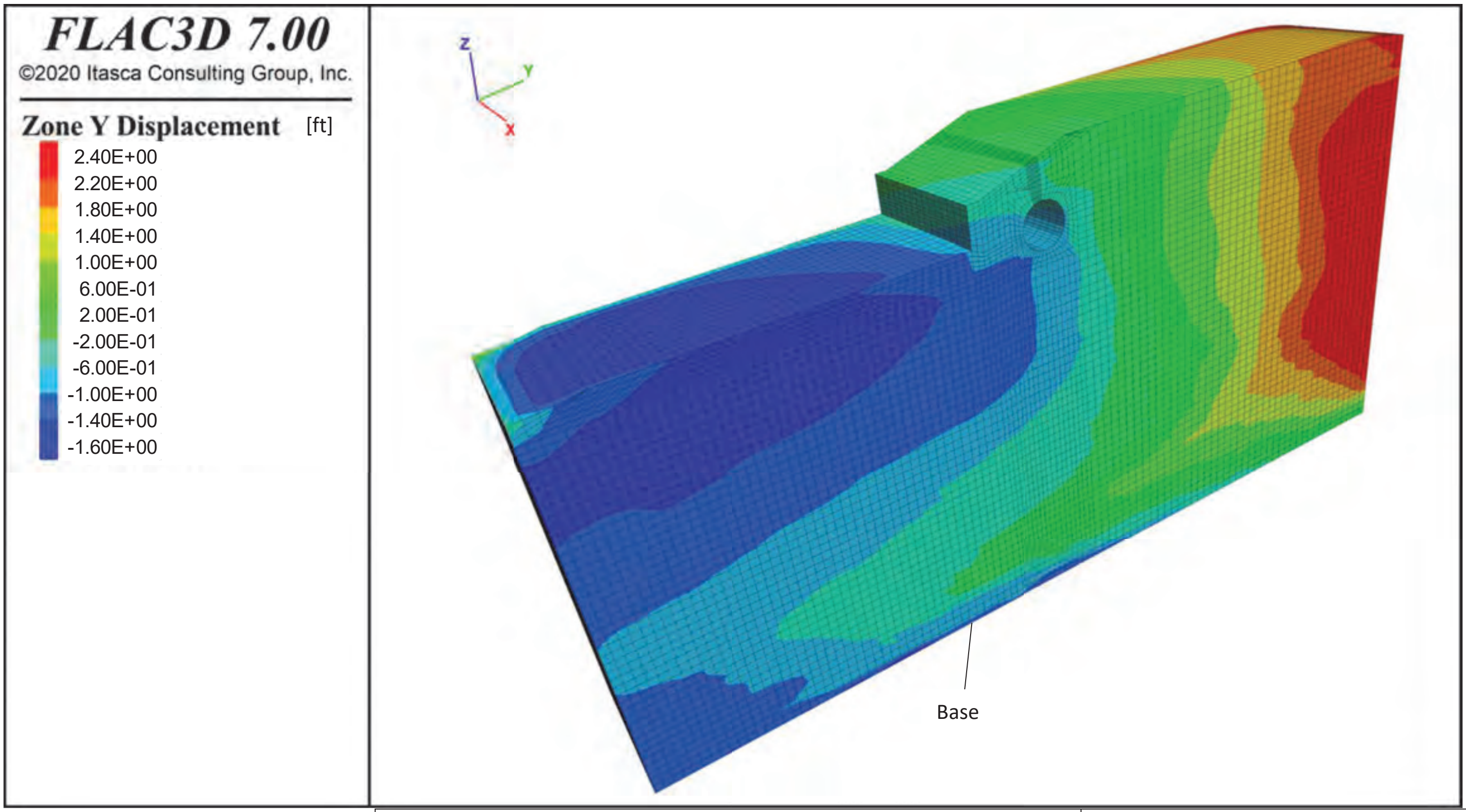


STRUCTURAL ELEMENTS OF THE MODEL **(EARTHQUAKE SCENARIO A - EQR)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS

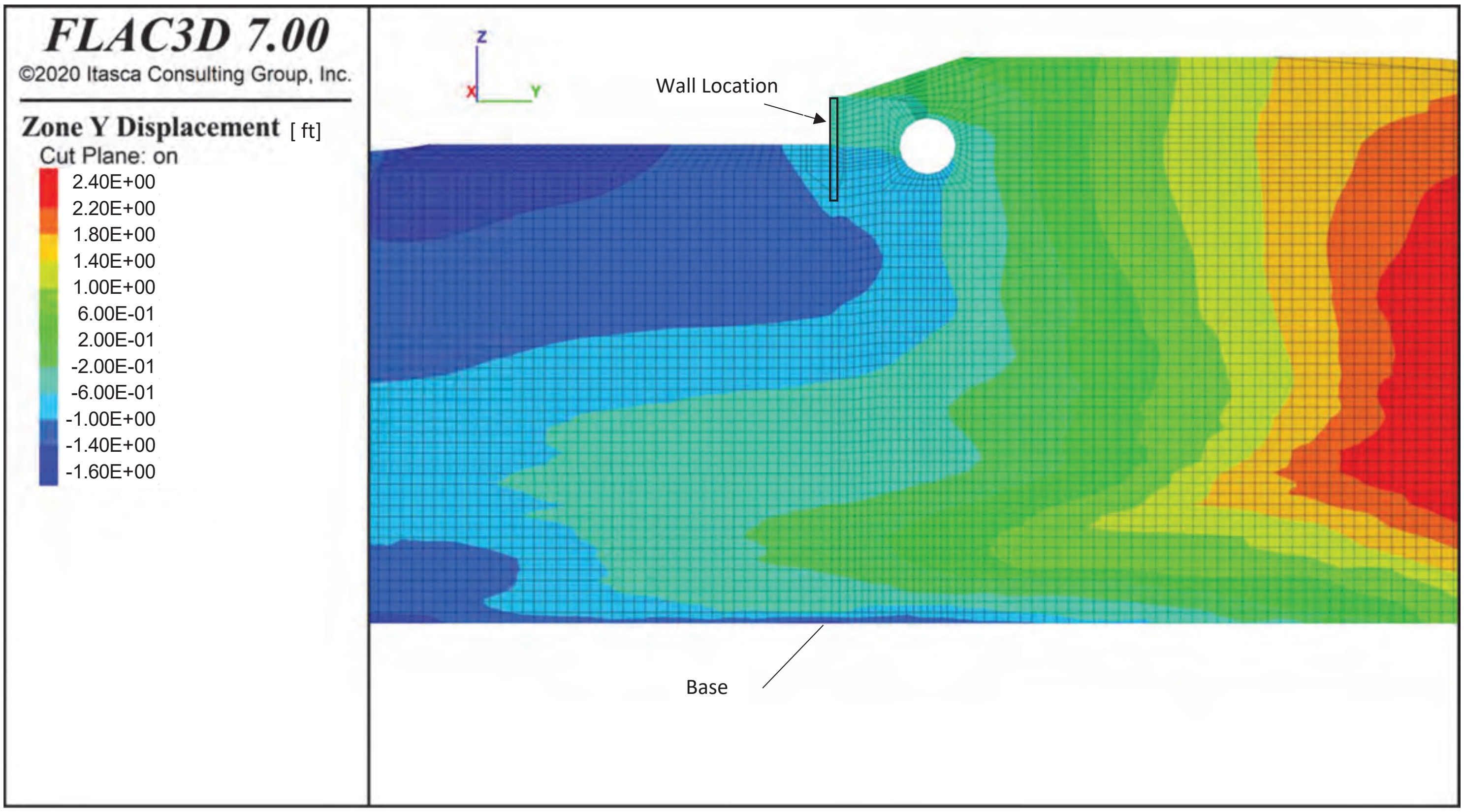
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-2



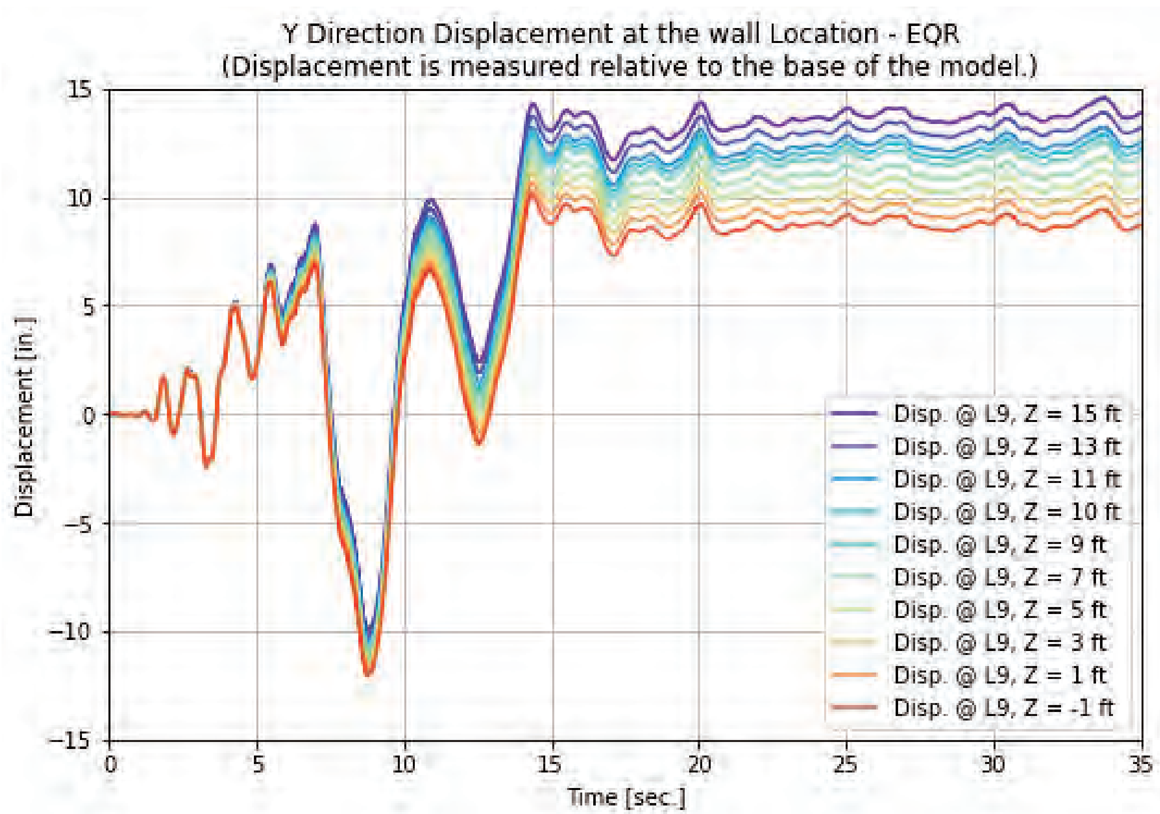
HYDROSTATIC PORE-WATER PRESSURE PRIOR TO SHAKING (EQR) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-3



RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 35 SEC. - EARTHQUAKE SCENARIO A - EQR)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-4

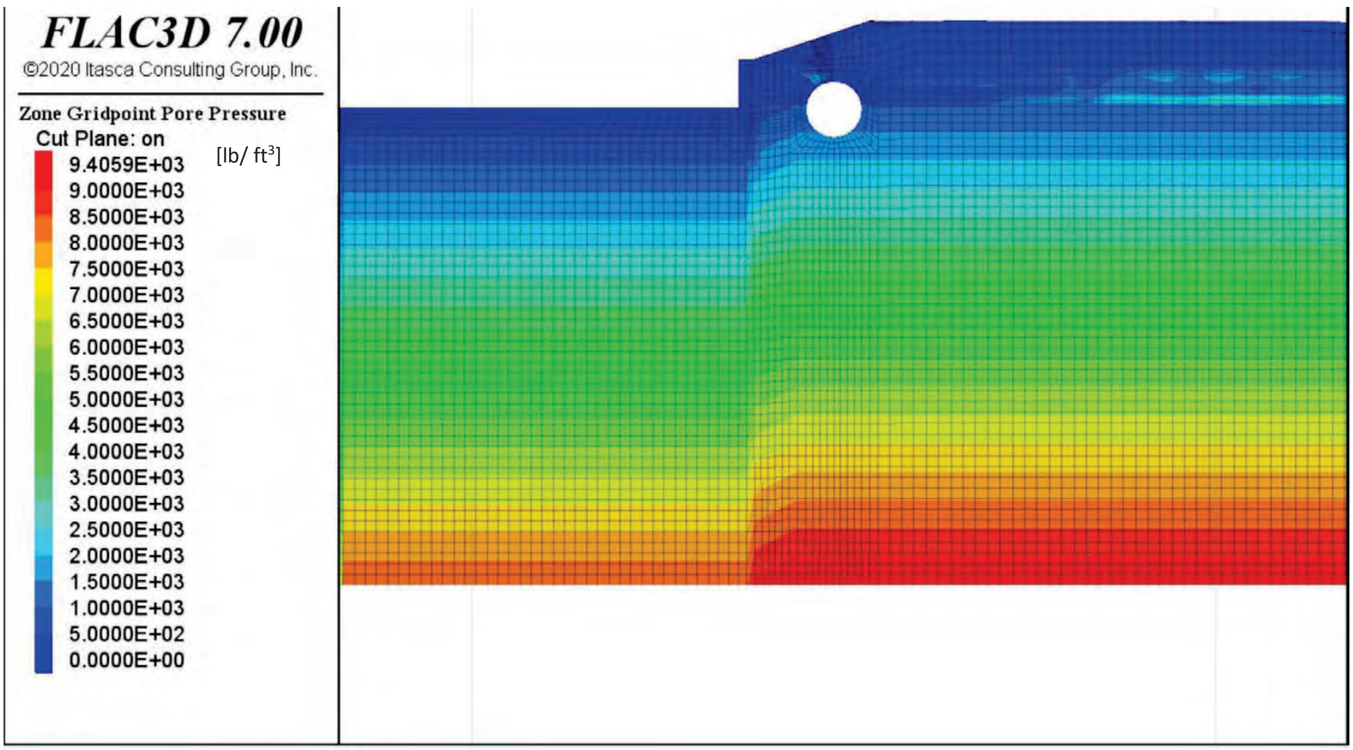


RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 35 SEC. - EARTHQUAKE SCENARIO A - EQR)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-5

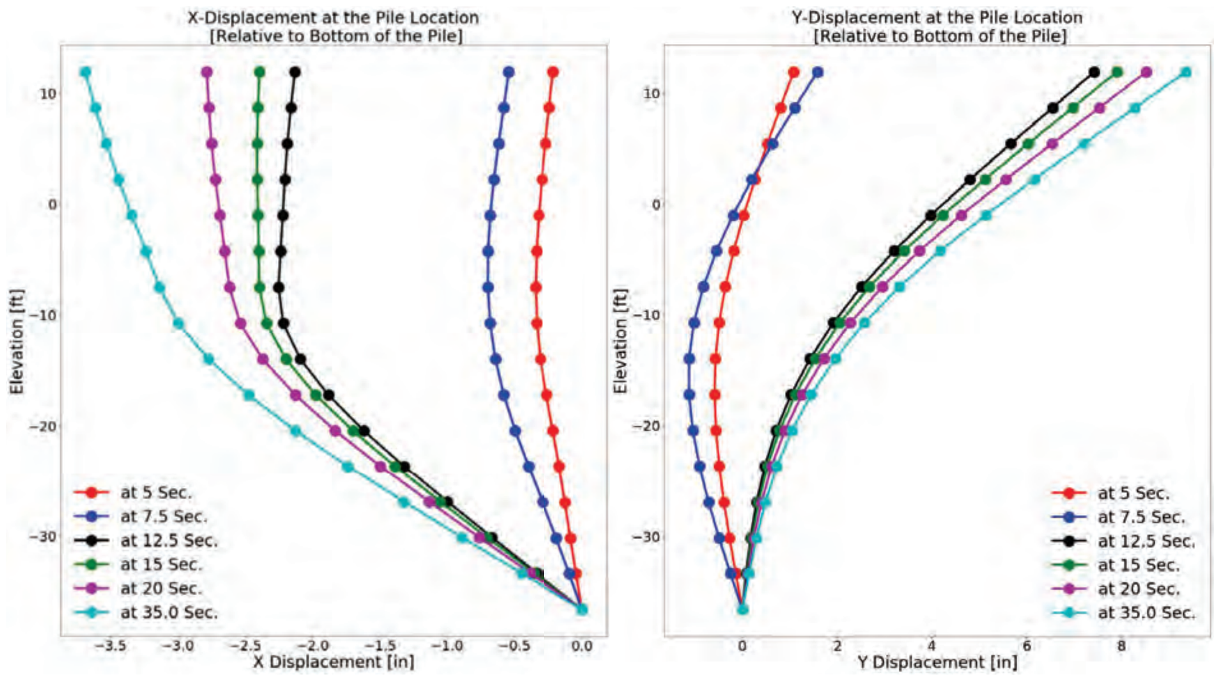


Note: Positive direction in Y direction is toward the slope (away from the ocean).

Y-DIRECTION DISPLACEMENT AT DIFFERENT ELEVATIONS OF THE WALL – MIDDLE SECTION (EARTHQUAKE SCENARIO A - EQR) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-6

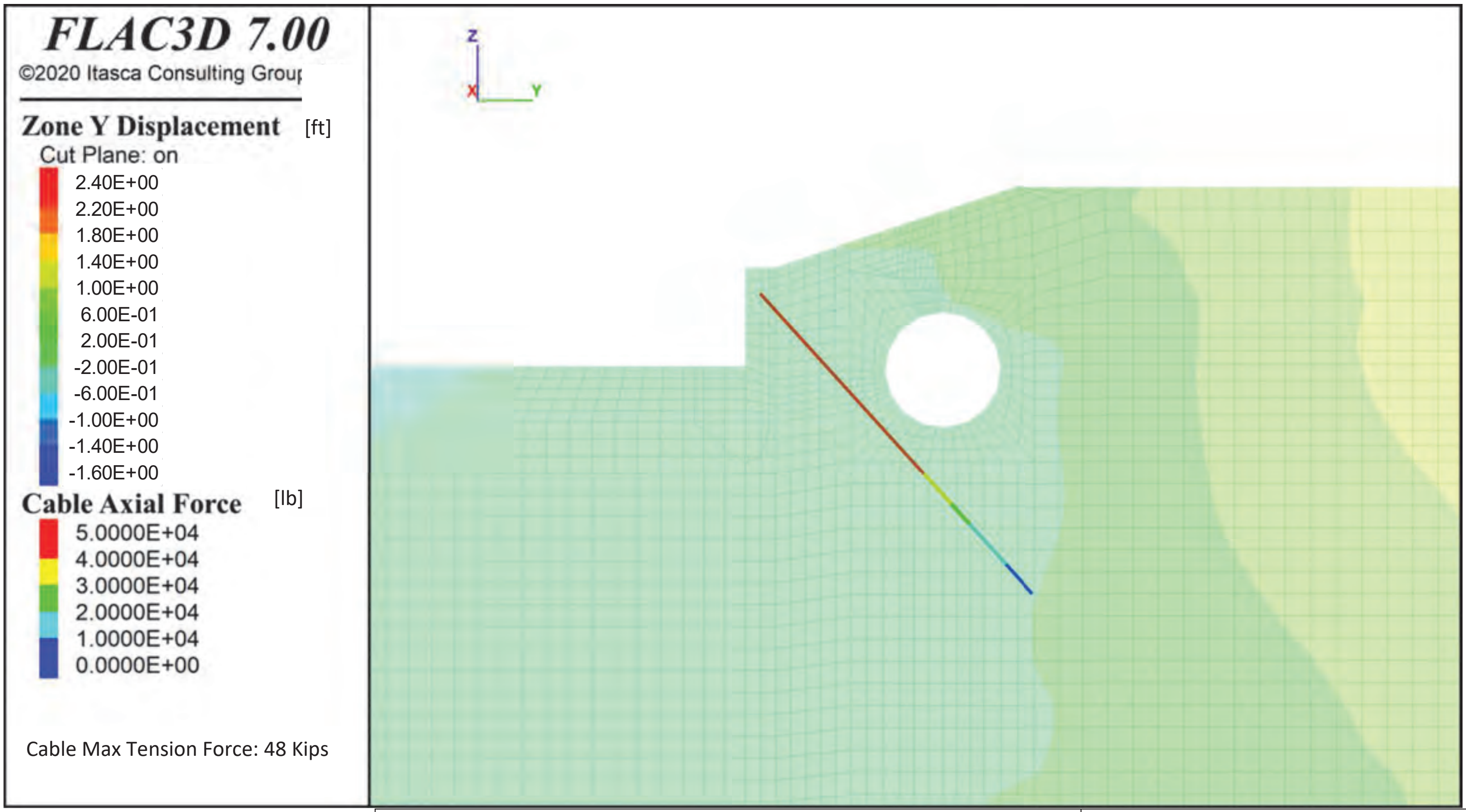


	PORE WATER PRESSURE **(EARTHQUAKE SCENARIO-A AT t= 7.4 SEC- EQR)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS
	JOB NO. AGS-18-003 / DATE: JUNE 2021	PLATE C-7



Note 1: Positive direction in Y direction is toward the slope (away from the ocean).
Note 2: Displacements are shown up to fixity depth of the pile.

DISPLACEMENT IN THE PILE OF THE WALL (MIDDLE PILE) (EQR - EARTHQUAKE SCENARIO A - EQR) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-8

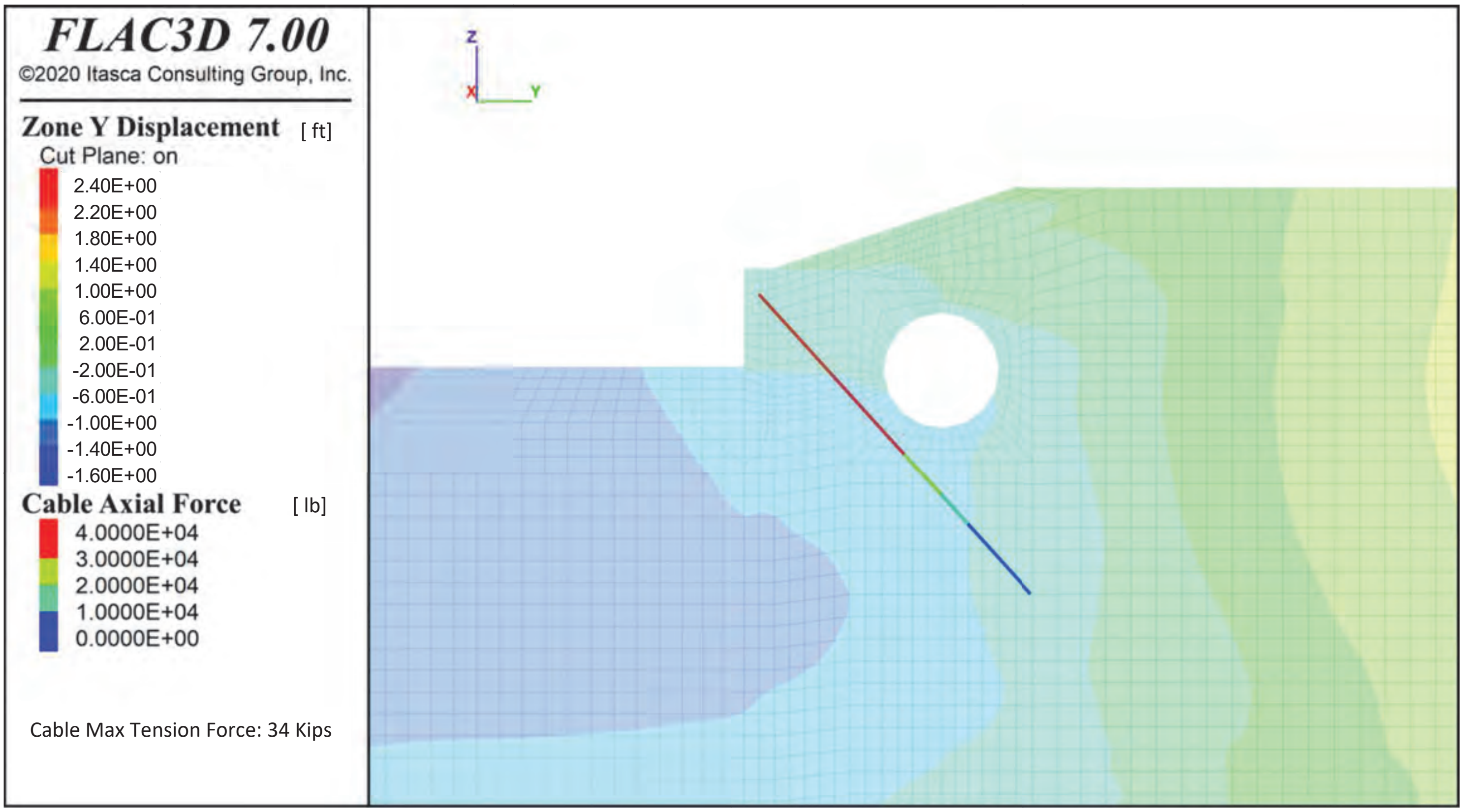


Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION
(AT t = 7.5 SEC. - EARTHQUAKE SCENARIO A - EQR)
AND TENSILE FORCE IN THE TIE-BACK
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA

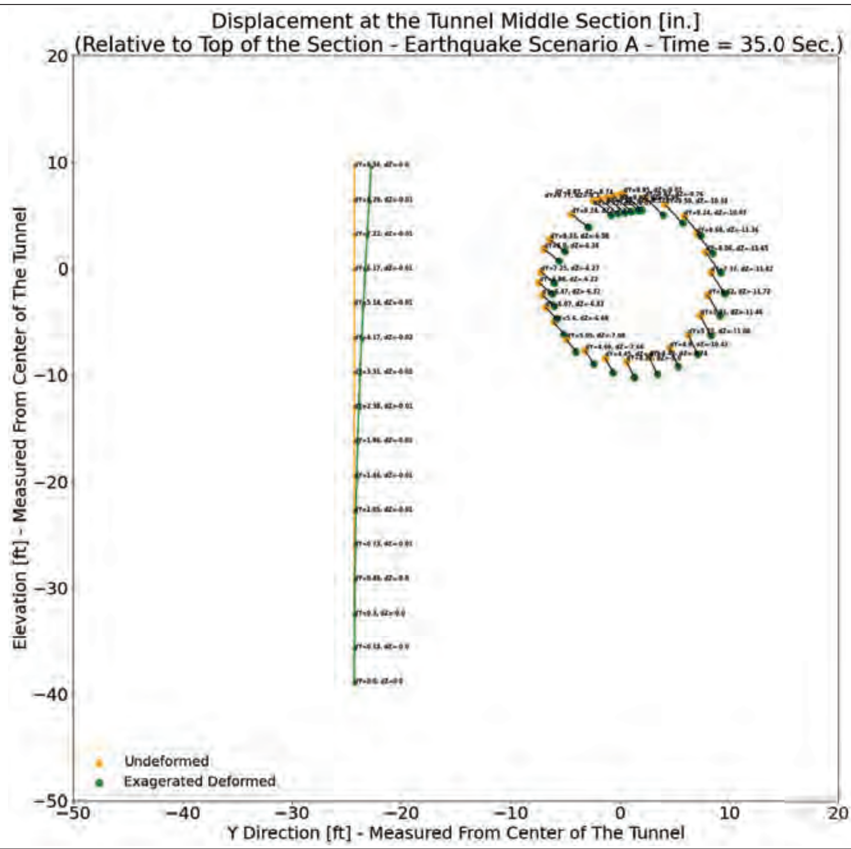
AGS

JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-9

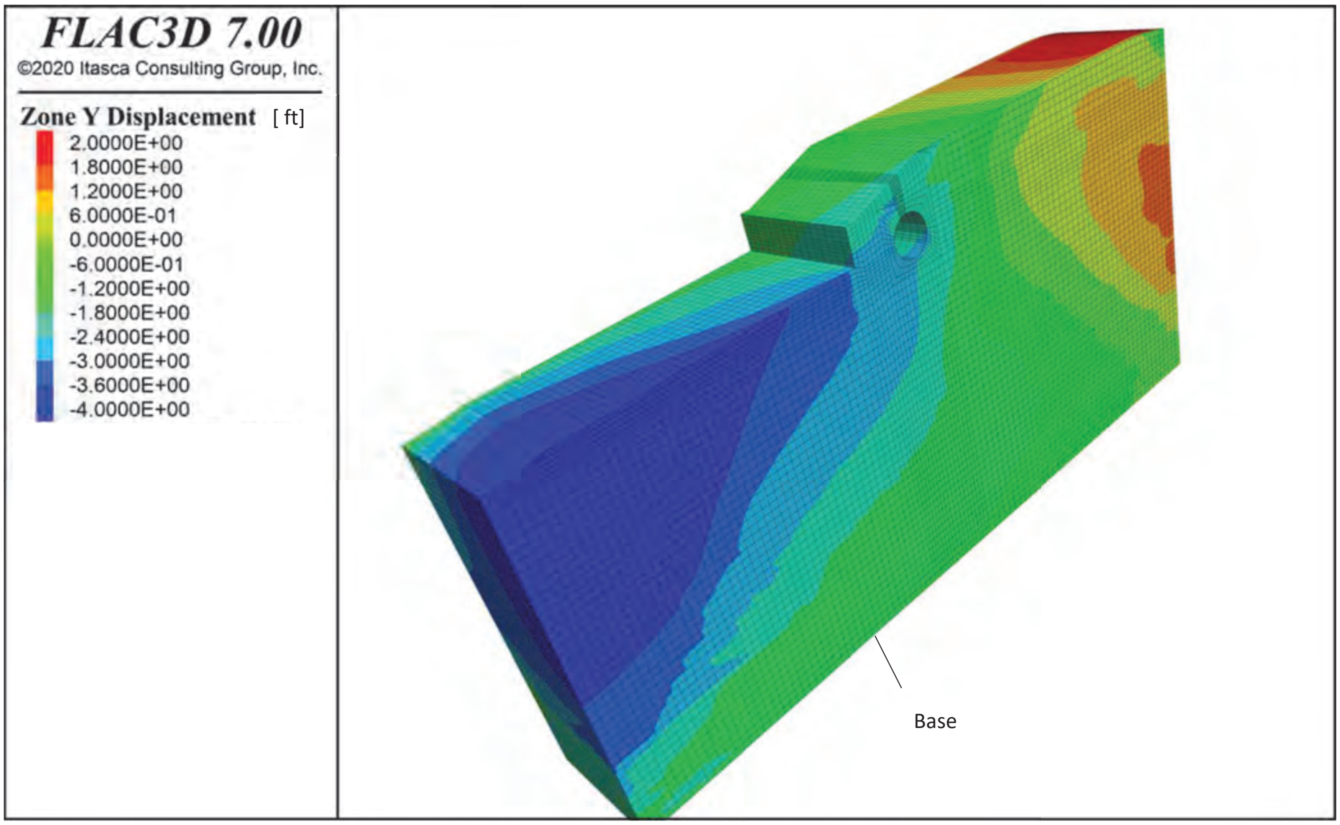


Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 35.0 SEC. - EARTHQUAKE SCENARIO A - EQR) AND TENSILE FORCE IN THE TIE-BACK SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-10



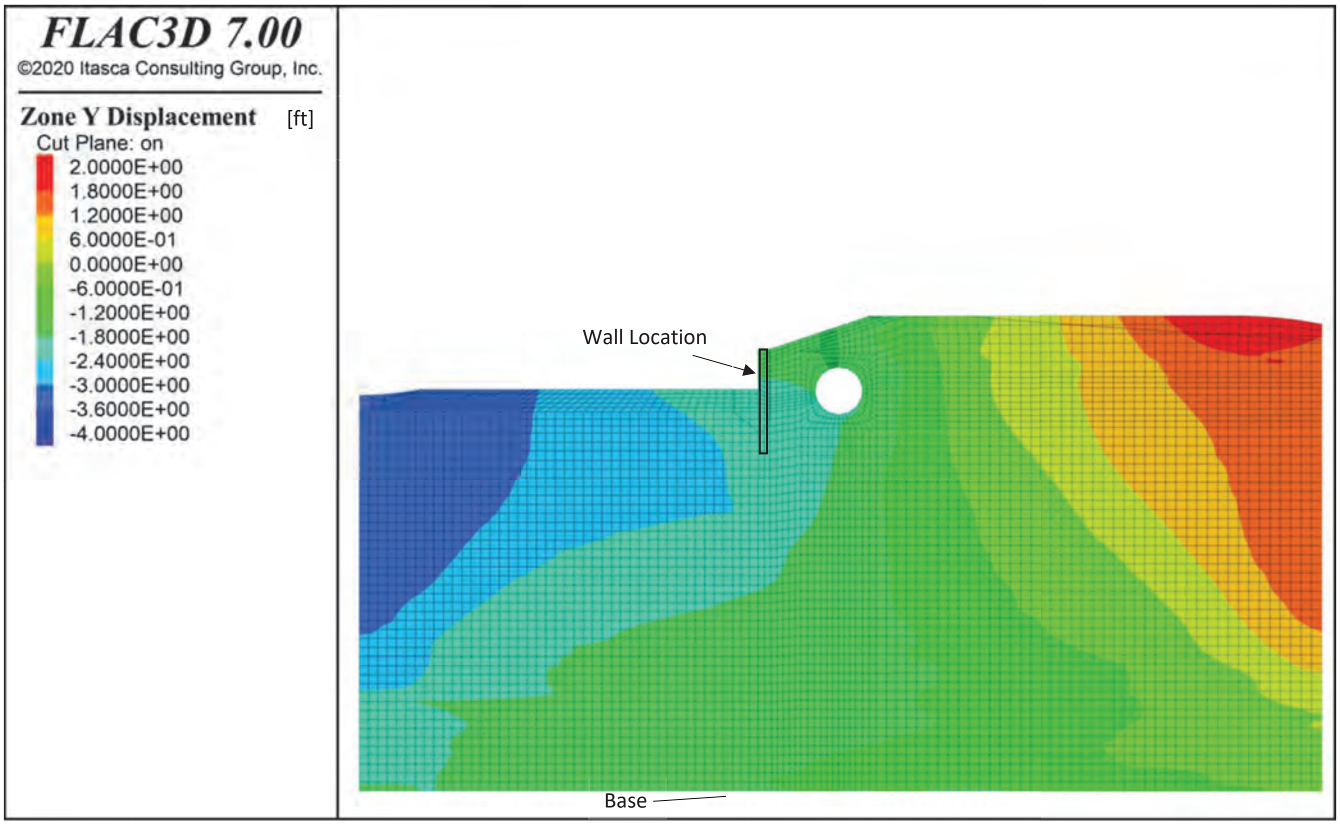
Note 1: Depicted displacements are only due to seismic simulation for the subject SCENARIO and are in addition to the gravitational loading. **Note 2:** dY and dZ are relative displacement (relative to bottom of the pile) in Y and Z (elevation) directions, respectively. **Note 3:** Displacements are exaggerated. Exaggeration factor in both Z and Y direction is 2. **Note 4:** Positive direction in Y direction is toward the slope (away from the ocean).	**TUNNEL AND PILE DISPLACEMENT** **(AT t = 35.0 SEC. - EARTHQUAKE SCENARIO A EQR)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS
	JOB NO. AGS-18-003 / DATE: JUNE 2021	PLATE C-10



RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION
(AT t = 40 SEC. - EARTHQUAKE SCENARIO B - EQR)
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA

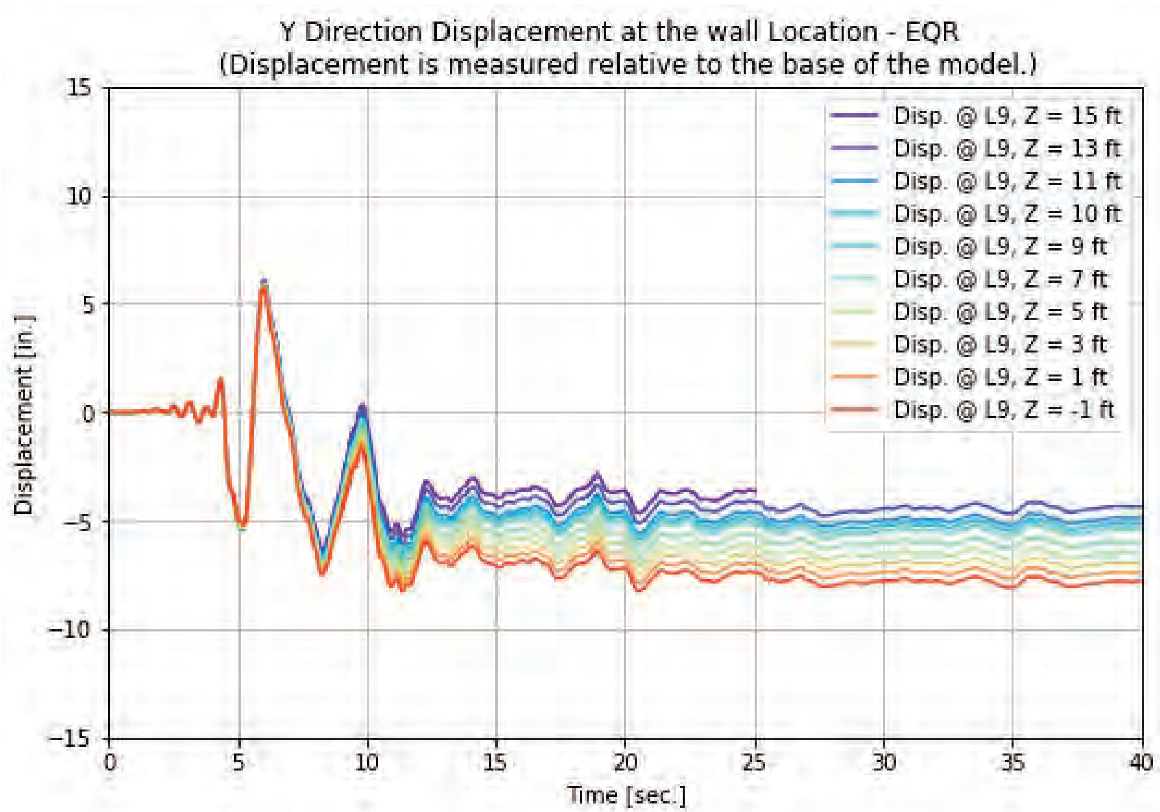
AGS

JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-13



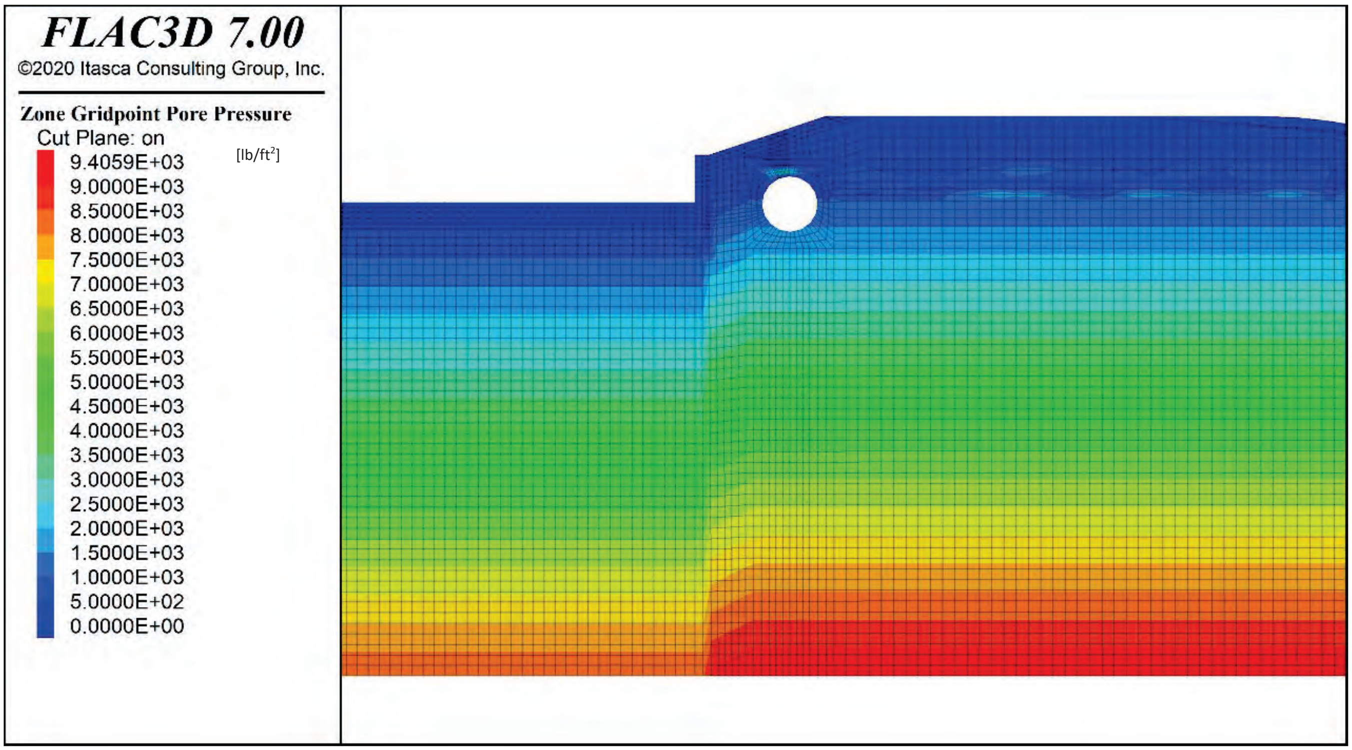
RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO B - EQR)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA	AGS

JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-14

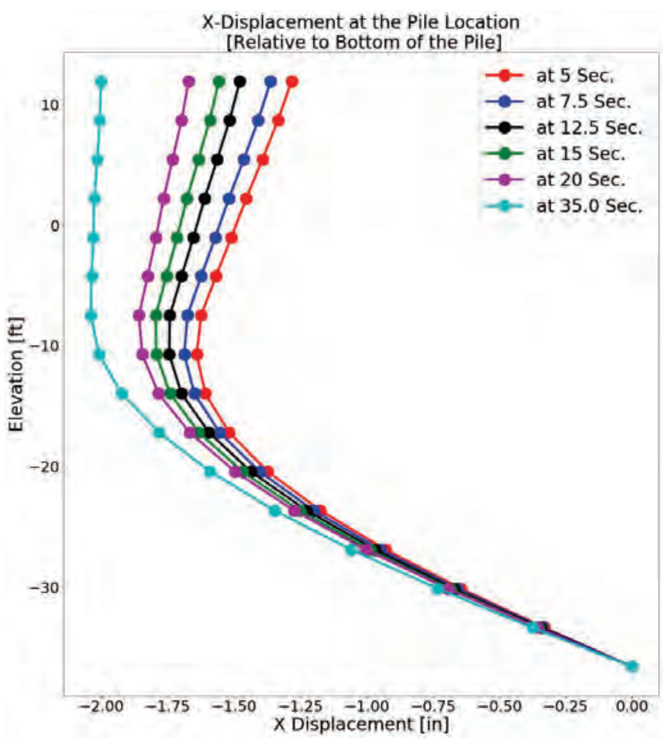


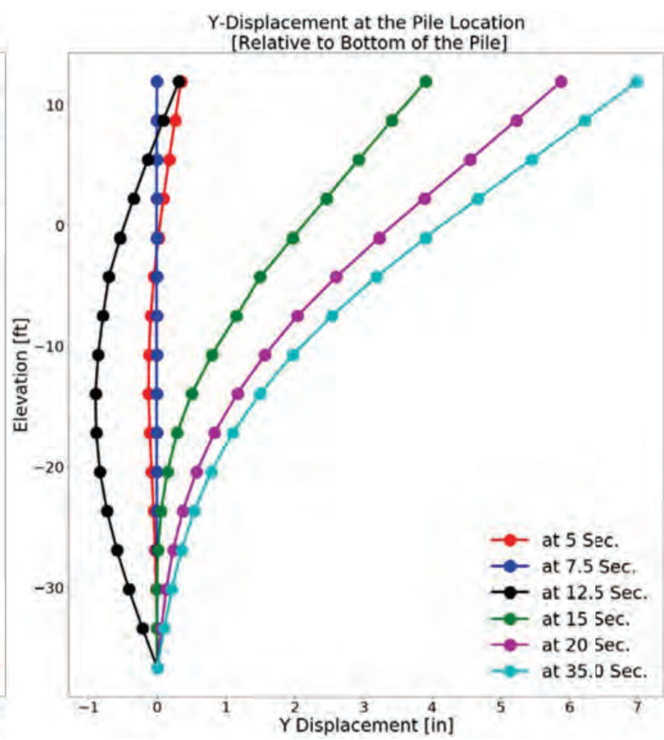
Note: Positive direction in Y direction is toward the slope (away from the ocean).

Y-DIRECTION DISPLACEMENT AT DIFFERENT ELEVATIONS OF THE WALL – MIDDLE SECTION (EARTHQUAKE SCENARIO B - EQR) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-15

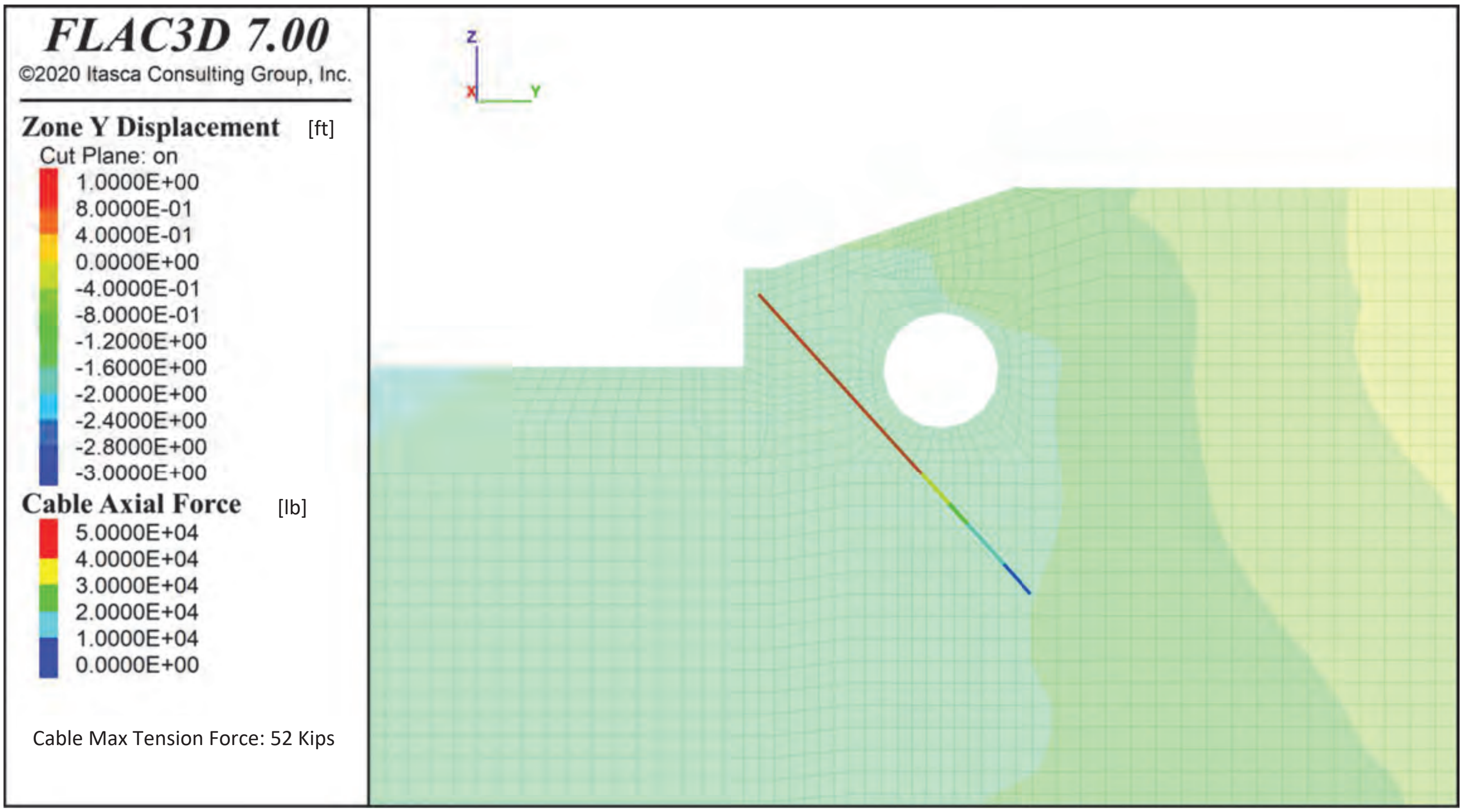


PORE WATER PRESSURE **(EARTHQUAKE SCENARIO-B AT t= 6.0 SEC. - EQR)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-16





Note 1: Positive direction in Y direction is toward the slope (away from the ocean). **Note 2**: Displacements are shown up to fixity depth of the pile.	**DISPLACEMENT IN THE PILE OF THE WALL (MIDDLE PILE)** **(EQR - EARTHQUAKE SCENARIO B - EQR)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA	AGS
	JOB NO. AGS-18-003 / DATE: JUNE 2021	PLATE C-17

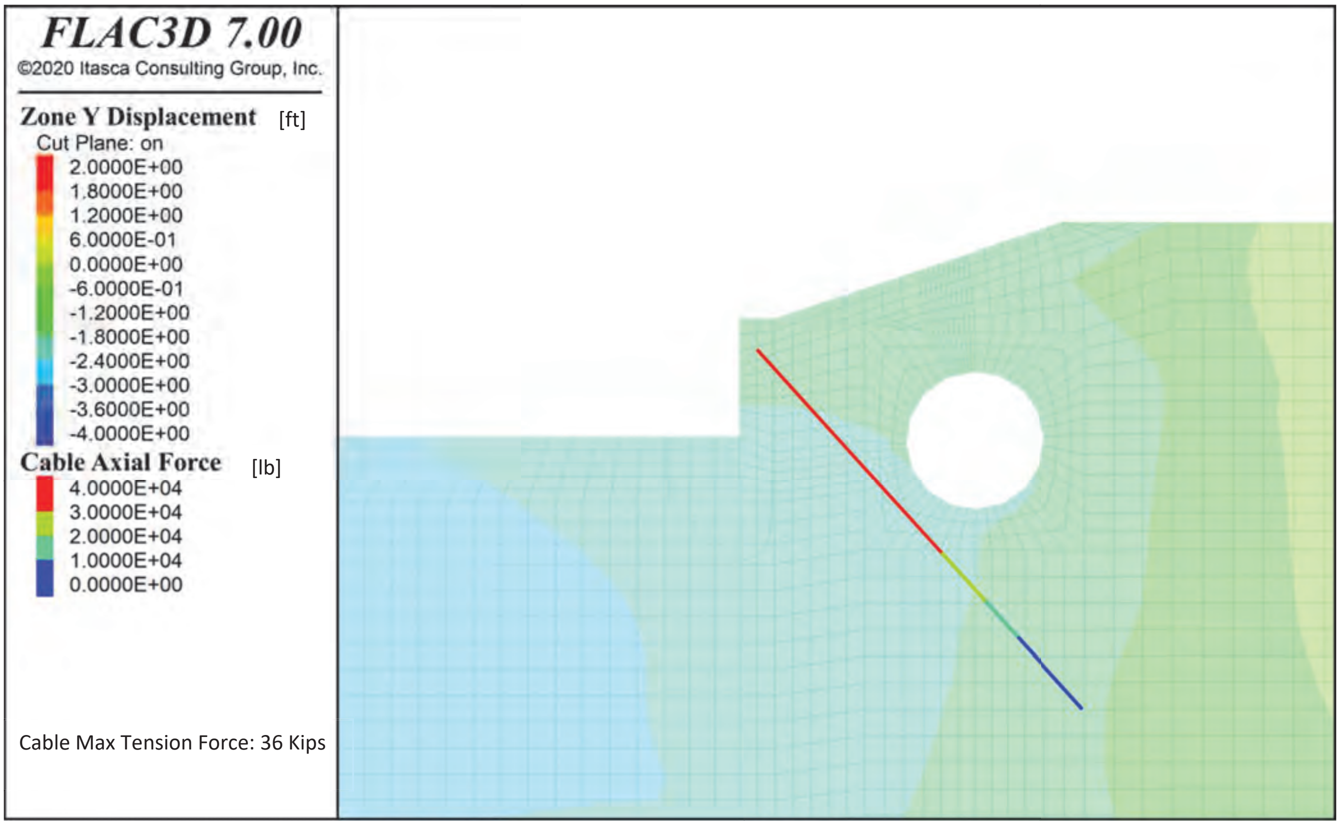


Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION
(AT t = 7.5 SEC. - EARTHQUAKE SCENARIO B - EQR)
AND TENSILE FORCE IN THE TIE-BACK
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA

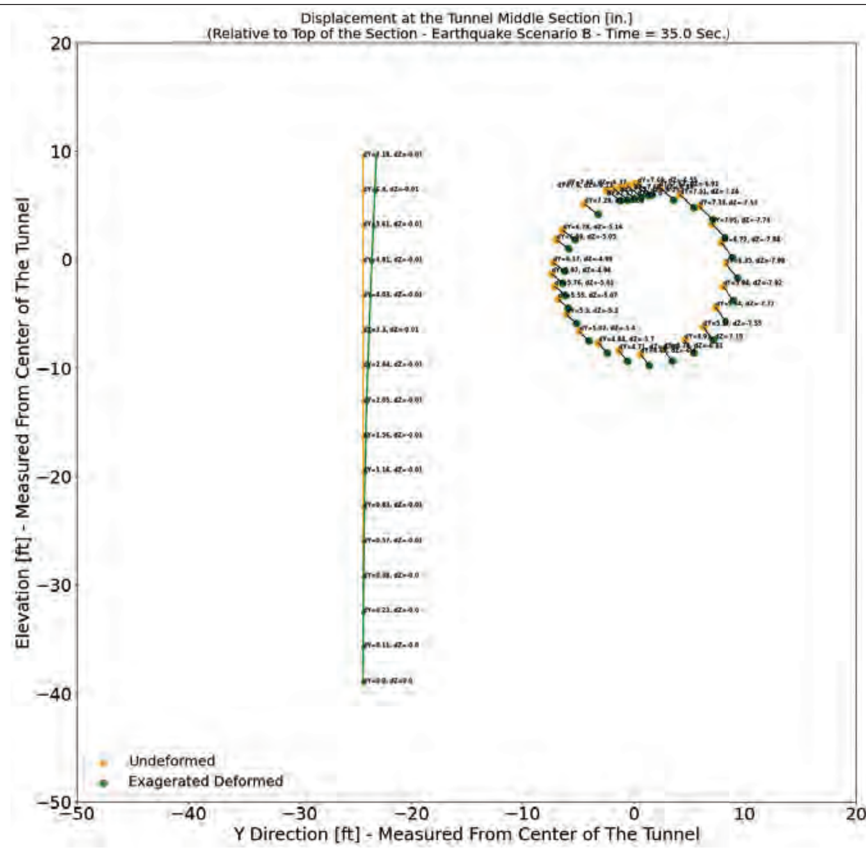
AGS

JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-18



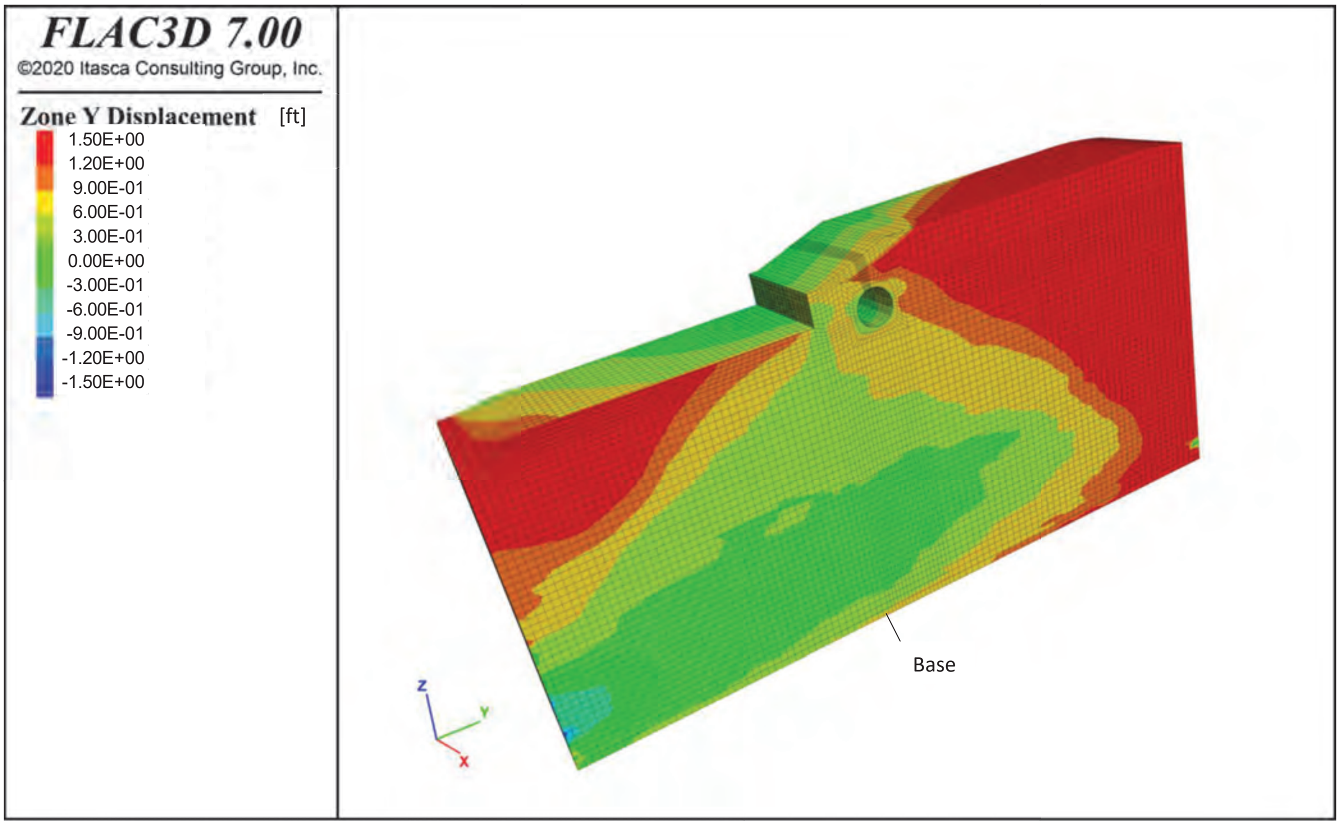
Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO B - EQR)** **AND TENSILE FORCE IN THE TIE-BACK** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-19

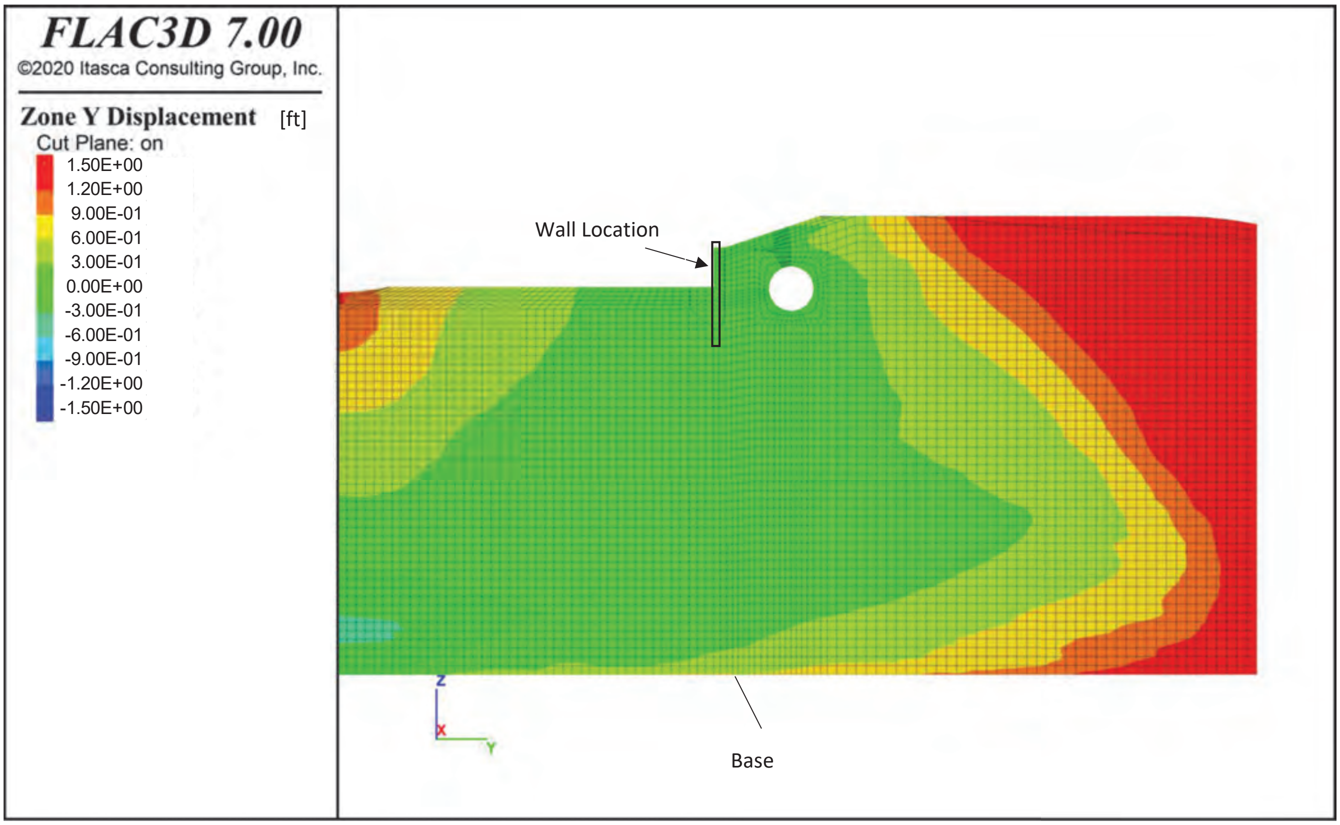


Note 1: Depicted displacements are only due to seismic simulation for the subject SCENARIO and are in addition to the gravitational loading.
Note 2: dY and dZ are relative displacement (relative to bottom of the pile) in Y and Z (elevation) directions, respectively.
Note 3: Displacements are exaggerated. Exaggeration factor in both Z and Y direction is 2.
Note 4: Positive direction in Y direction is toward the slope (away from the ocean).

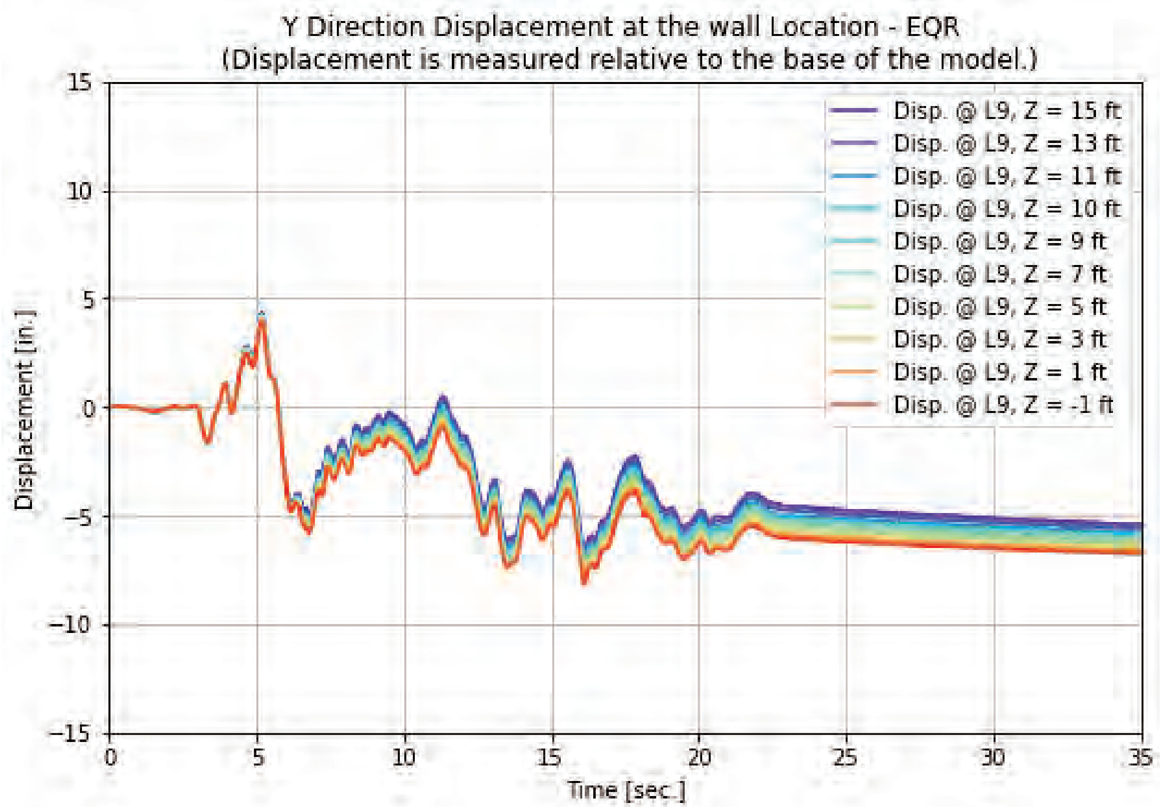
TUNNEL AND PILE DISPLACEMENT **(AT t = 35.0 SEC. - EARTHQUAKE SCENARIO B EQR)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-20



RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 40 SEC. - EARTHQUAKE SCENARIO C - EQR) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-21

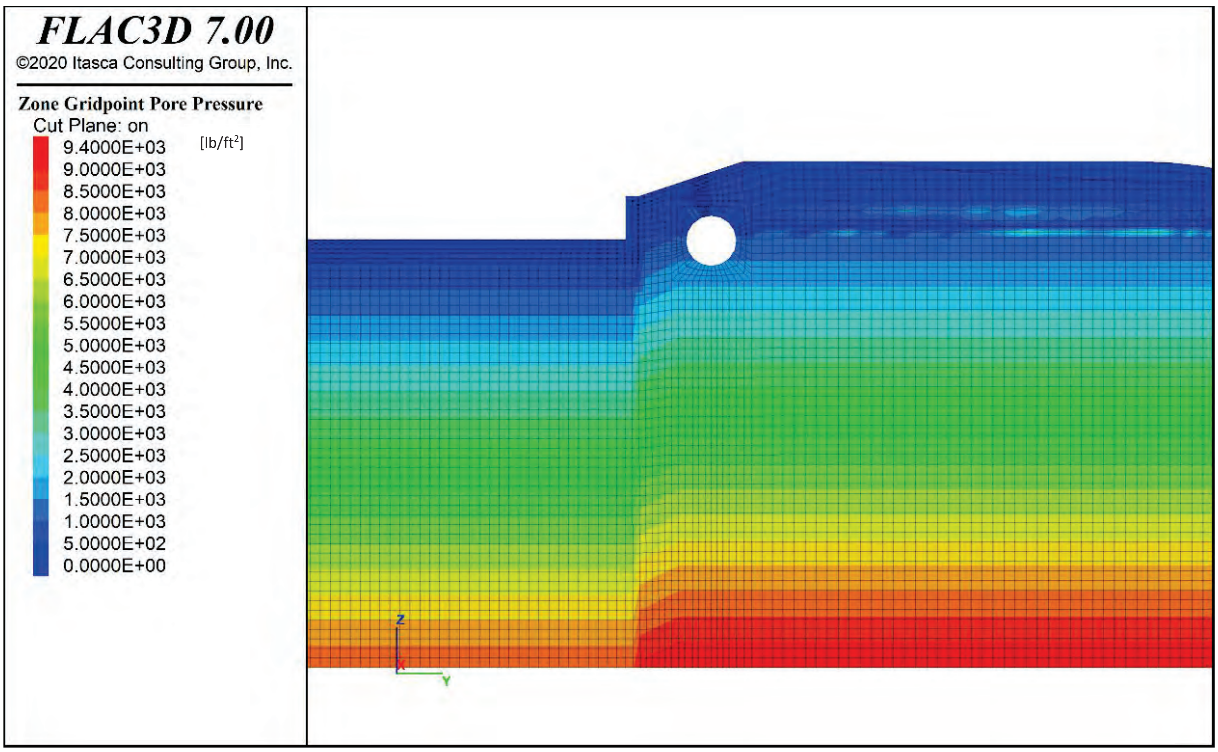


RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO C - EQR)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA	AGS
JOB NO. AGS-18-003 / DATE: JUNE 2021	PLATE C-22

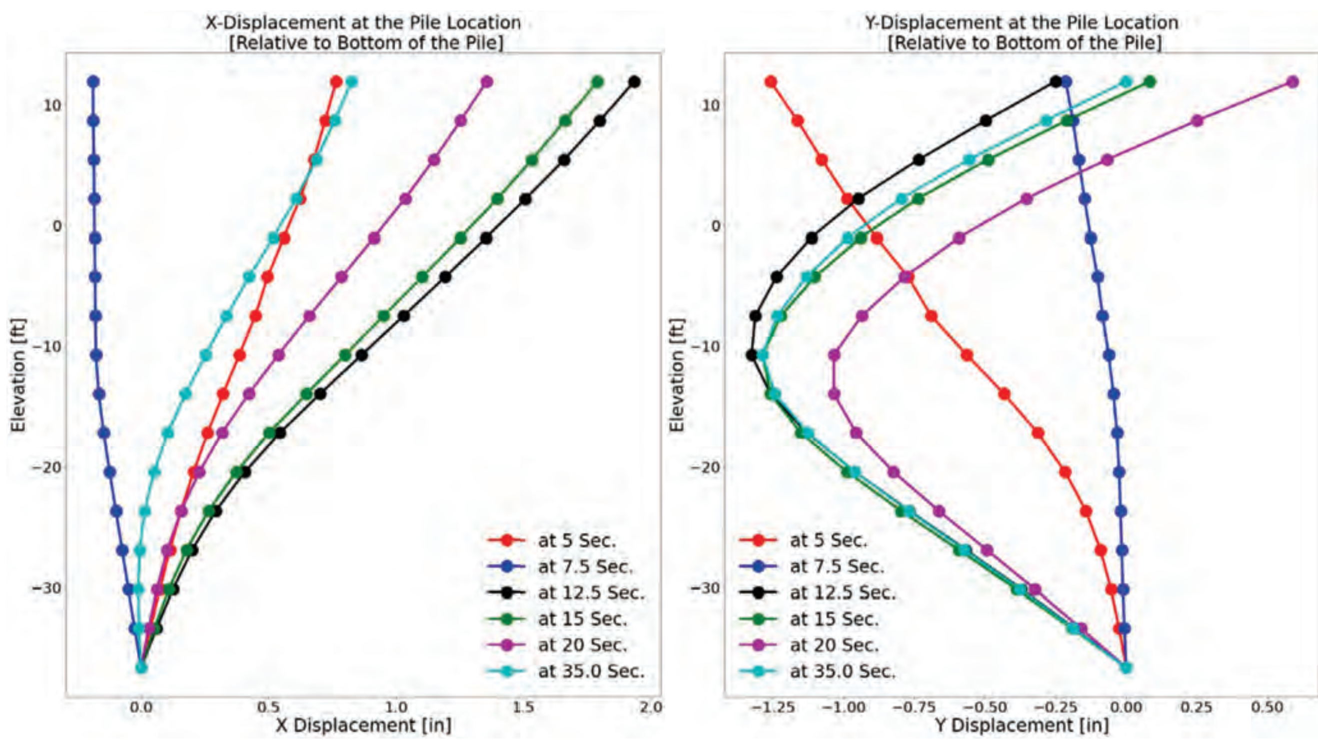


Note: Positive direction in Y direction is toward the slope (away from the ocean).

Y-DIRECTION DISPLACEMENT AT DIFFERENT ELEVATIONS OF THE WALL – MIDDLE SECTION **(EARTHQUAKE SCENARIO C - EQR)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-23

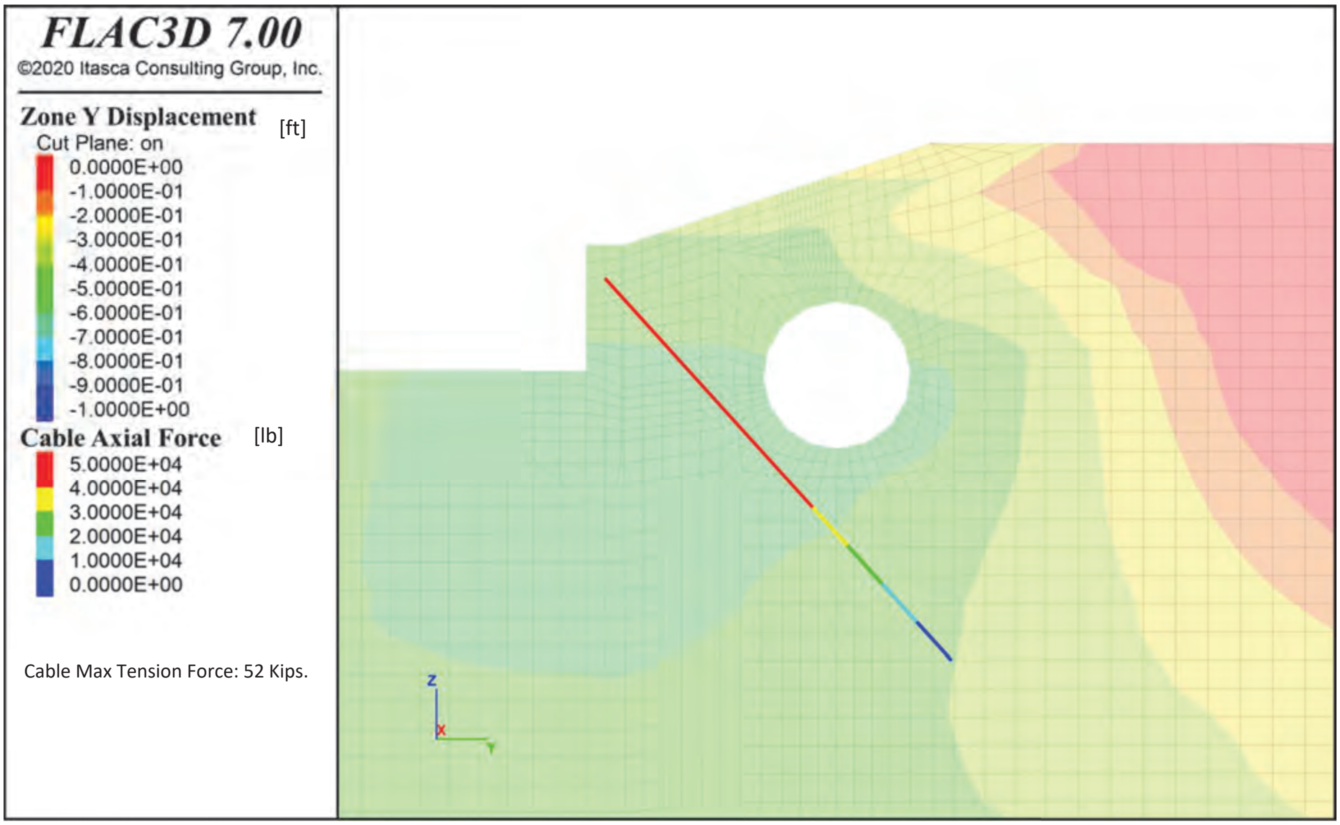


	PORE WATER PRESSURE **(EARTHQUAKE SCENARIO-C AT t= 4.0 SEC- EQR)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA	AGS
	JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-24



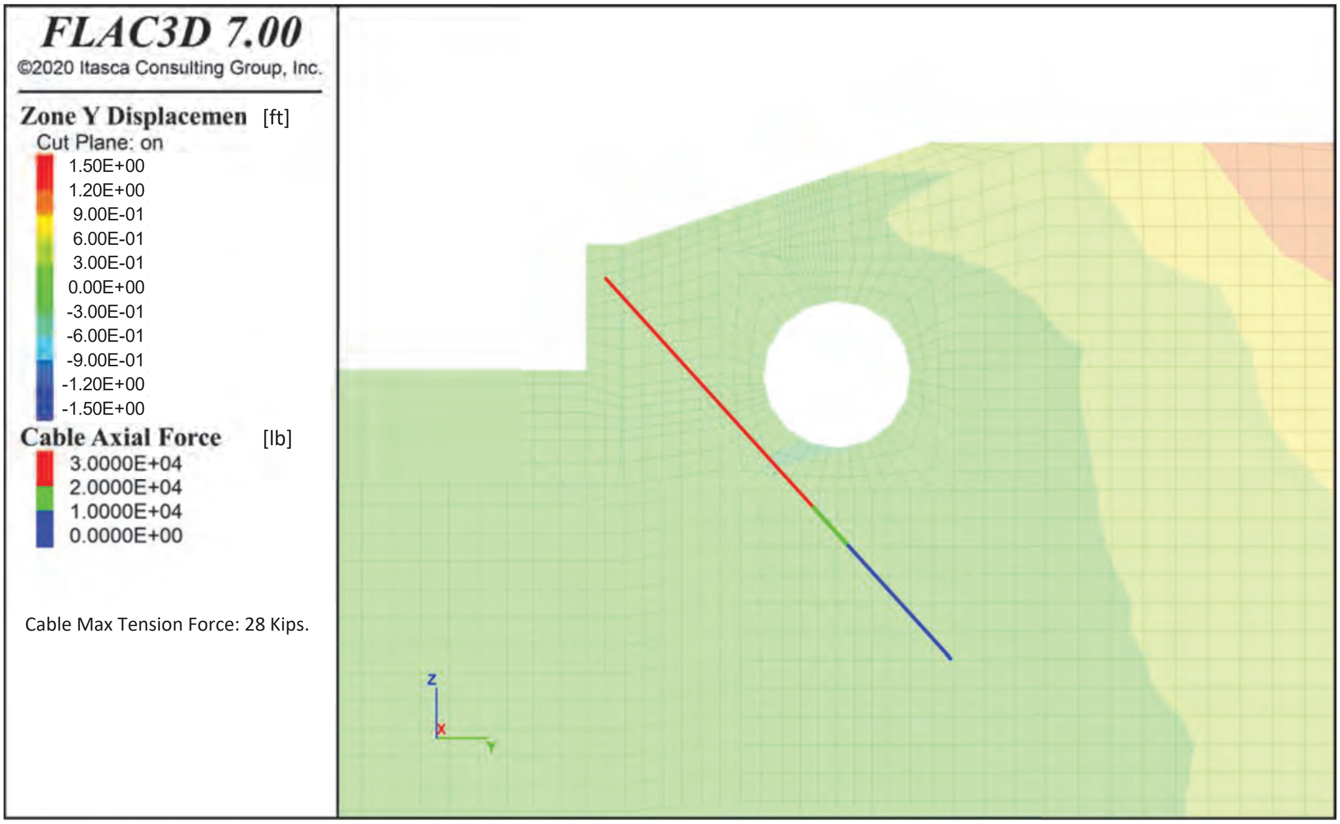
Note 1: Positive direction in Y direction is toward the slope (away from the ocean).
Note 2: Displacements are shown up to fixity depth of the pile.

DISPLACEMENT IN THE PILE OF THE WALL (MIDDLE PILE) (EQR - EARTHQUAKE SCENARIO C - EQR) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-25



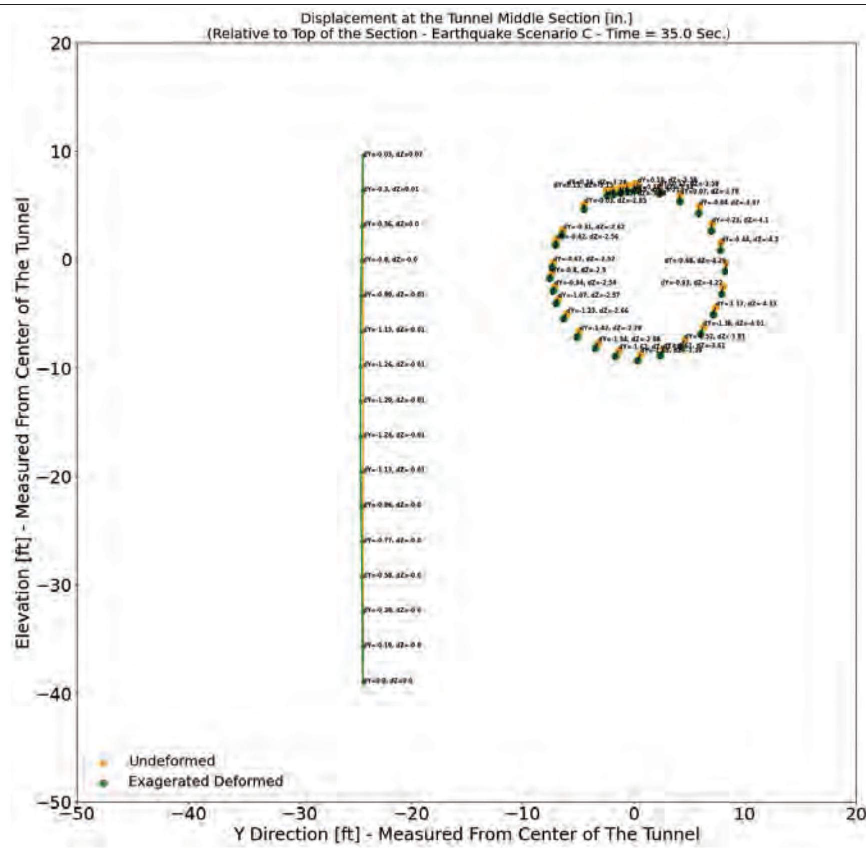
Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 12.0 SEC. - EARTHQUAKE SCENARIO C - EQR) AND TENSILE FORCE IN THE TIE-BACK SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-26



Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 35.0 SEC. - EARTHQUAKE SCENARIO C - EQR) AND TENSILE FORCE IN THE TIE-BACK SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-27

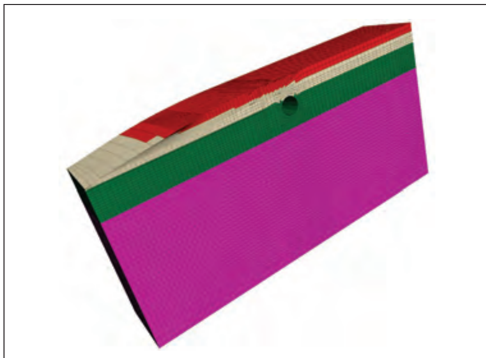


Note 1: Depicted displacements are only due to seismic simulation for the subject SCENARIO and are in addition to the gravitational loading.
Note 2: dY and dZ are relative displacement (relative to bottom of the pile) in Y and Z (elevation) directions, respectively.
Note 3: Displacements are exaggerated. Exaggeration factor in both Z and Y direction is 2.
Note 4: Positive direction in Y direction is toward the slope (away from the ocean).

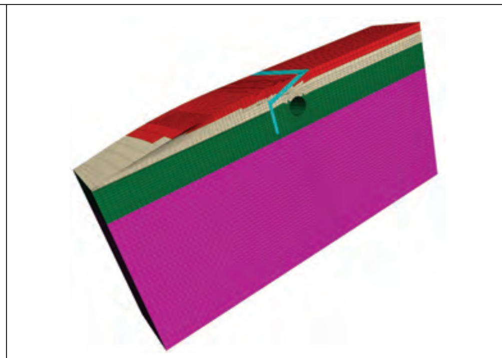
TUNNEL AND PILE DISPLACEMENT **(AT t = 35.0 SEC. - EARTHQUAKE SCENARIO C EQR)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE C-28

Appendix D

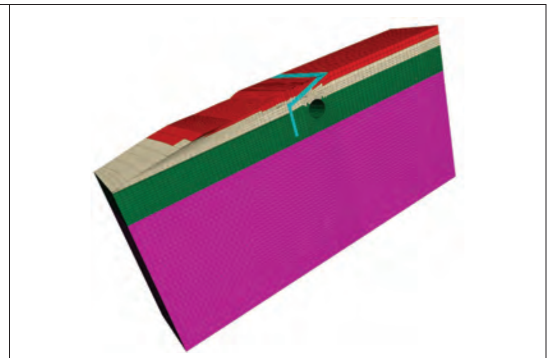
FLAC3D NUMERICAL SSSI DYNAMIC ANALYSES RESULTS FOR RUBBLE REACH



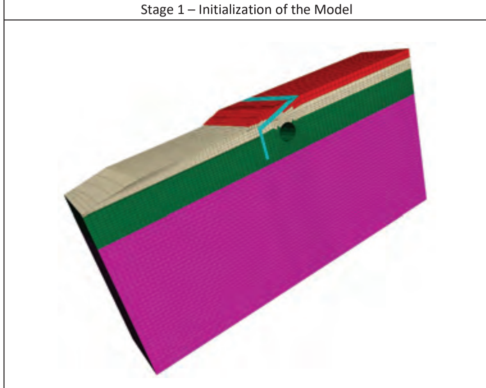
Stage 1 – Initialization of the Model



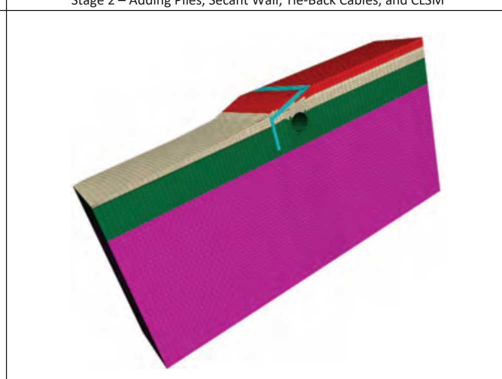
Stage 2 – Adding Piles, Secant Wall, Tie-Back Cables, and CLSM



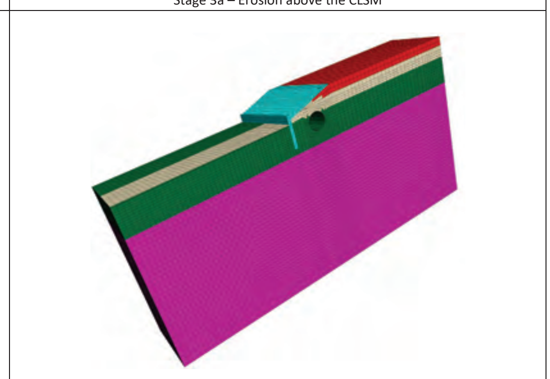
Stage 3a – Erosion above the CLSM



Stage 3b – Erosion above the CLSM



Stage 3c – Erosion above the CLSM

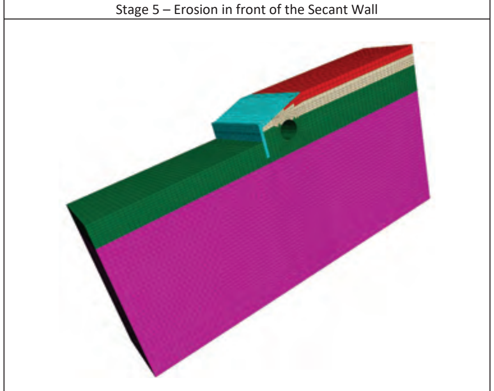


Stage 4 – Erosion in front of the Secant Wall

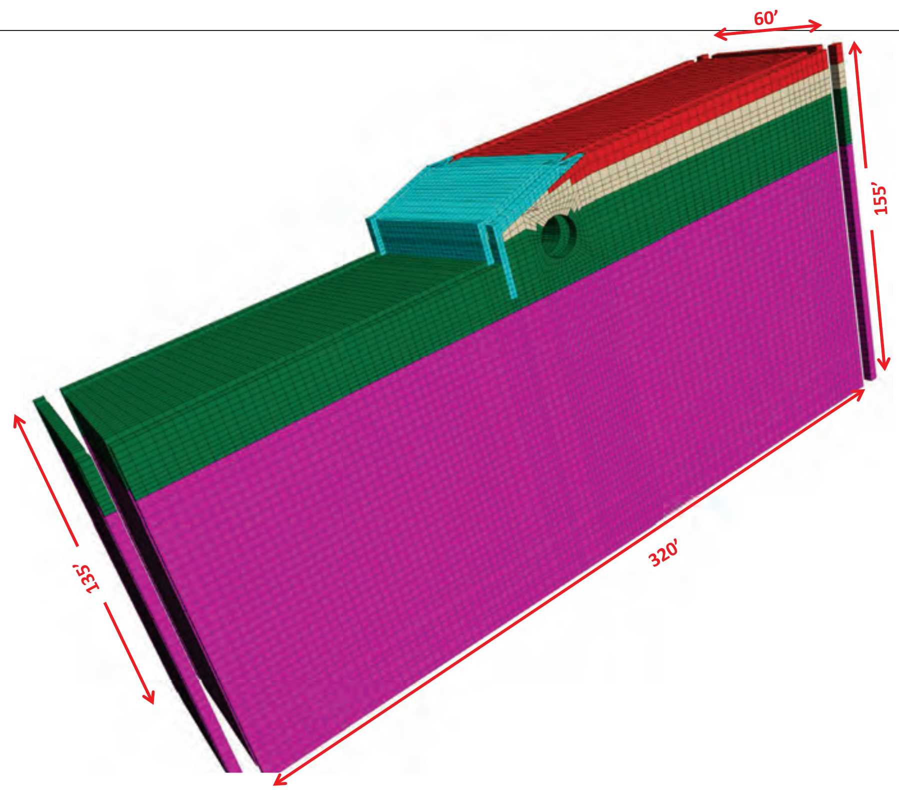
CLSM and Wall	Liquefiable Layer
Fill	Colma Formation
	Merced Formation

RUBBLE REACH MESH – DIFFERENT STAGES OF THE NUMERICAL SIMULATION SOUTH OCEAN BEACH C OASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE D-1 a

Stage 5 – Erosion in front of the Secant Wall



Stage 6 – Erosion in front of the Secant Wall



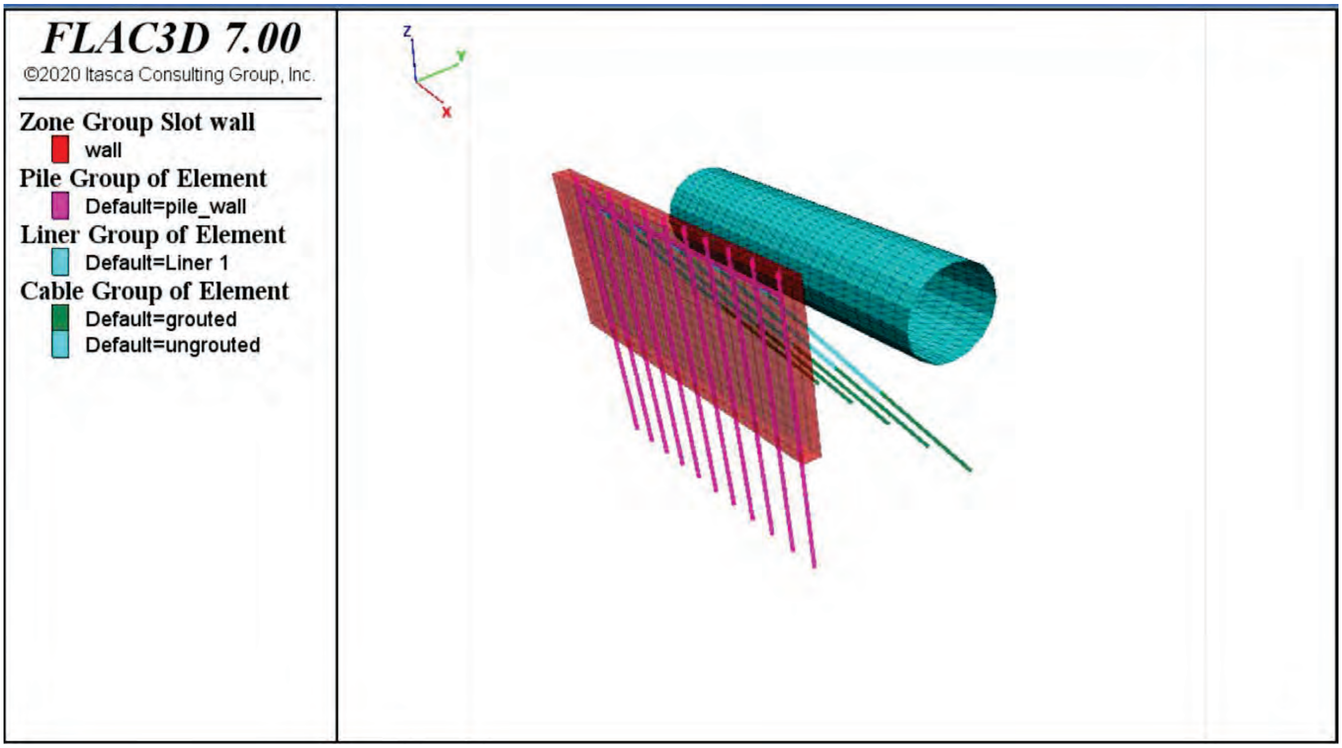
Mesh Used to Seismic Analysis – Includes Free Field Boundary Conditions

CLSM and Wall	Liquefiable Layer
Fill	Colma Formation
	Merced Formation

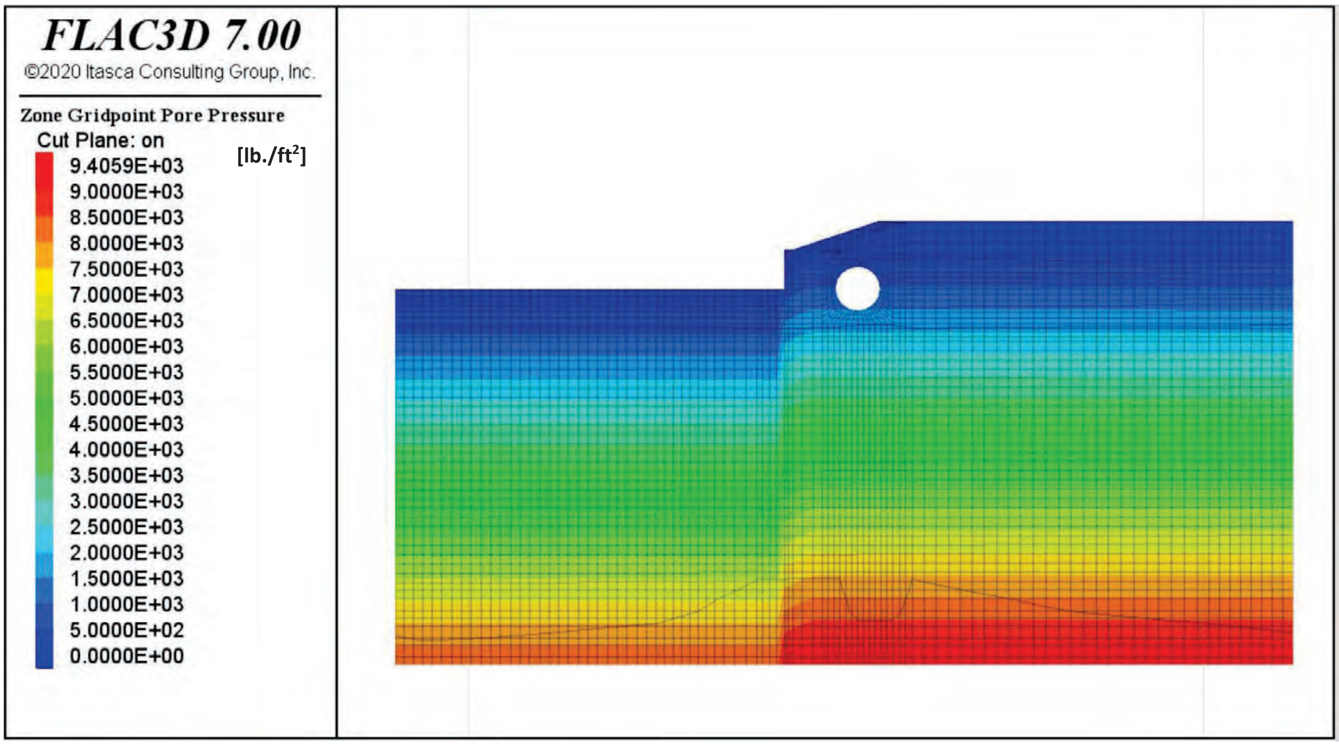
RUBBLE REACH MESH – DIFFERENT STAGES OF THE NUMERICAL SIMULATION
SOUTH OCEAN BEACH C OASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE D-1 b

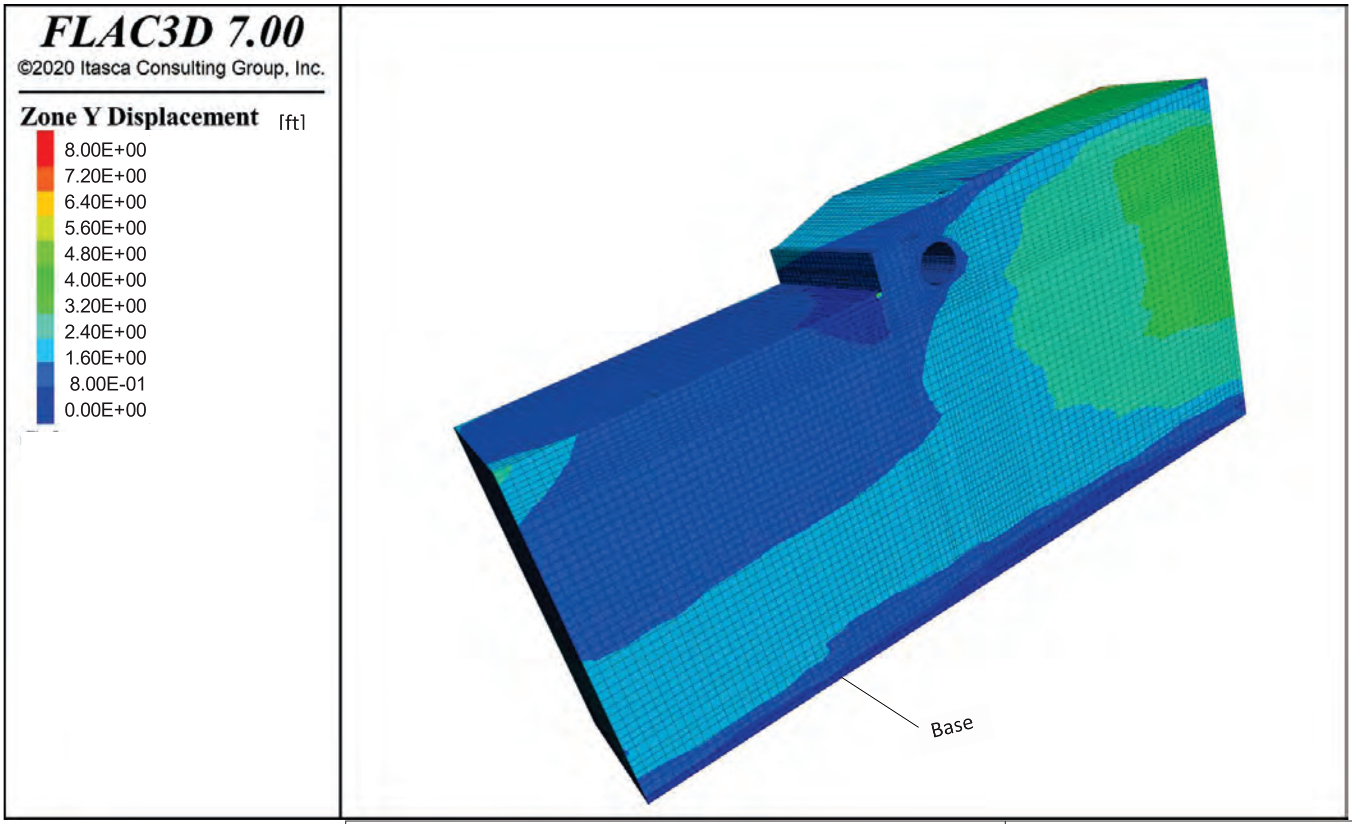


STRUCTURAL ELEMENTS OF THE MODEL **(EARTHQUAKE SCENARIO A - RUBBLE REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS
JOB NO. AGS-18-003	DATE: JUNE 2021
PLATE D-2	

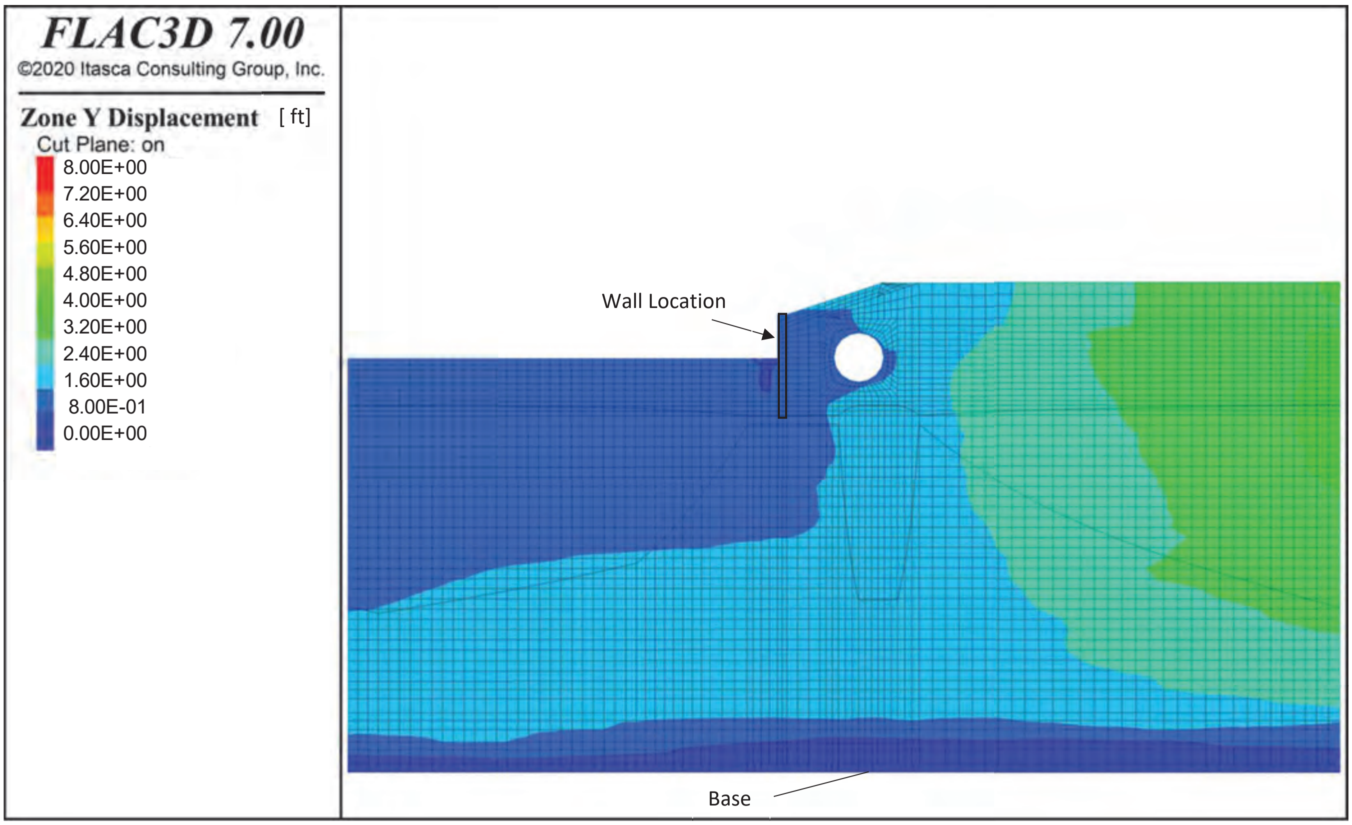


HYDROSTATIC PORE-WATER PRESSURE PRIOR TO SHAKING (RUBBLE REACH) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS

JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-3

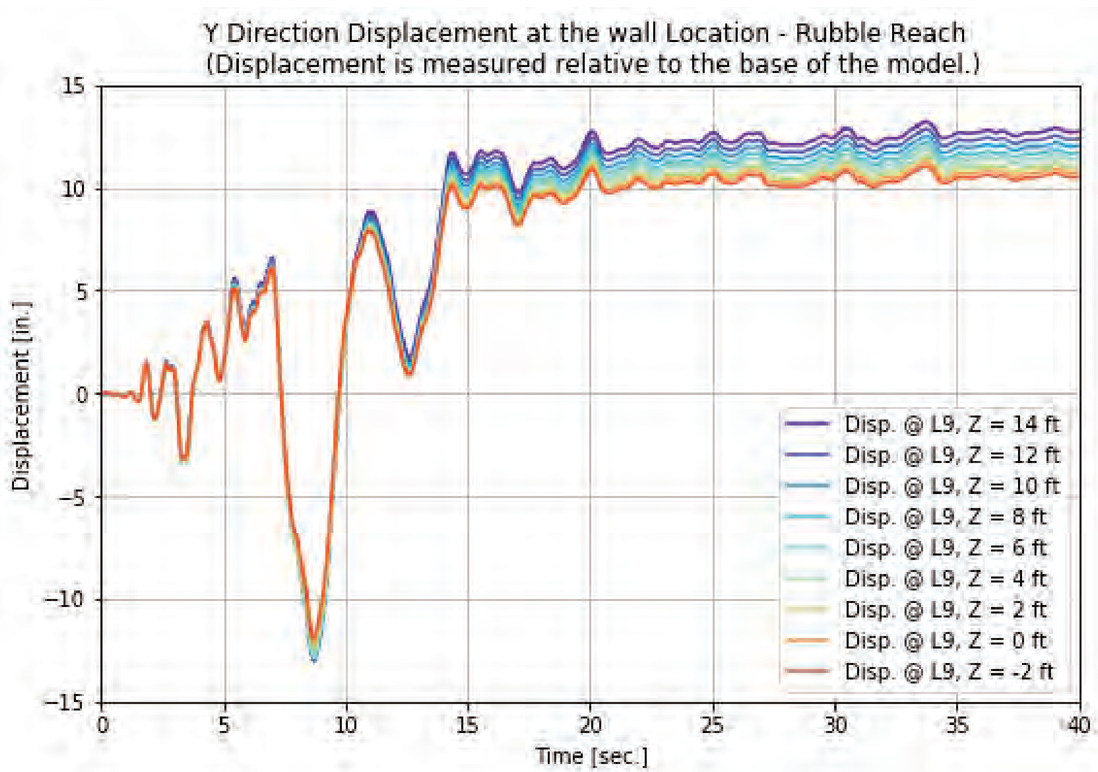


RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO A - RUBBLE REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-4



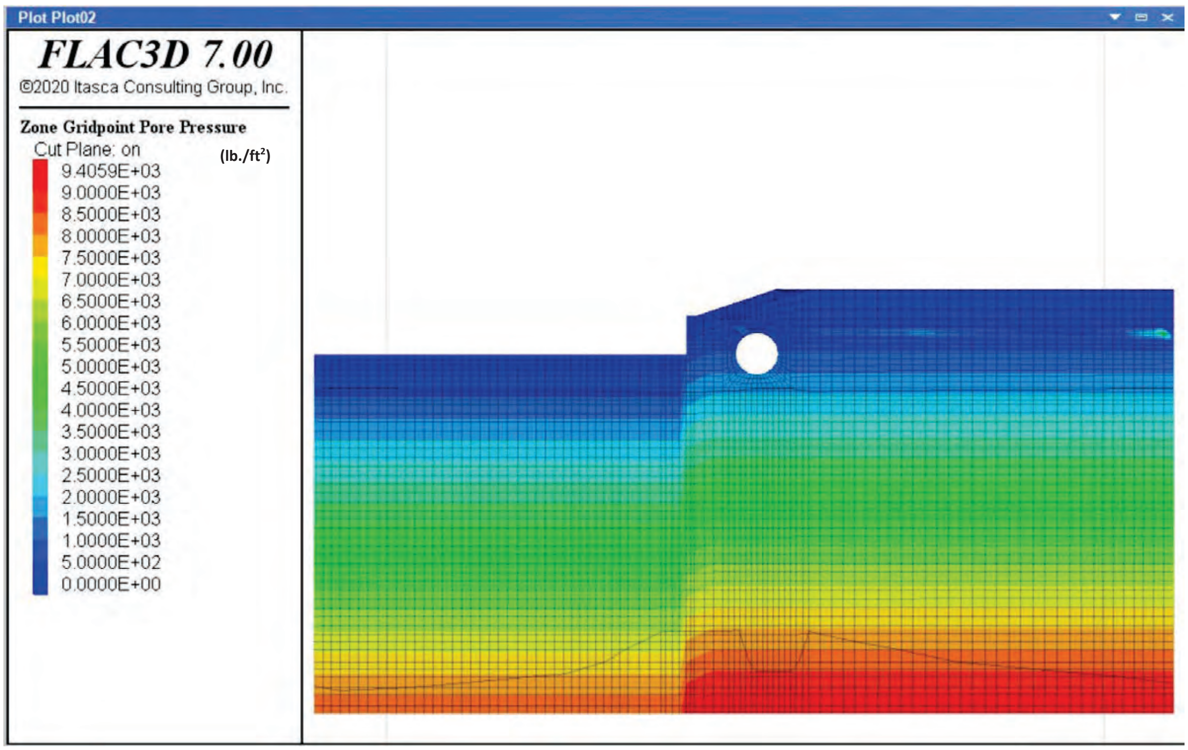
RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO A - RUBBLE REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS

JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-5

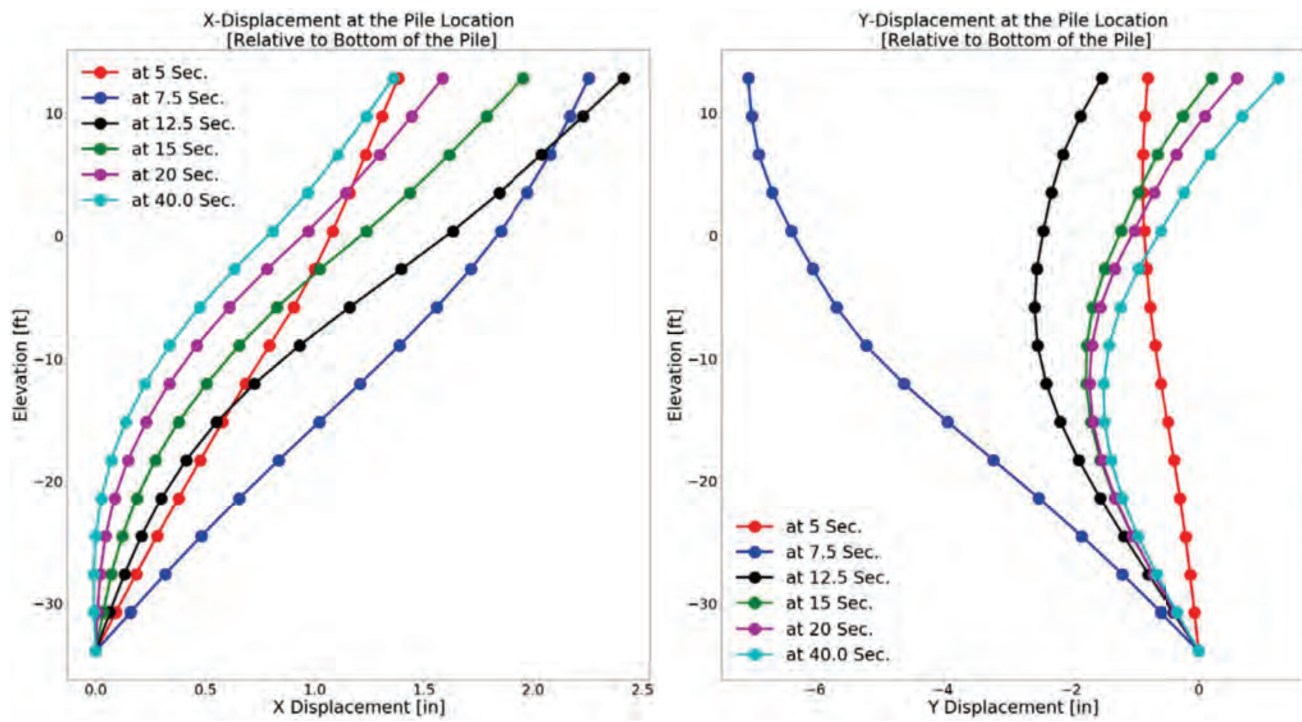


Note: Positive direction in Y direction is toward the slope (away from the ocean).

Y-DIRECTION DISPLACEMENT AT DIFFERENT ELEVATIONS OF THE WALL – MIDDLE SECTION **(EARTHQUAKE SCENARIO A - RUBBLE REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-6

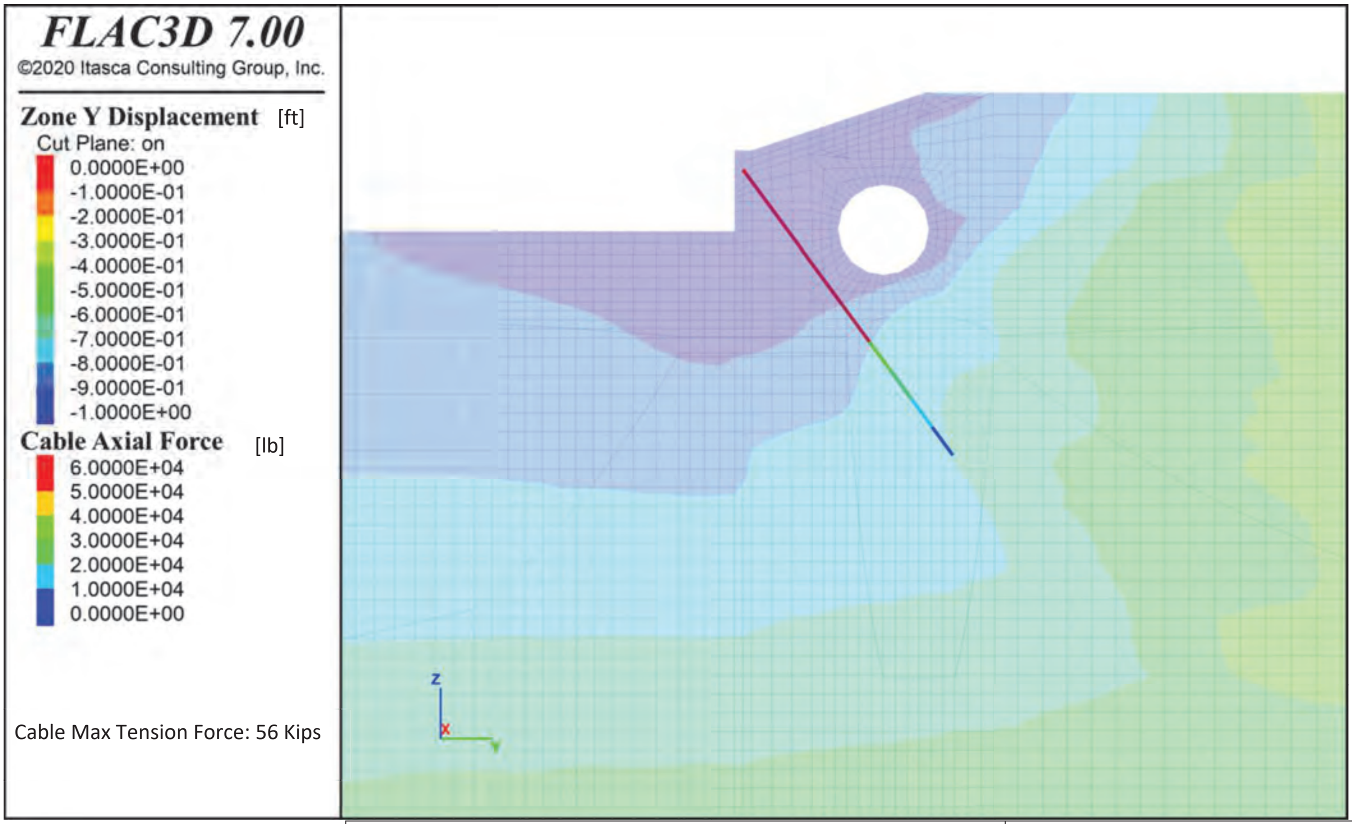


PORE WATER PRESSURE **(EARTHQUAKE SCENARIO-A AT t= 7.4 SEC- RUBBLE REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-7



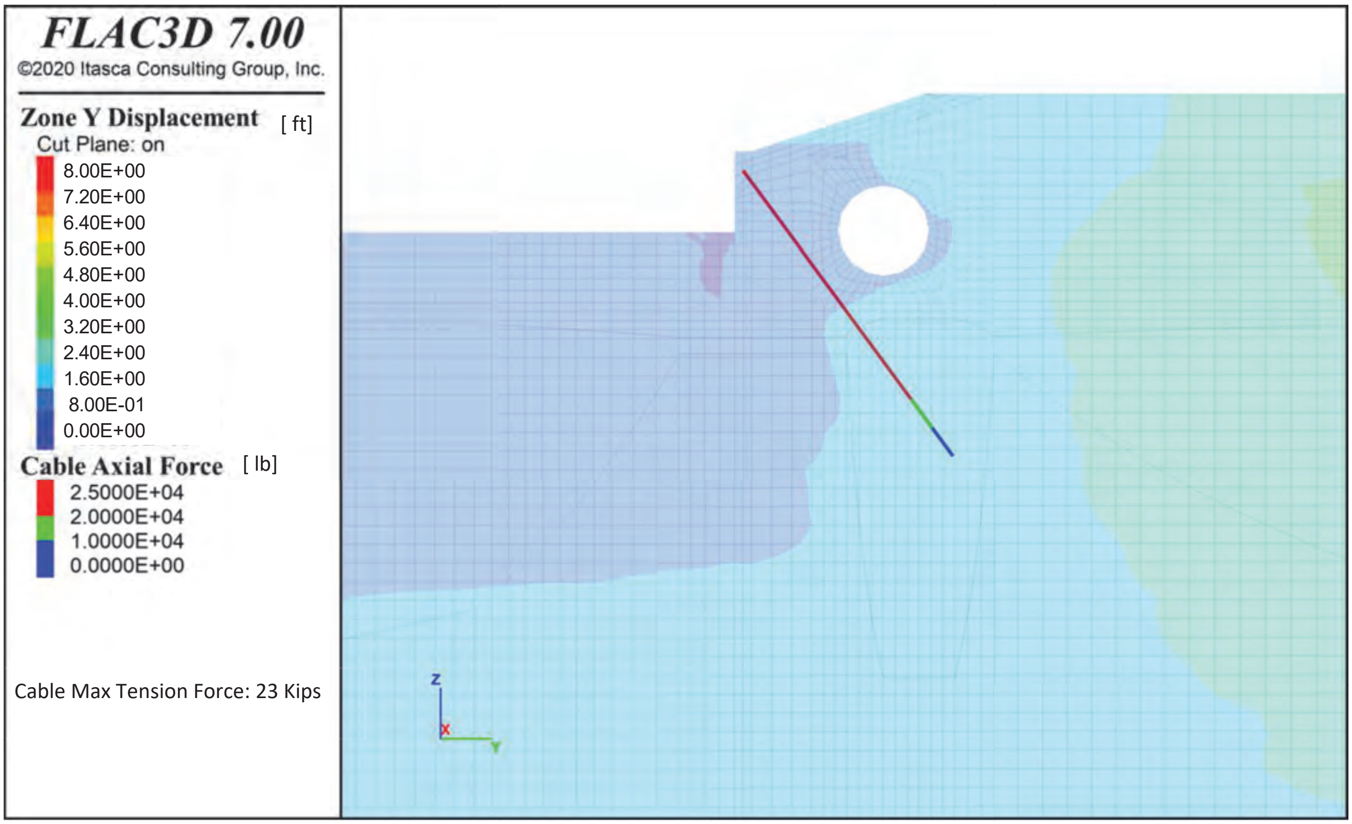
Note 1: Positive direction in Y direction is toward the slope (away from the ocean).
Note 2: Displacements are shown up to fixity depth of the pile.

DISPLACEMENT IN THE PILE OF THE WALL (MIDDLE PILE) (RUBBLE REACH - EARTHQUAKE SCENARIO A - RUBBLE REACH) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-8



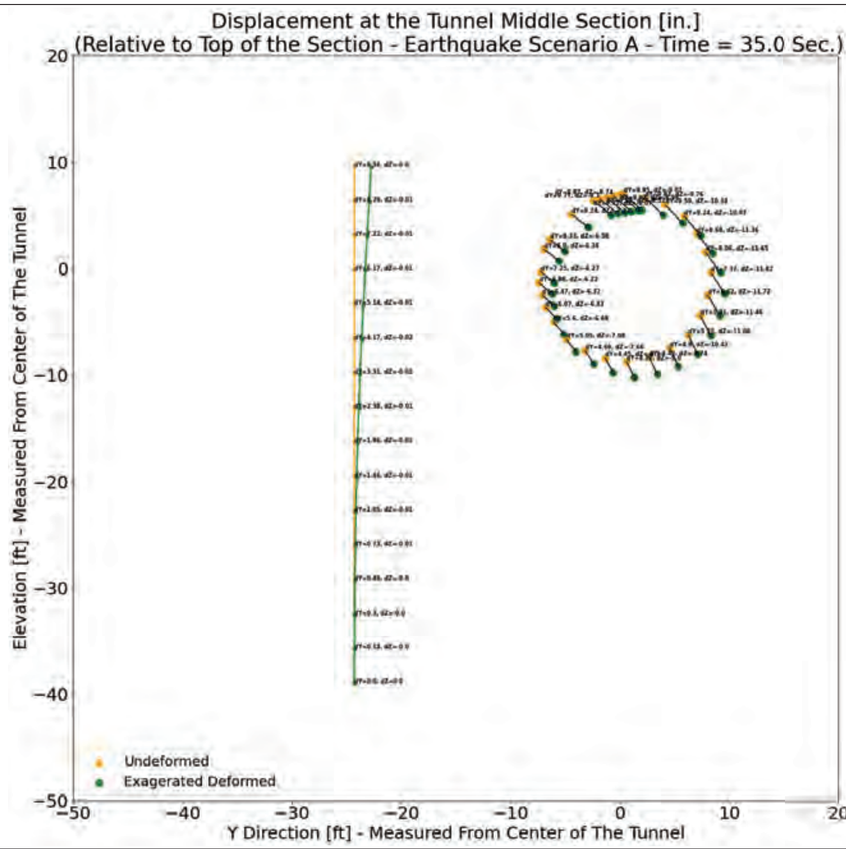
Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 7.4 SEC. - EARTHQUAKE SCENARIO A - RUBBLE REACH) AND TENSILE FORCE IN THE TIE-BACK SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-9

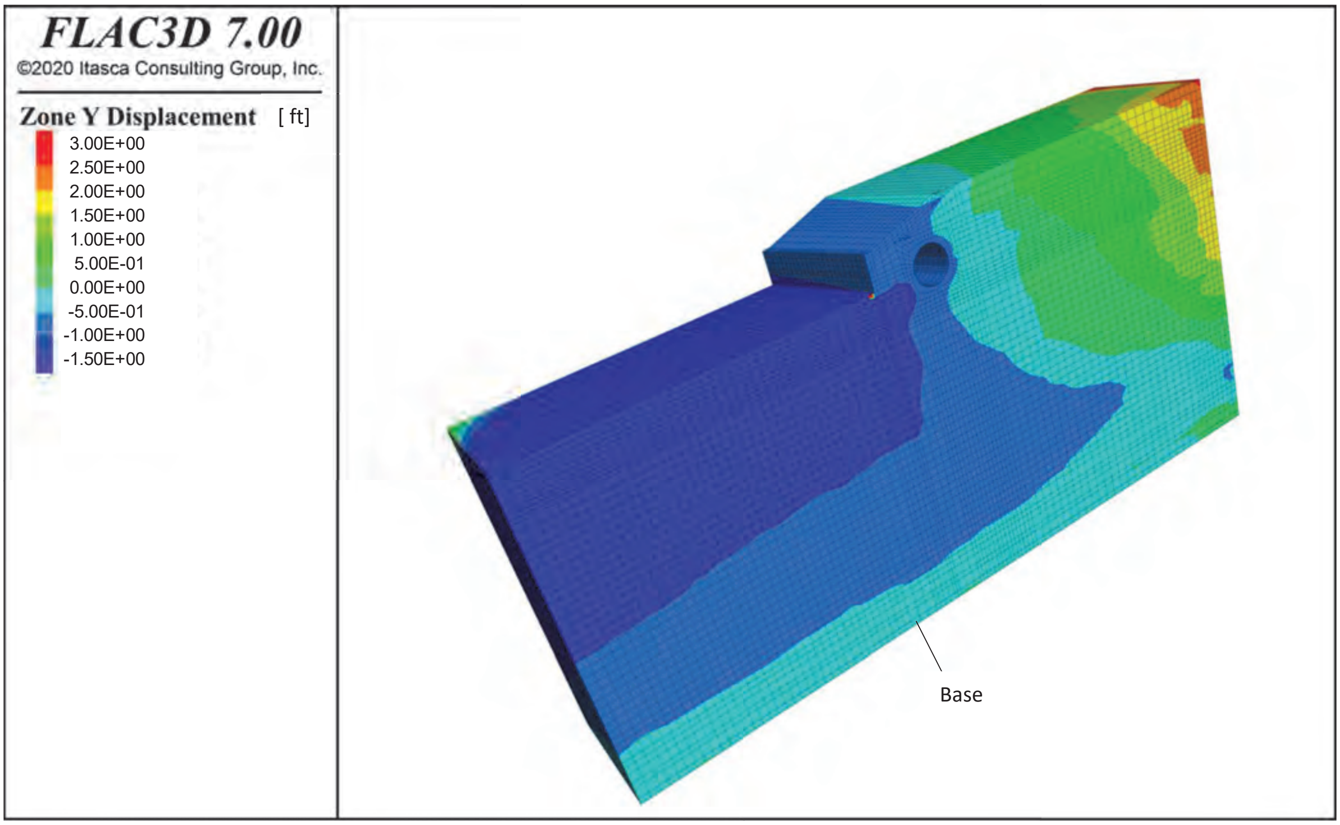


Note: Y Displacement contours shown in transparent.

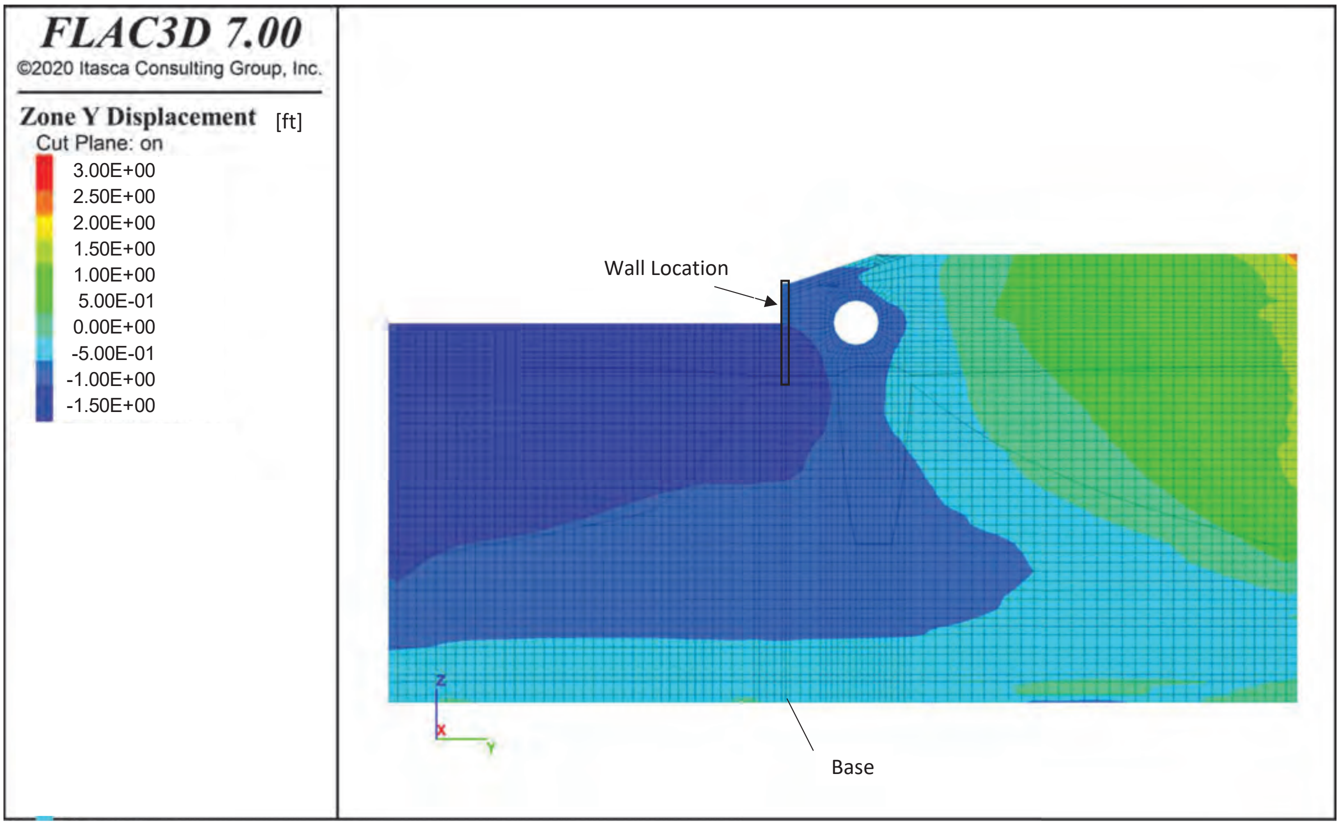
RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 40.0 SEC. - EARTHQUAKE SCENARIO A - RUBBLE REACH) AND TENSILE FORCE IN THE TIE-BACK SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-10



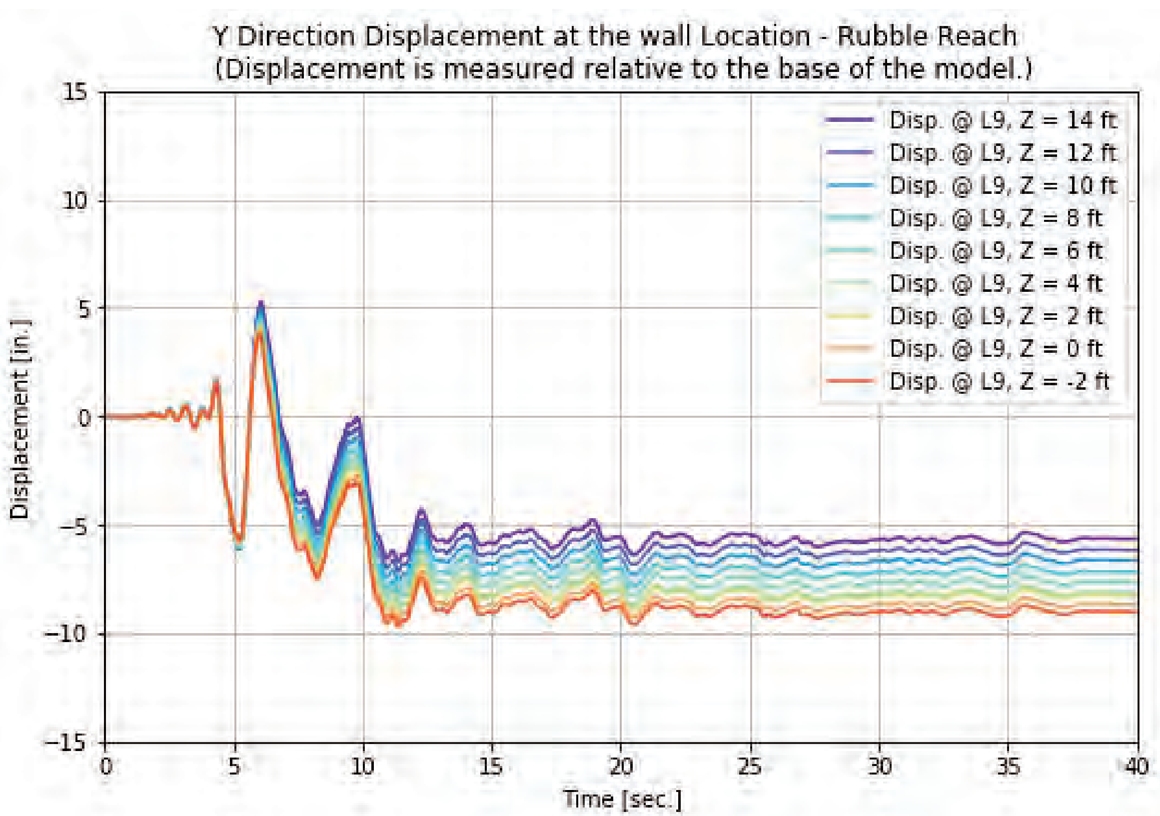
Note 1: Depicted displacements are only due to seismic simulation for the subject SCENARIO and are in addition to the gravitational loading. **Note 2:** dY and dZ are relative displacement (relative to bottom of the pile) in Y and Z (elevation) directions, respectively. **Note 3:** Displacements are exaggerated. Exaggeration factor in both Z and Y direction is 2. **Note 4:** Positive direction in Y direction is toward the slope (away from the ocean).	**TUNNEL AND PILE DISPLACEMENT** **(AT t = 40.0 SEC. - EARTHQUAKE SCENARIO A RUBBLE REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
	JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-10



RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 40 SEC. - EARTHQUAKE SCENARIO B - RUBBLE REACH) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-13

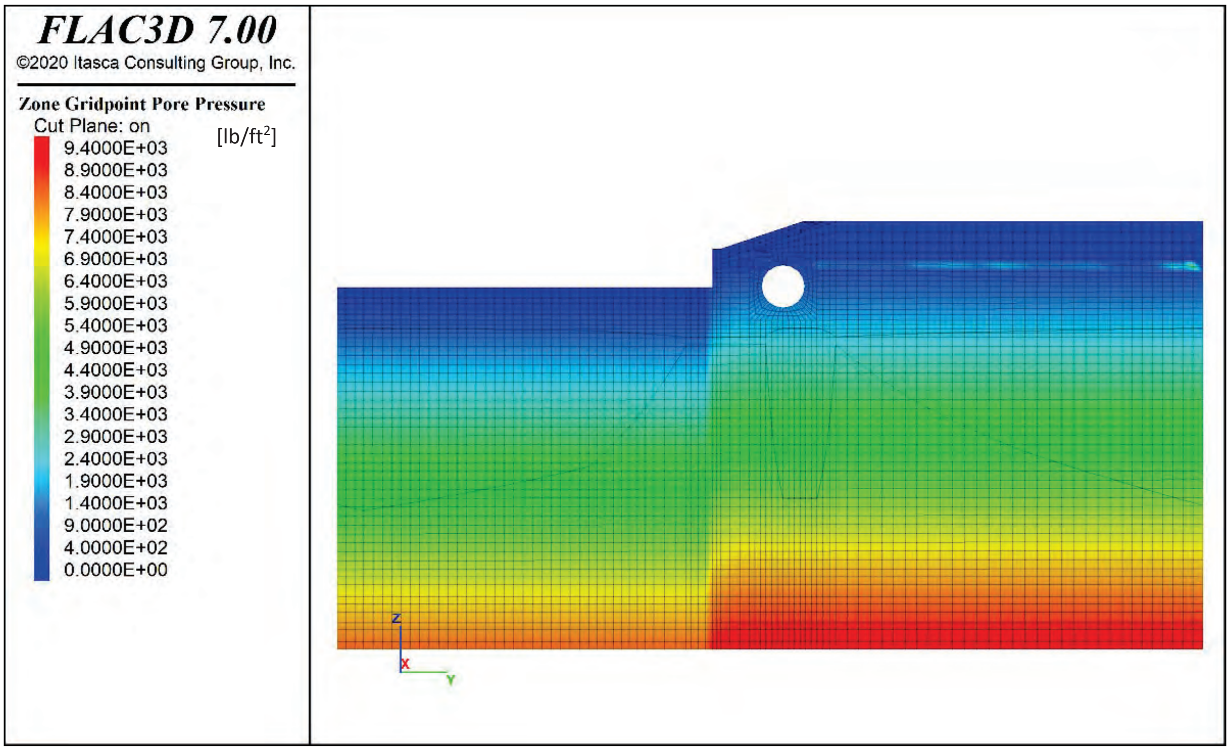


RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 40 SEC. - EARTHQUAKE SCENARIO B - RUBBLE REACH) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-14

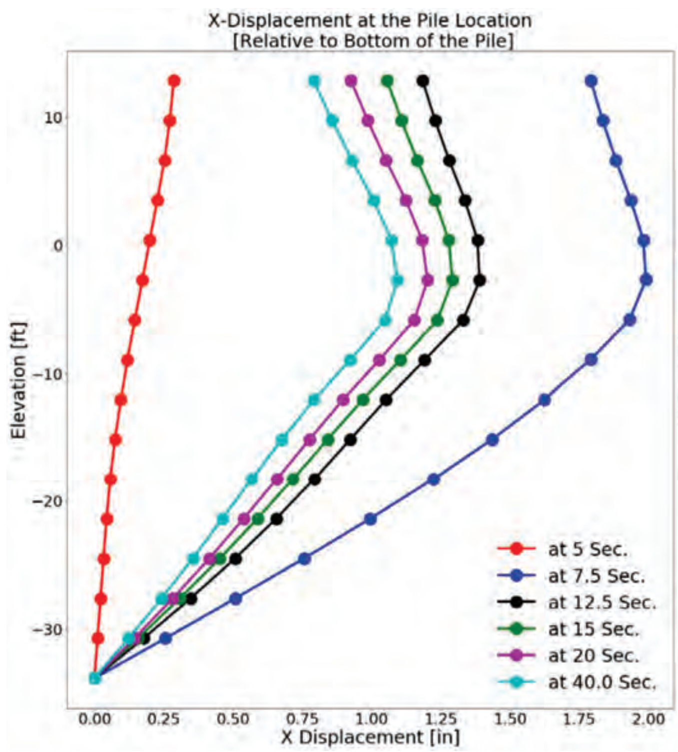


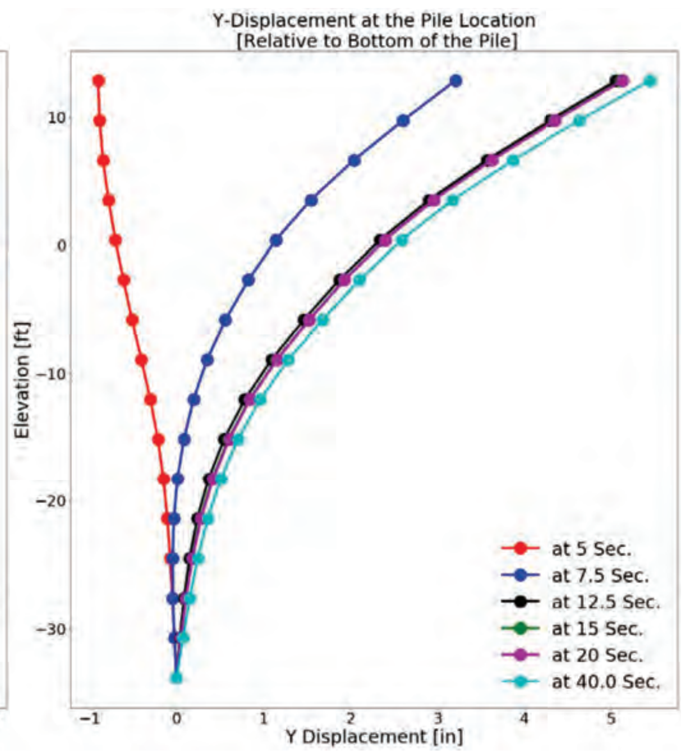
Note: Positive direction in Y direction is toward the slope (away from the ocean).

Y-DIRECTION DISPLACEMENT AT DIFFERENT ELEVATIONS OF THE WALL – MIDDLE SECTION **(EARTHQUAKE SCENARIO B - RUBBLE REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-15



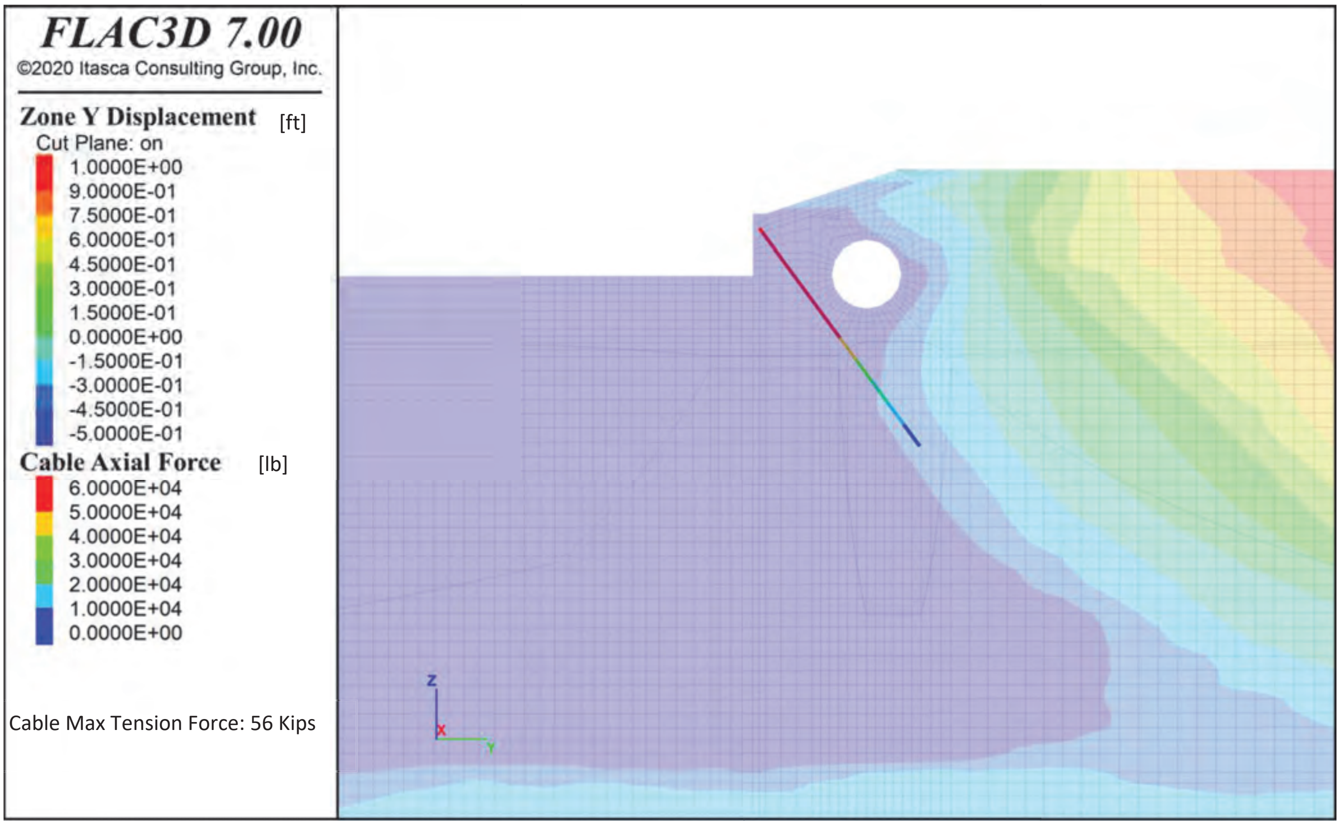
	PORE WATER PRESSURE **(EARTHQUAKE SCENARIO-B AT t= 7.4 SEC. - RUBBLE REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA	AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-16





Note 1: Positive direction in Y direction is toward the slope (away from the ocean).
Note 2: Displacements are shown up to fixity depth of the pile.

DISPLACEMENT IN THE PILE OF THE WALL (MIDDLE PILE) **(RUBBLE REACH - EARTHQUAKE SCENARIO B - RUBBLE REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-17

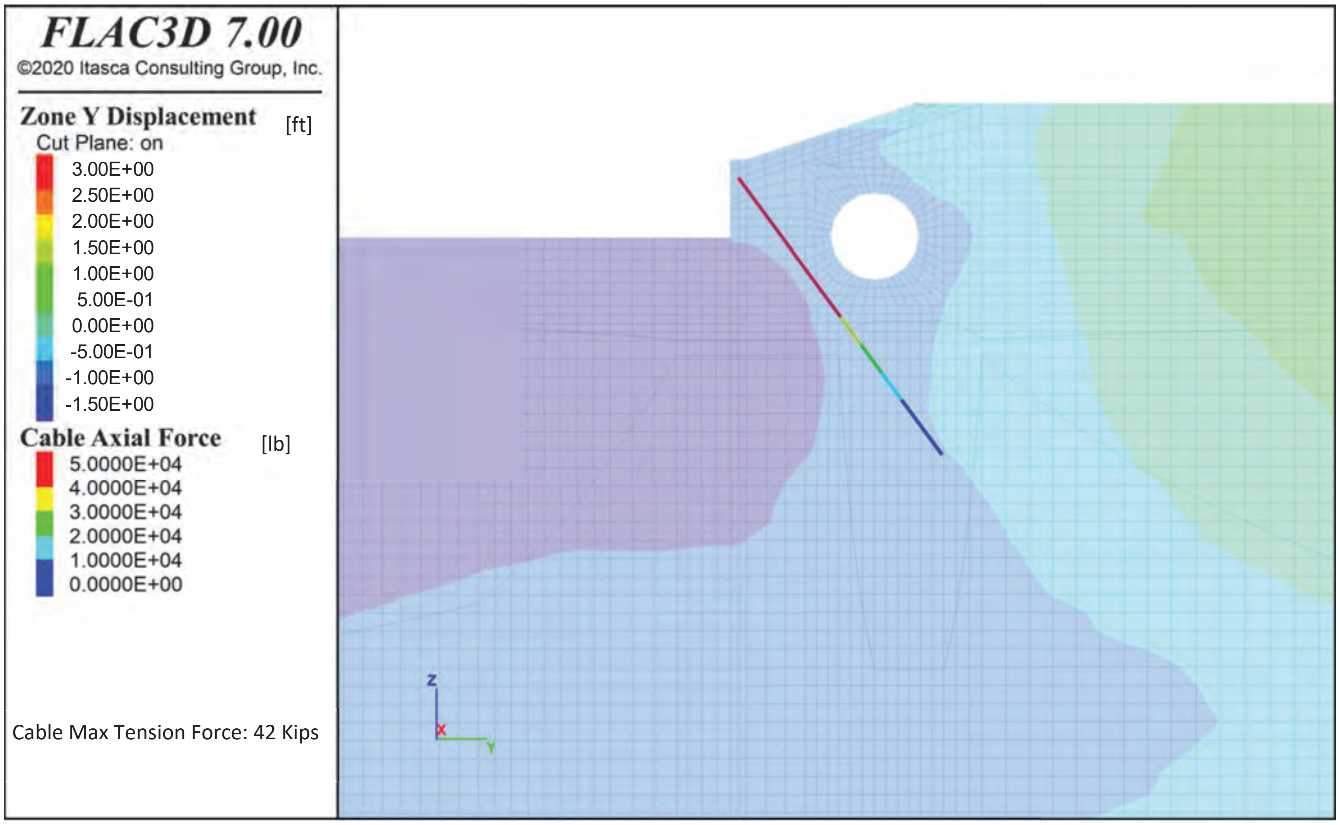


Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION
(AT t = 7.4 SEC. - EARTHQUAKE SCENARIO B - RUBBLE REACH)
AND TENSILE FORCE IN THE TIE-BACK
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA

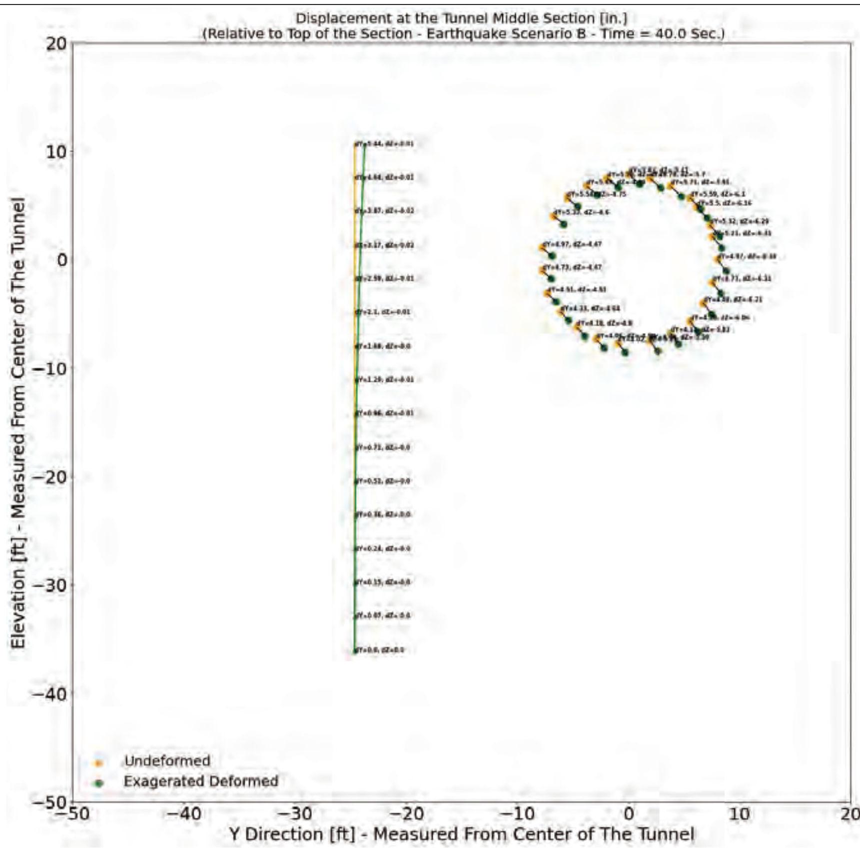
AGS

JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-18



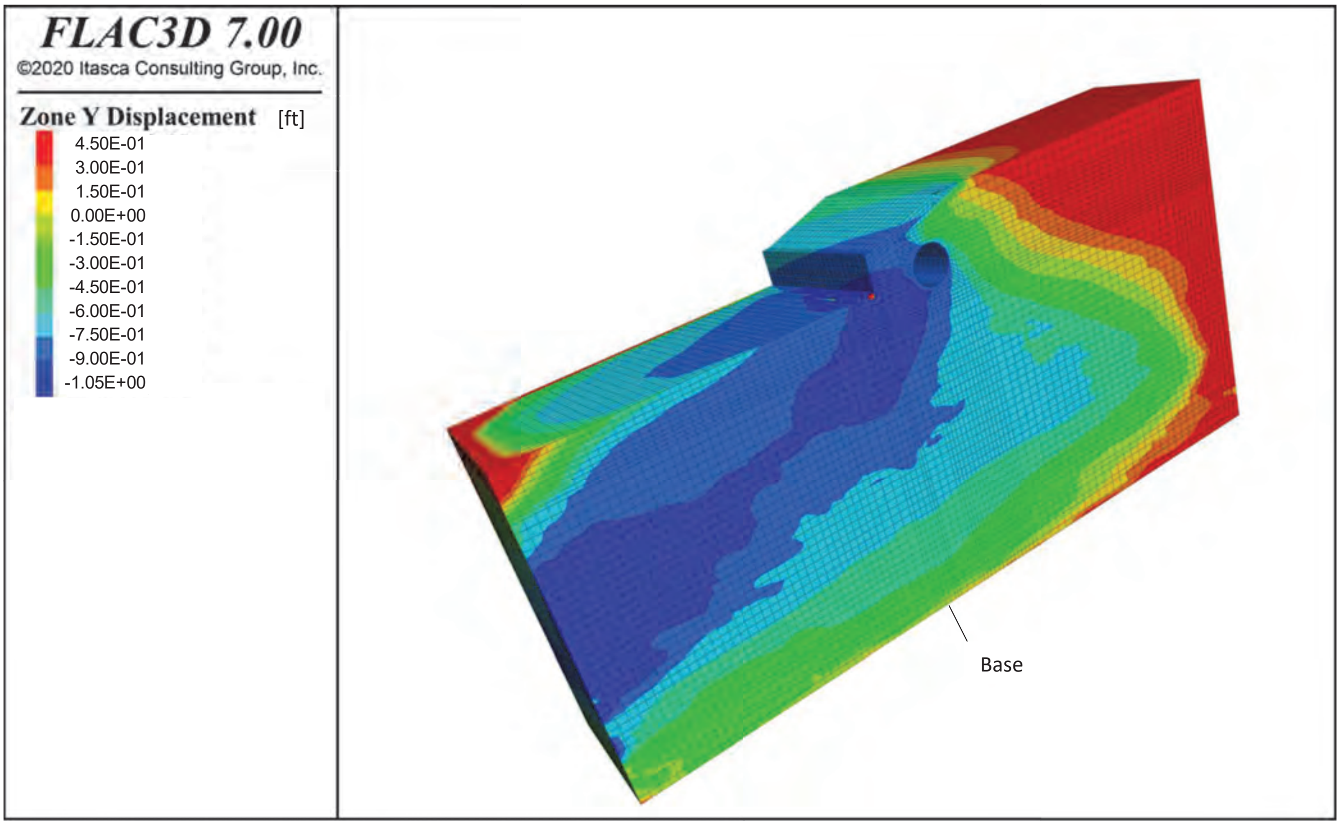
Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO B - RUBBLE REACH)** **AND TENSILE FORCE IN THE TIE-BACK** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-19

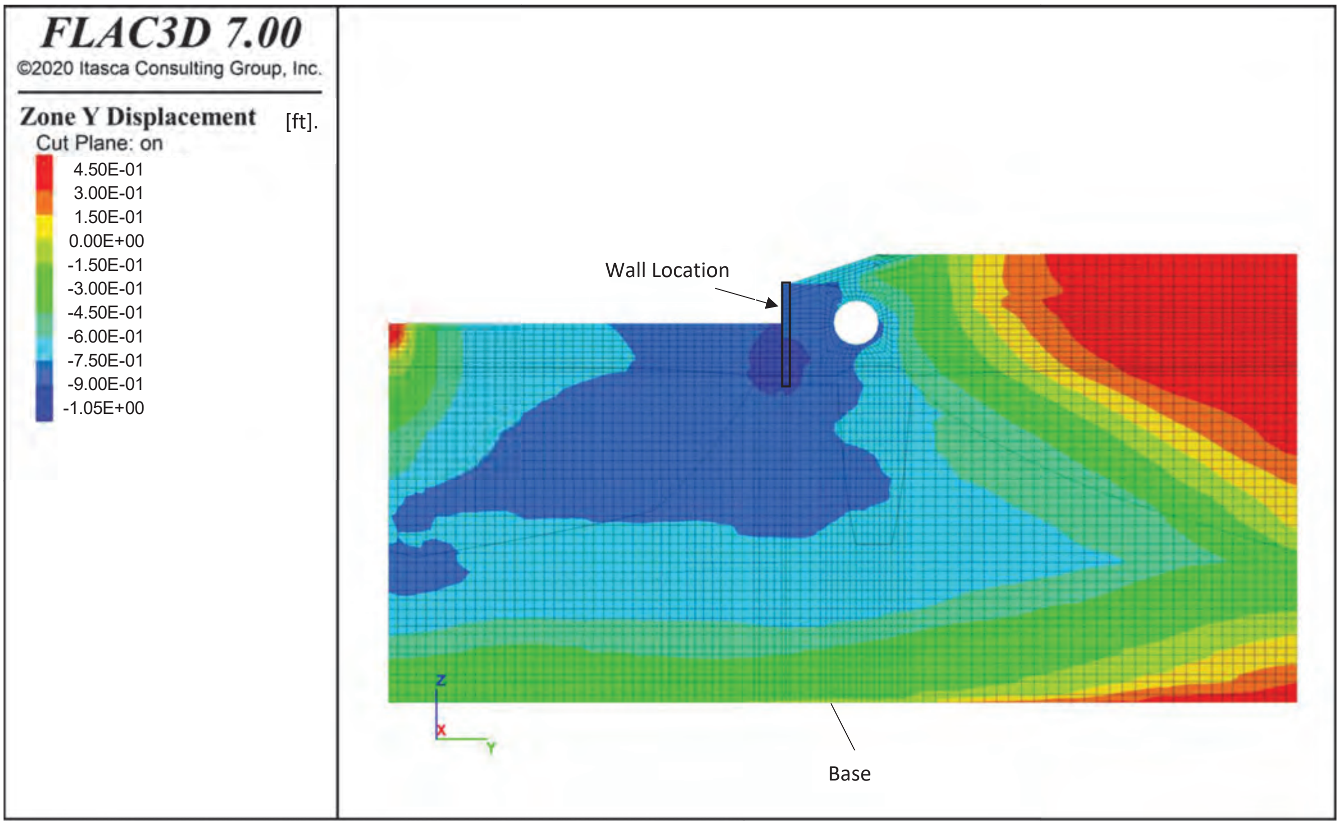


Note 1: Depicted displacements are only due to seismic simulation for the subject SCENARIO and are in addition to the gravitational loading.
Note 2: dY and dZ are relative displacement (relative to bottom of the pile) in Y and Z (elevation) directions, respectively.
Note 3: Displacements are exaggerated. Exaggeration factor in both Z and Y direction is 2.
Note 4: Positive direction in Y direction is toward the slope (away from the ocean).

TUNNEL AND PILE DISPLACEMENT **(AT t = 40.0 SEC. - EARTHQUAKE SCENARIO B RUBBLE REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-20

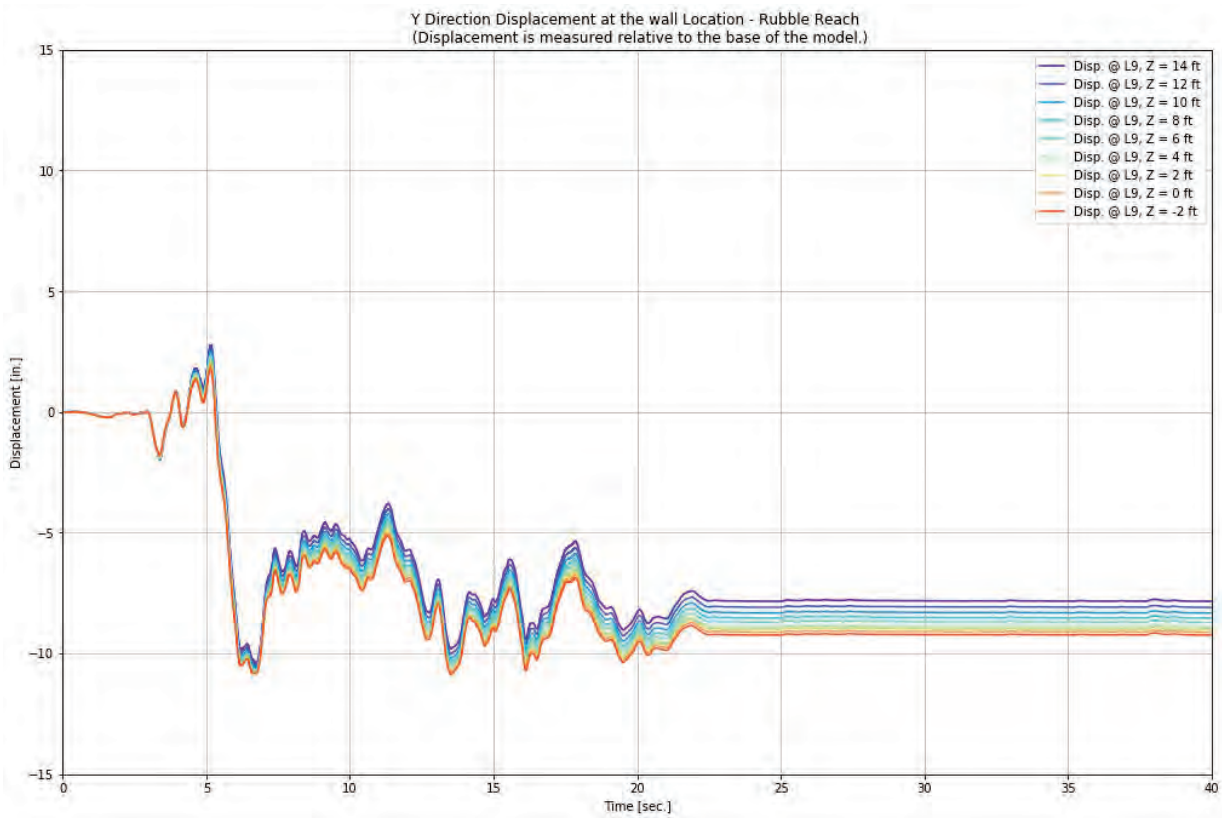


RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO C - RUBBLE REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA	AGS
JOB NO. AGS-18-003 DATE: MAY 2021	PLATE D-21

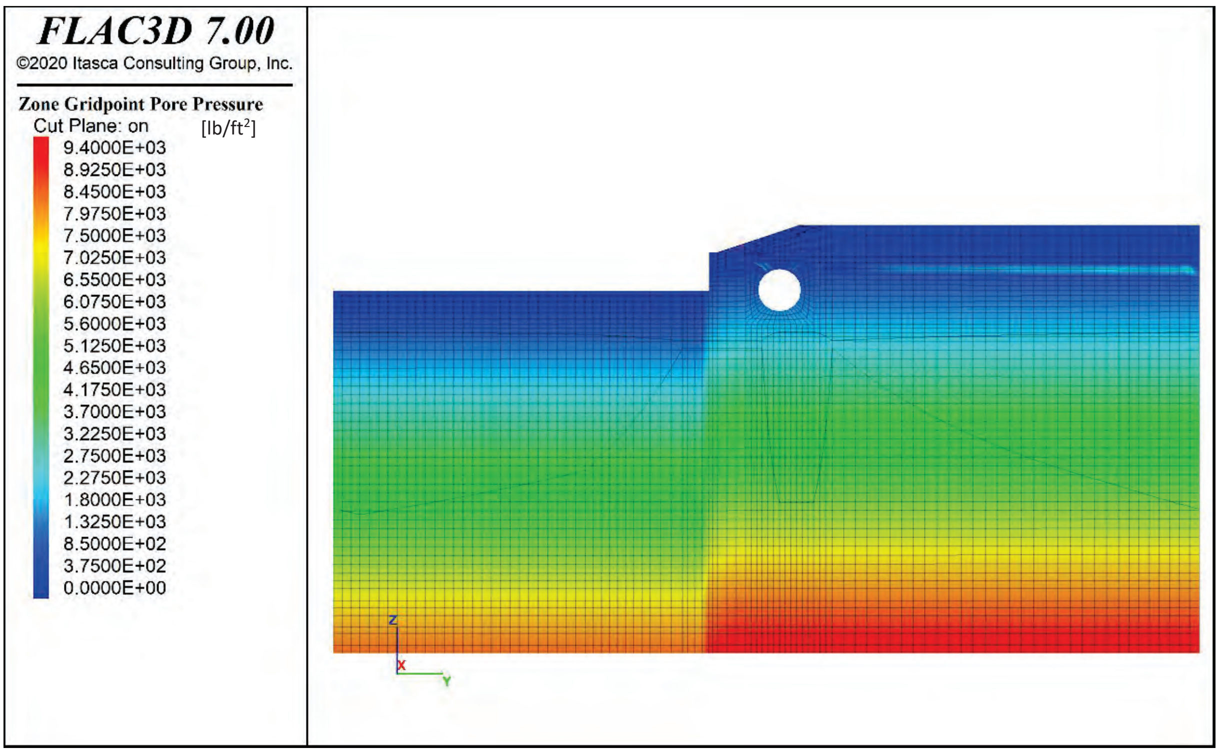


RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO C - RUBBLE REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA	AGS

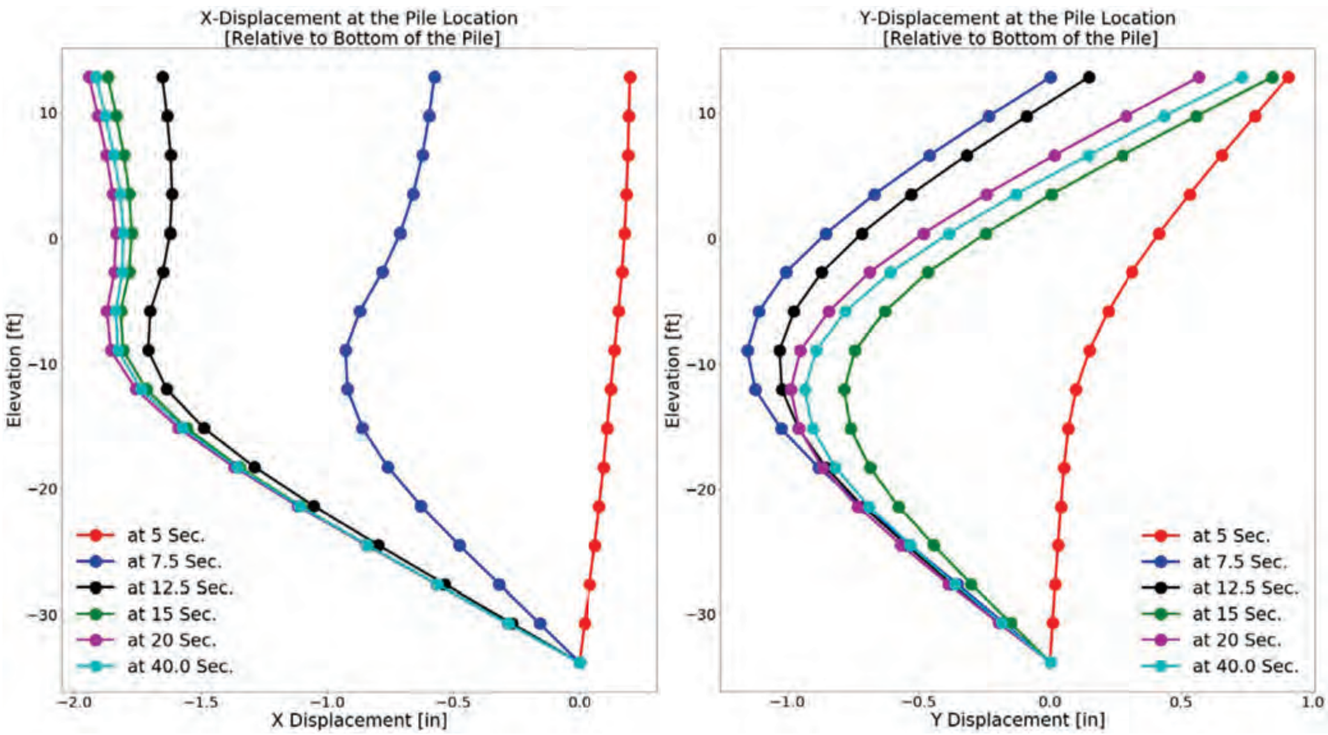
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-22



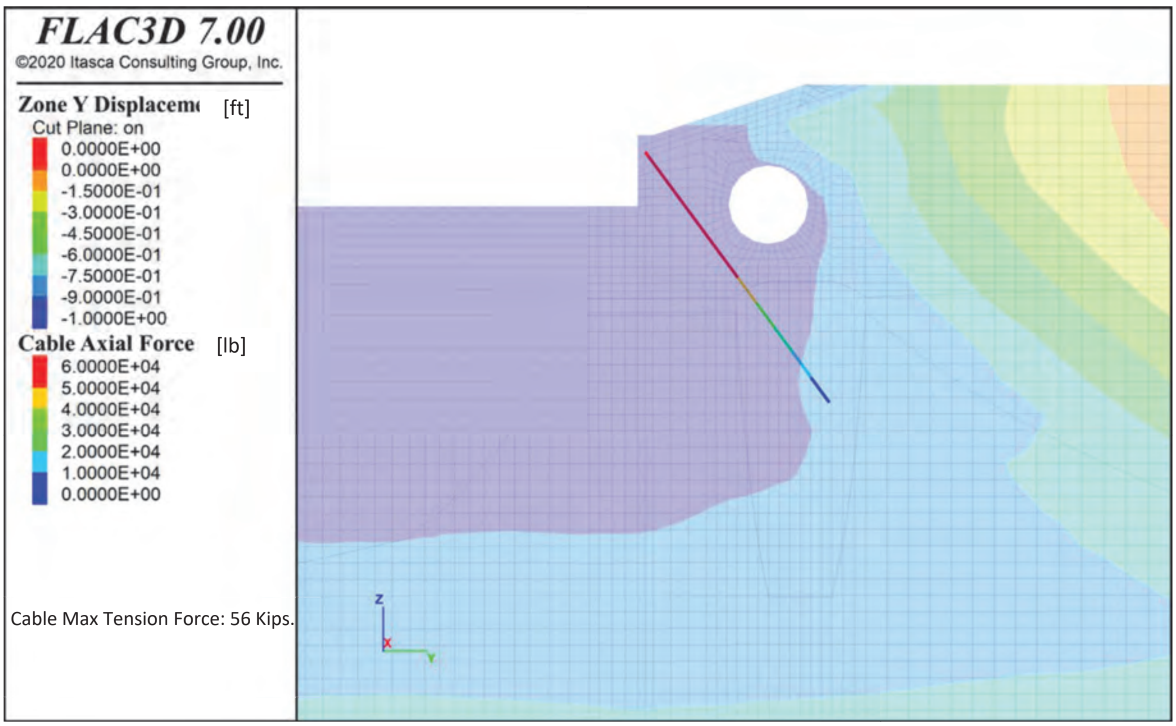
Note: Positive direction in Y direction is toward the slope (away from the ocean).	**Y-DIRECTION DISPLACEMENT AT DIFFERENT ELEVATIONS OF THE WALL – MIDDLE SECTION** **(EARTHQUAKE SCENARIO C - RUBBLE REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION	AGS
	JOB NO. AGS-18-003 / DATE: MAY 2021	PLATE D-23



	PORE WATER PRESSURE **(EARTHQUAKE SCENARIO-C AT t= 6.0 SEC- RUBBLE REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA	AGS
	JOB NO. AGS-18-003 · DATE: MAY 2021	PLATE D-24

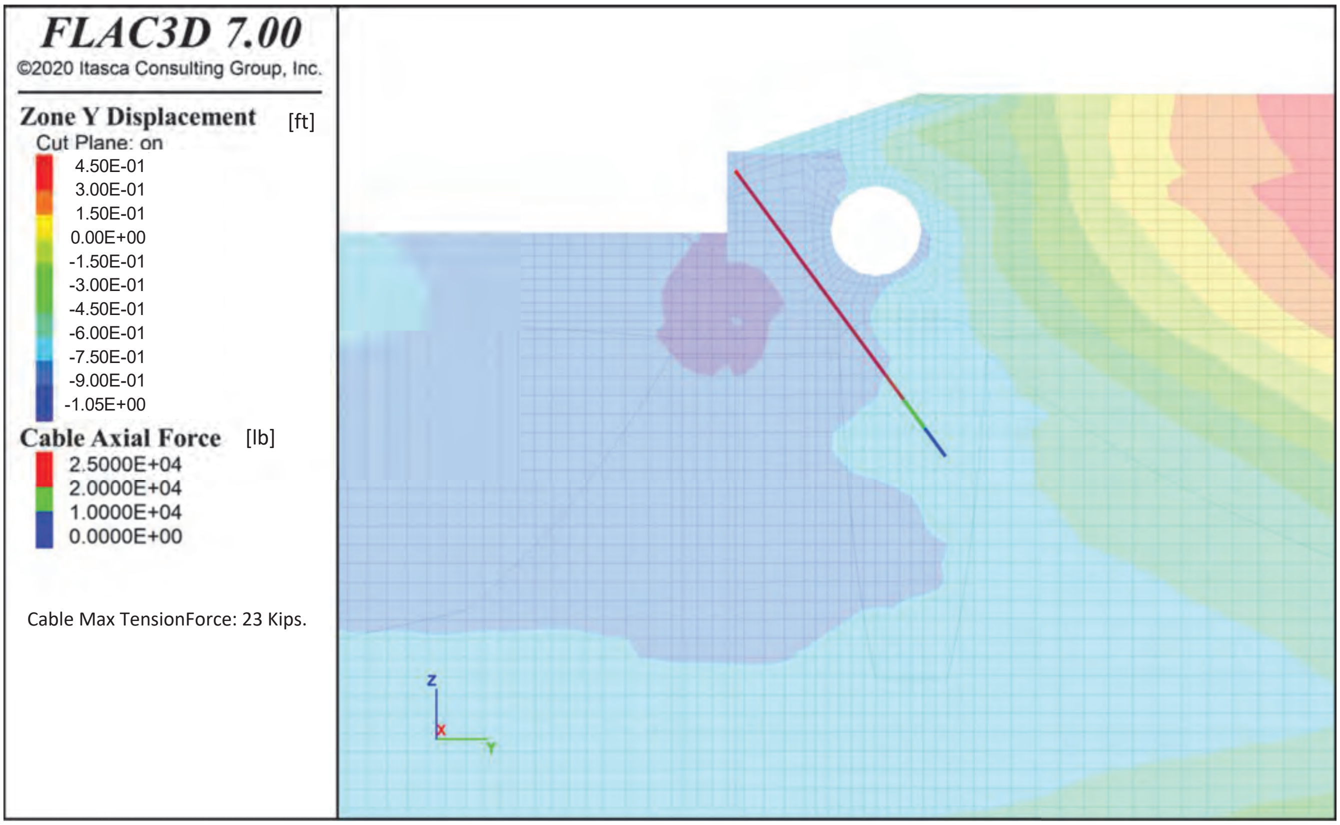


Note 1: Positive direction in Y direction is toward the slope (away from the ocean). **Note 2**: Displacements are shown up to fixity depth of the pile.	**DISPLACEMENT IN THE PILE OF THE WALL (MIDDLE PILE)** **(RUBBLE REACH - EARTHQUAKE SCENARIO C - RUBBLE REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA	AGS
	JOB NO. AGS-18-003 / DATE: MAY 2021	PLATE D-25



Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 6 SEC. - EARTHQUAKE SCENARIO C - RUBBLE REACH) AND TENSILE FORCE IN THE TIE-BACK SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE D-26



Note: Y Displacement contours shown in transparent.

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(AT t = 40.0 SEC. - EARTHQUAKE SCENARIO C - RUBBLE REACH)
AND TENSILE FORCE IN THE TIE-BACK
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA</td><td>AGS</td></tr>
<tr><td>JOB NO. AGS-18-003</td><td>DATE: MAY 2021</td><td>PLATE D-27</td></tr>
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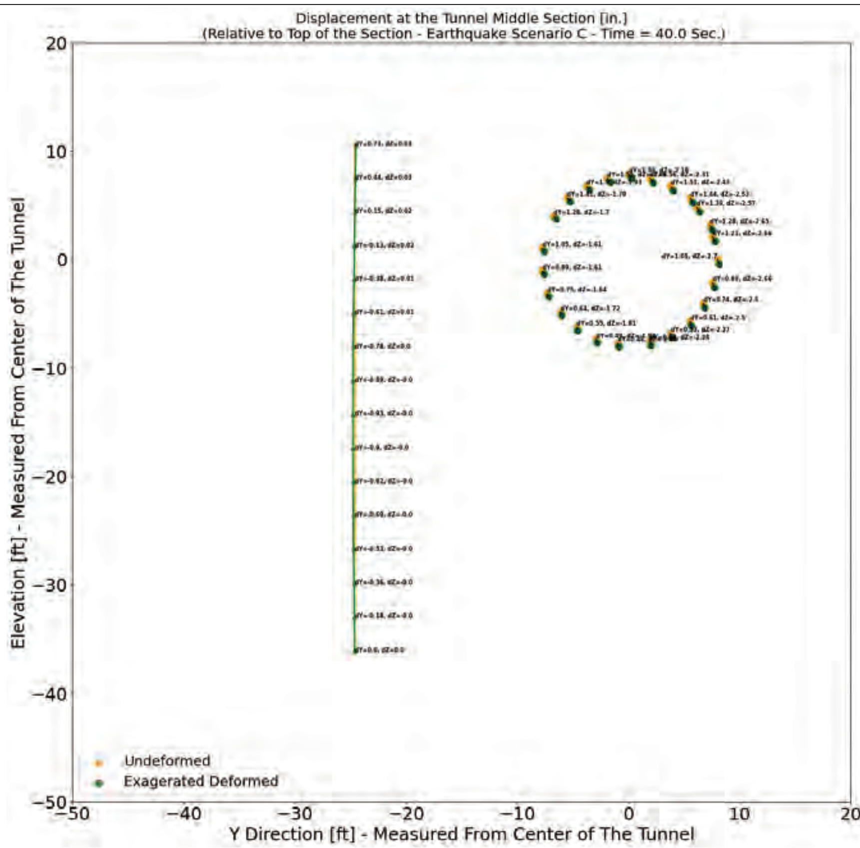
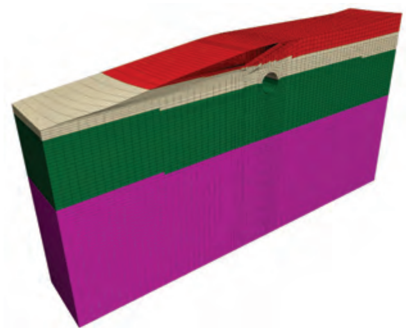


Note 1: Depicted displacements are only due to seismic simulation for the subject SCENARIO and are in addition to the gravitational loading.
Note 2: dY and dZ are relative displacement (relative to bottom of the pile) in Y and Z (elevation) directions, respectively.
Note 3: Displacements are exaggerated. Exaggeration factor in both Z and Y direction is 2.
Note 4: Positive direction in Y direction is toward the slope (away from the ocean).

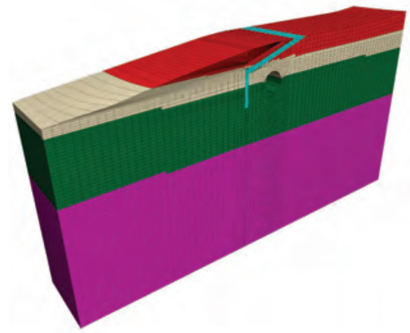
TUNNEL AND PILE DISPLACEMENT **(AT t = 40.0 SEC. - EARTHQUAKE SCENARIO C RUBBLE REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS
JOB NO. AGS-18-003 DATE: MAY 2021	PLATE D-28

Appendix E

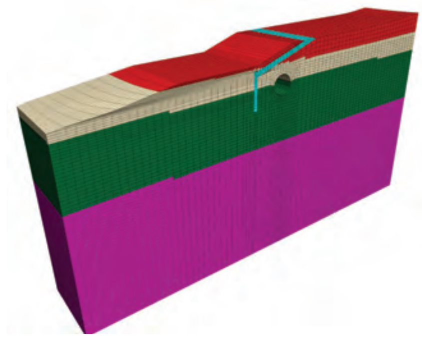
FLAC3D NUMERICAL SSSI DYNAMIC ANALYSES RESULTS FOR BLUFF REACH



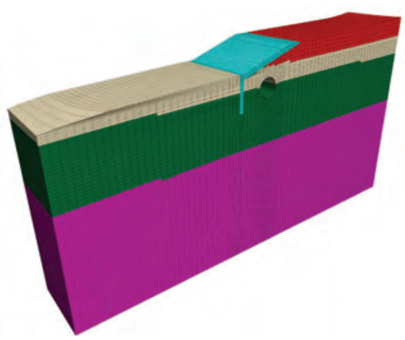
Stage 1 – Initialization of the Model



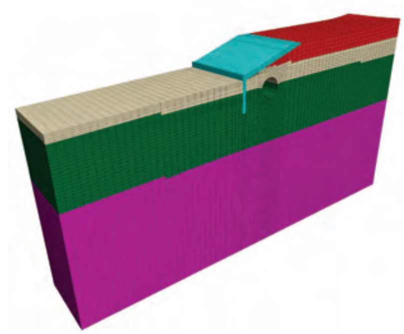
Stage 2 – Adding Piles, Secant Wall, Tie-Back Cables, and CLSM



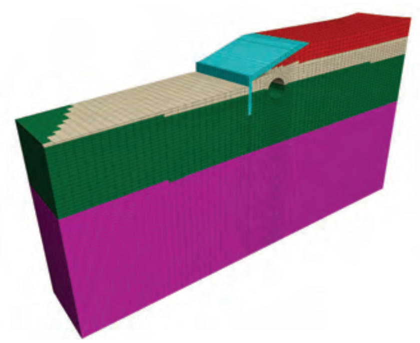
Stage 3a – Erosion above the CLSM



Stage 3b – Erosion above the CLSM



Stage 3c – Erosion above the CLSM



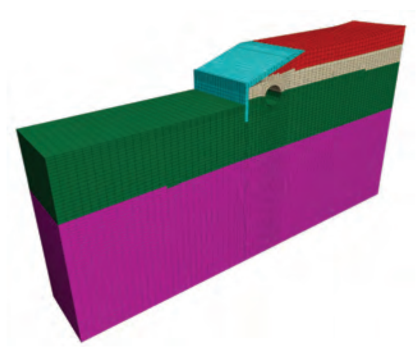
Stage 4 – Erosion in front of the Secant Wall

CLSM and Wall	Liquefiable Layer
Fill	Colma Formation
	Merced Formation

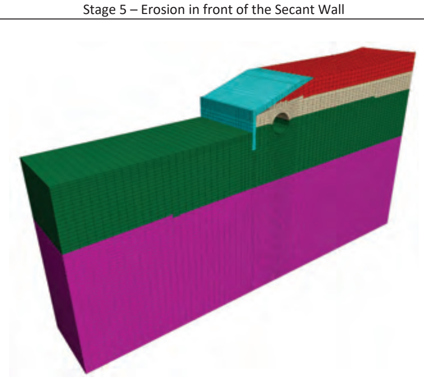
BLUFF REACH MESH – DIFFERENT STAGES OF THE NUMERICAL SIMULATION
SOUTH OCEAN BEACH C OASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO, CALIFORNIA

AGS

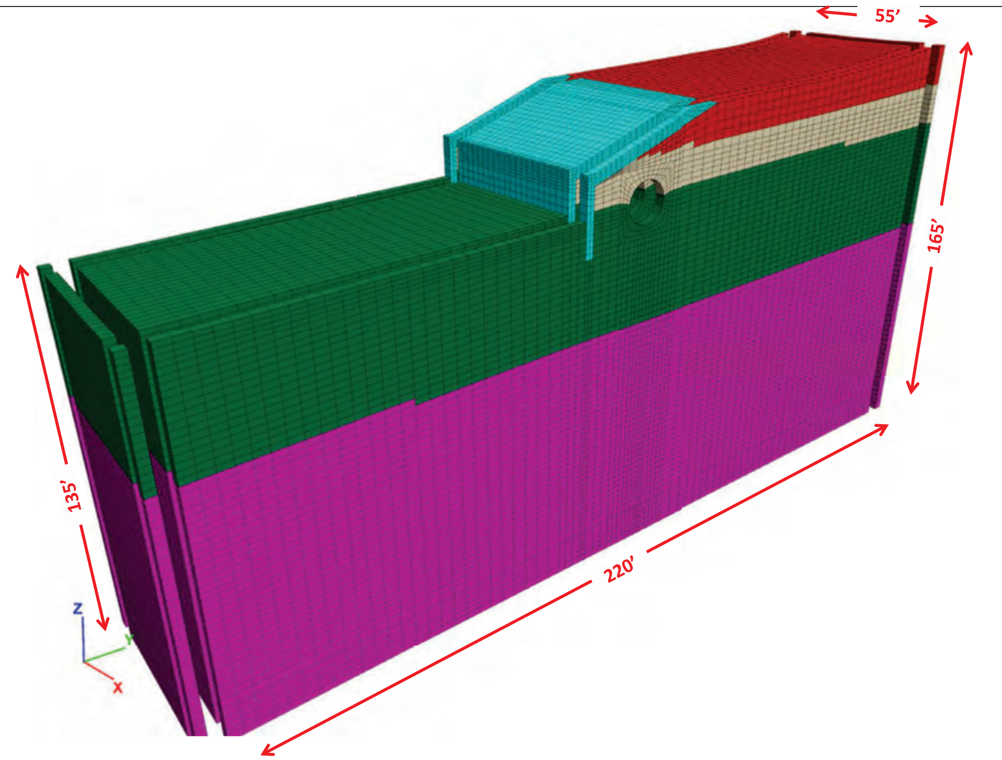
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-1 a



Stage 5 – Erosion in front of the Secant Wall



Stage 6 – Erosion in front of the Secant Wall



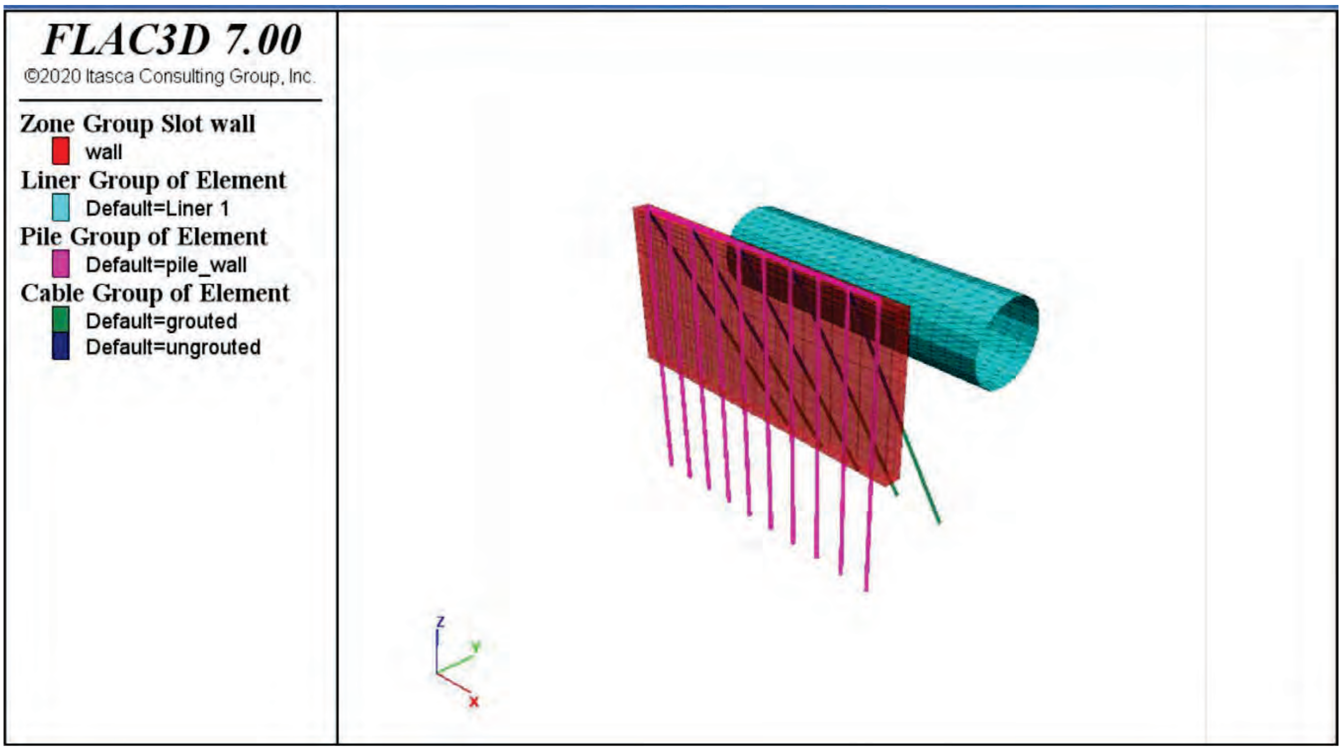
Mesh Used to Seismic Analysis – Includes Free Field Boundary Conditions

CLSM and Wall	Liquefiable Layer
Fill	Colma Formation
	Merced Formation

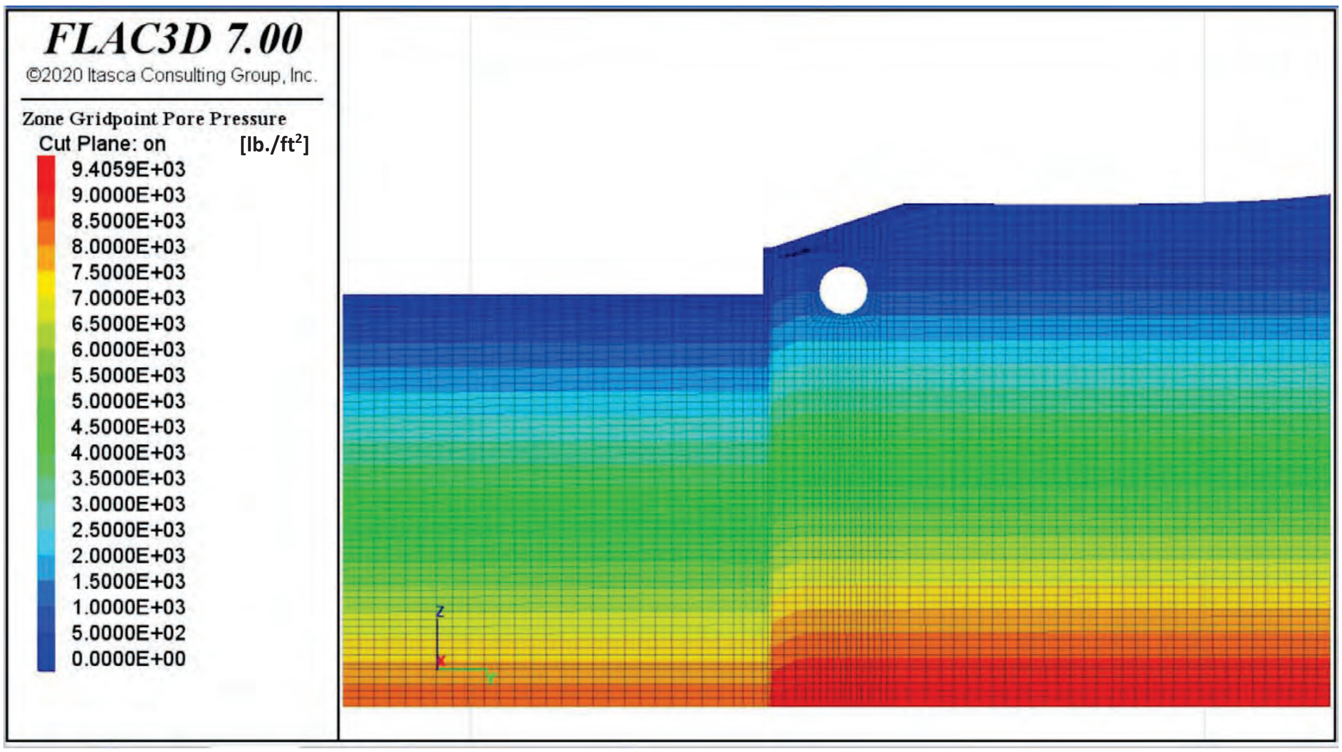
BLUFF REACH MESH – DIFFERENT STAGES OF THE NUMERICAL SIMULATION
SOUTH OCEAN BEACH C OASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO, CALIFORNIA

AGS

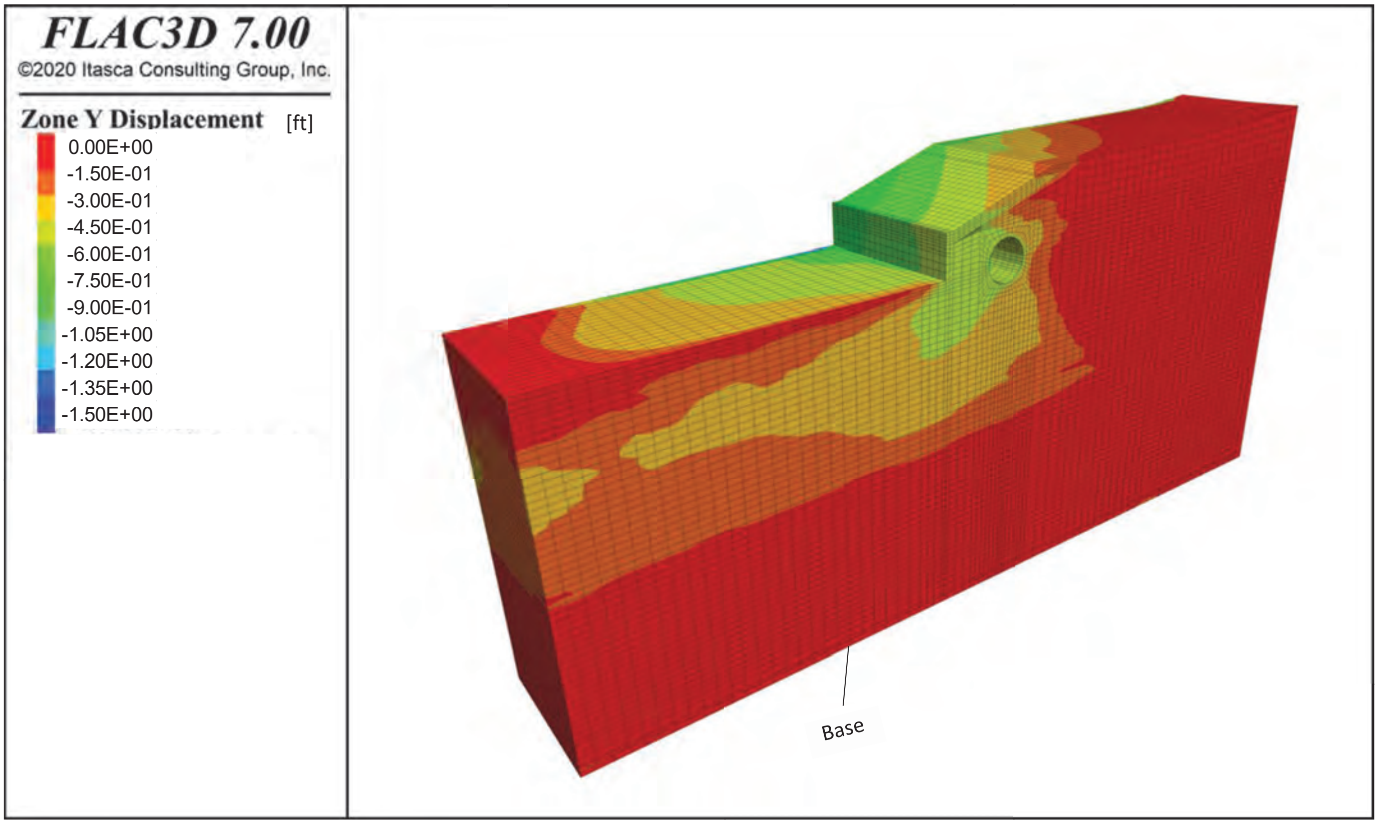
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-1 b



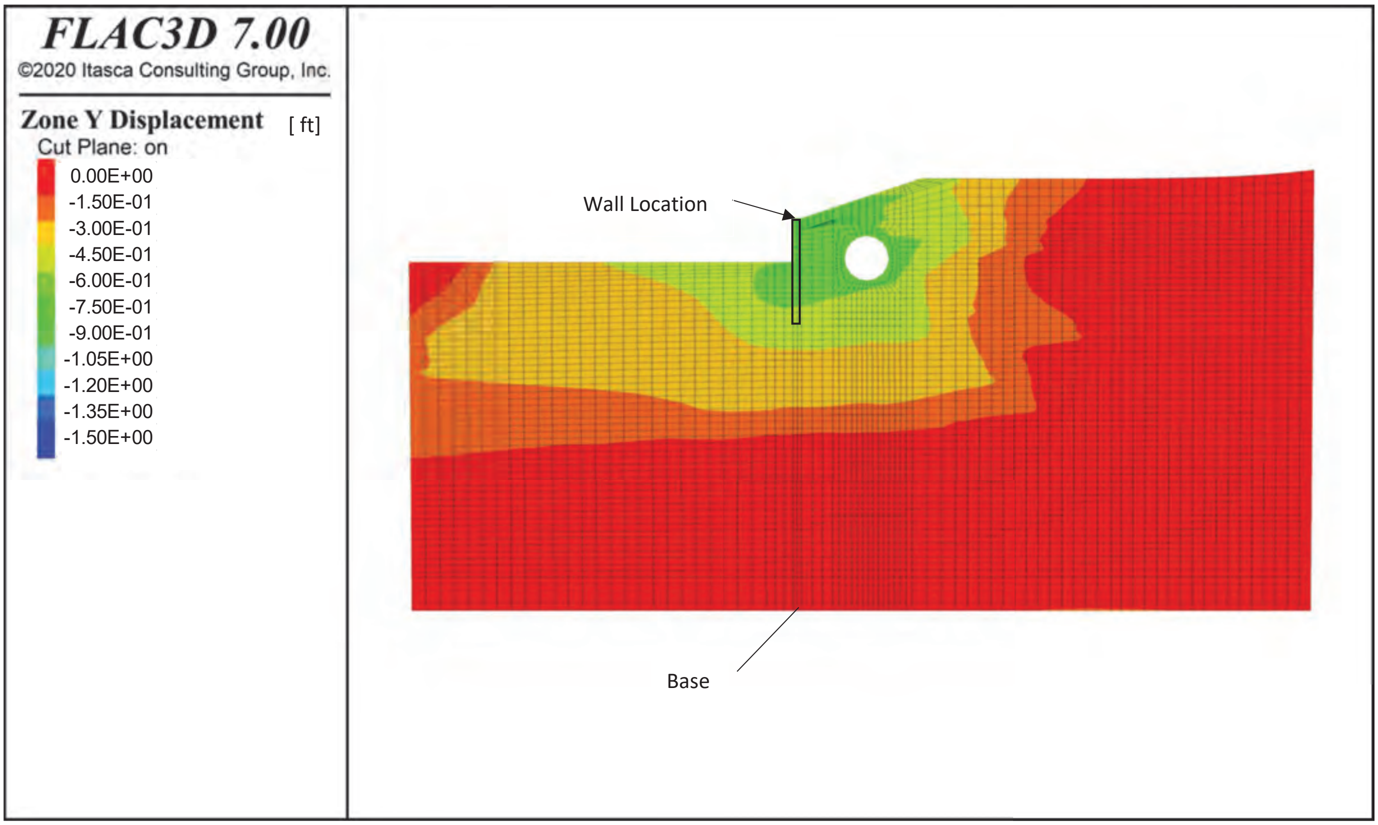
STRUCTURAL ELEMENTS OF THE MODEL (EARTHQUAKE SCENARIO A - BLUFF REACH) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-2



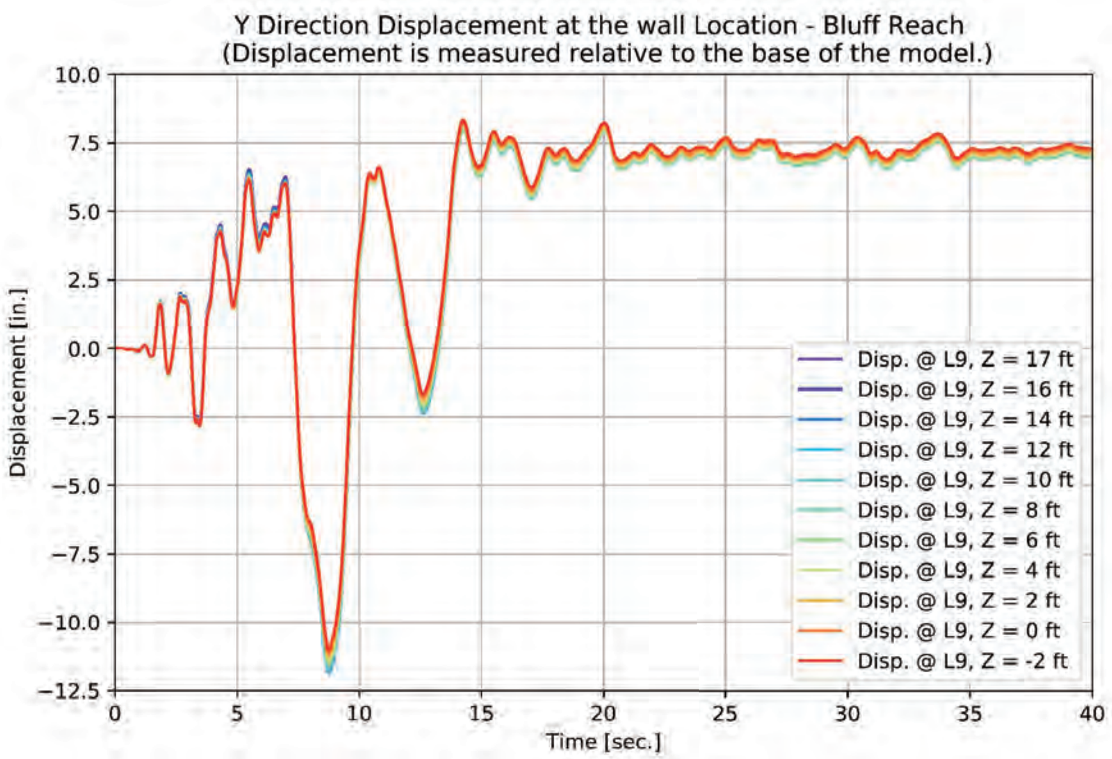
HYDROSTATIC PORE-WATER PRESSURE PRIOR TO SHAKING (BLUFF REACH) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS
JOB NO. AGS-18-003 / DATE: JUNE 2021	PLATE E-3



RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO A - BLUFF REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-4



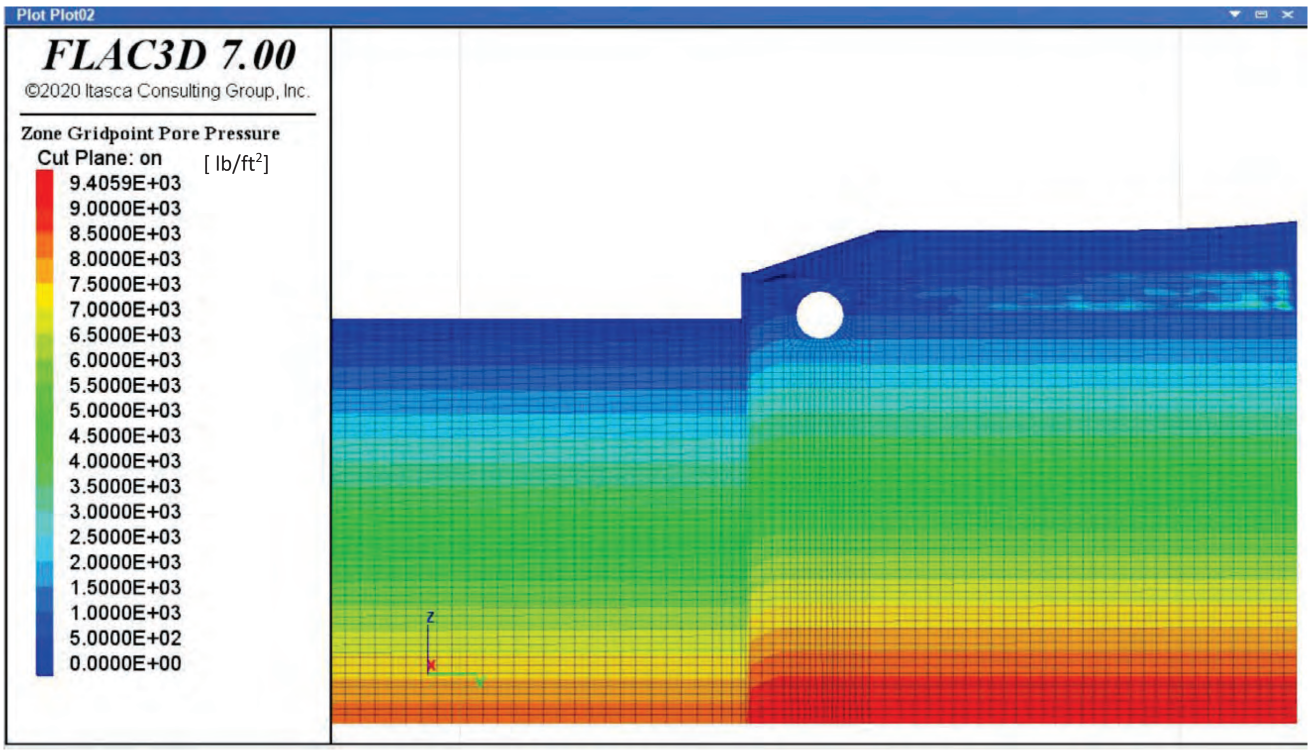
RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO A - BLUFF REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-5



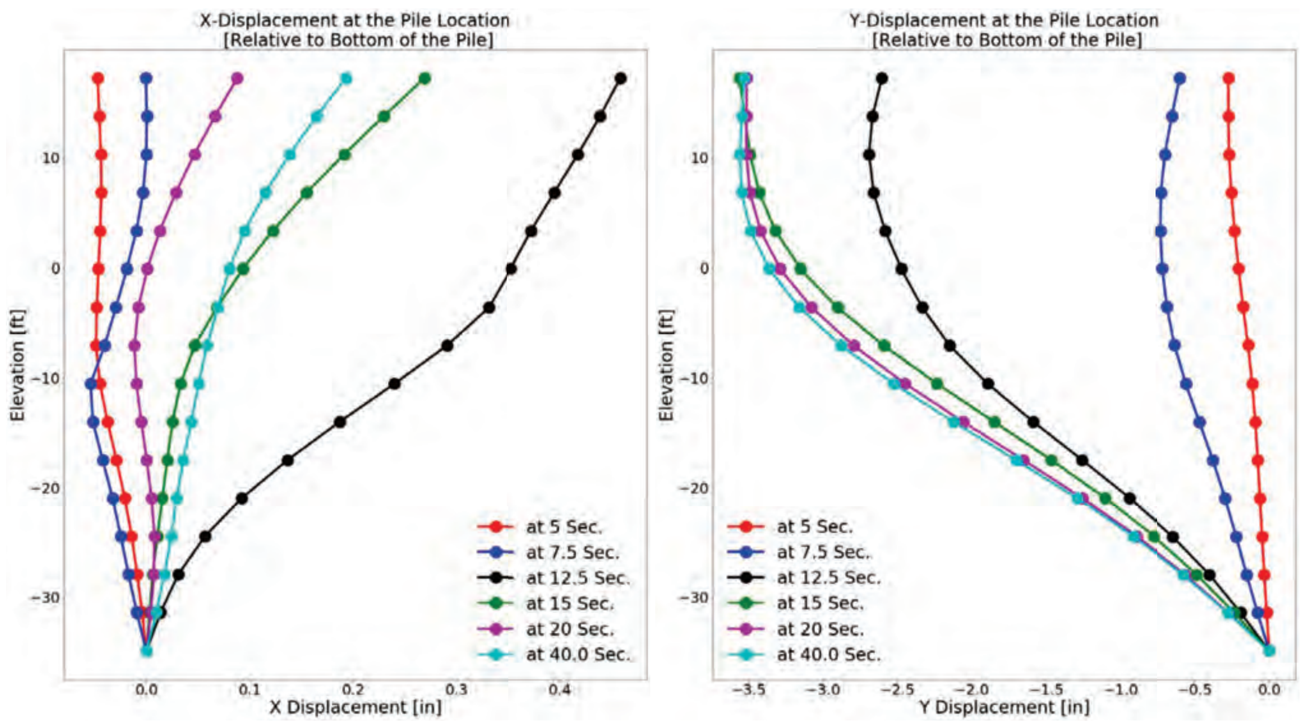
Note: Positive direction in Y direction is toward the slope (away from the ocean).

Y-DIRECTION DISPLACEMENT AT DIFFERENT ELEVATIONS OF THE WALL – MIDDLE SECTION **(EARTHQUAKE SCENARIO A - BLUFF REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS

JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-6

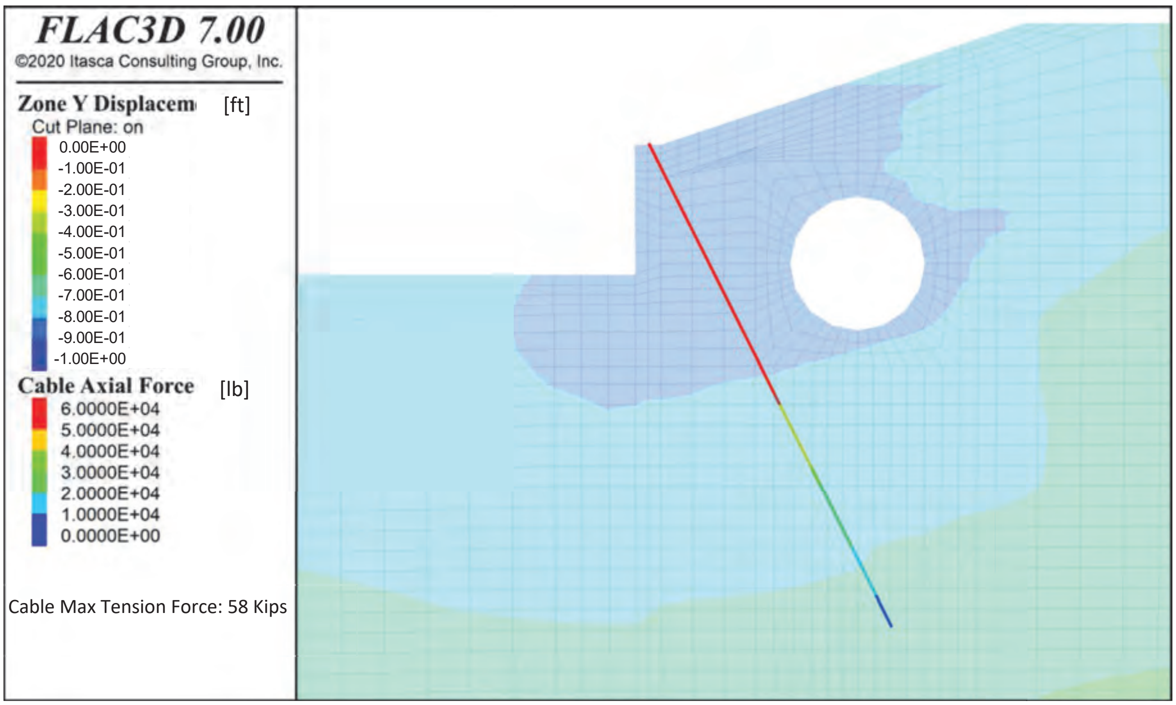


	PORE WATER PRESSURE **(EARTHQUAKE SCENARIO-A AT t= 7.4 SEC- BLUFF REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-7



Note 1: Positive direction in Y direction is toward the slope (away from the ocean).
Note 2: Displacements are shown up to fixity depth of the pile.

DISPLACEMENT IN THE PILE OF THE WALL (MIDDLE PILE) **(BLUFF REACH - EARTHQUAKE SCENARIO A - BLUFF REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-8

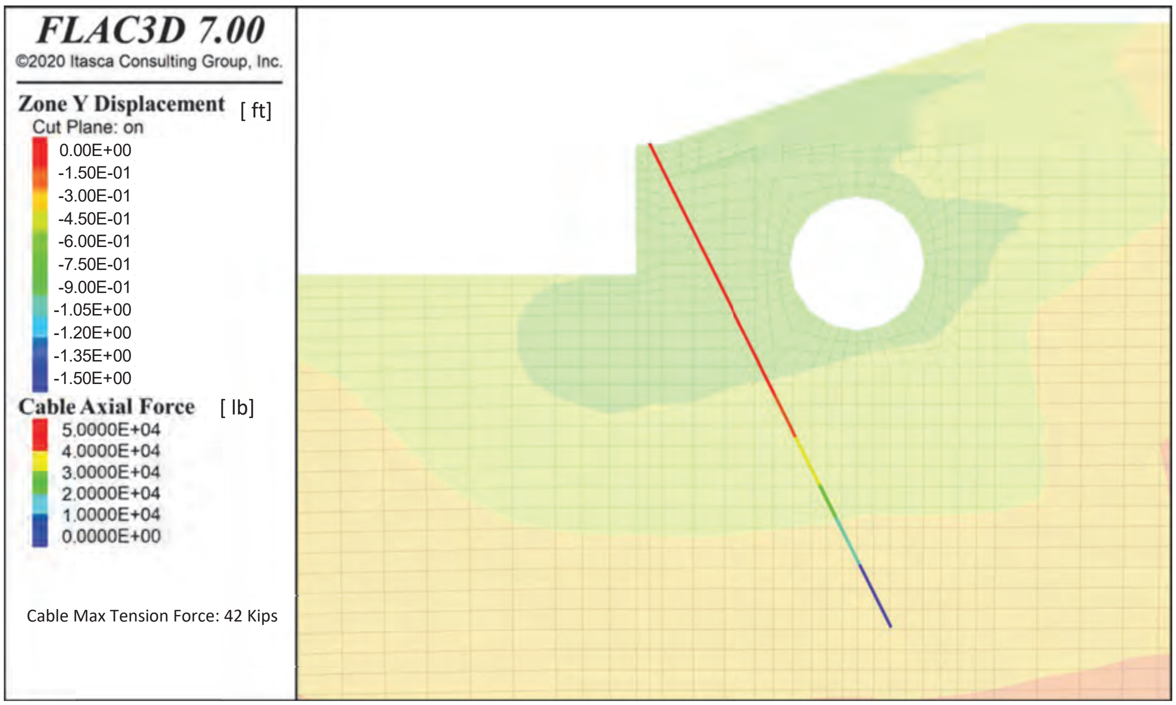


Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION
(AT t = 7.4 SEC. - EARTHQUAKE SCENARIO A - BLUFF REACH)
AND TENSILE FORCE IN THE TIE-BACK
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA

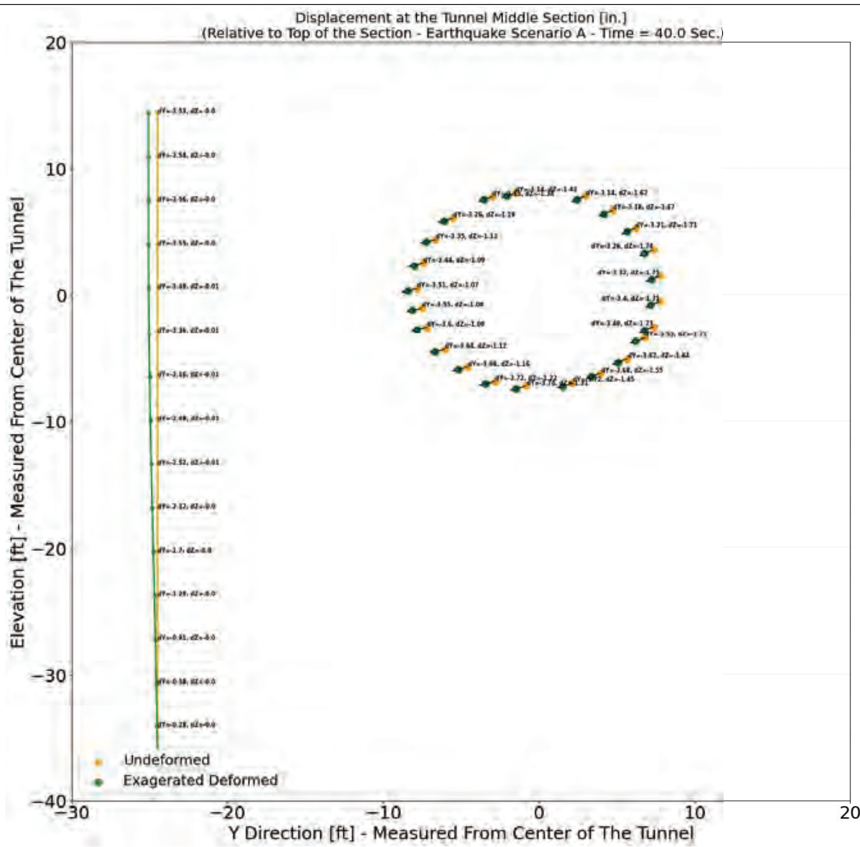
AGS

JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-9



Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 40.0 SEC. - EARTHQUAKE SCENARIO A - BLUFF REACH) AND TENSILE FORCE IN THE TIE-BACK SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-10

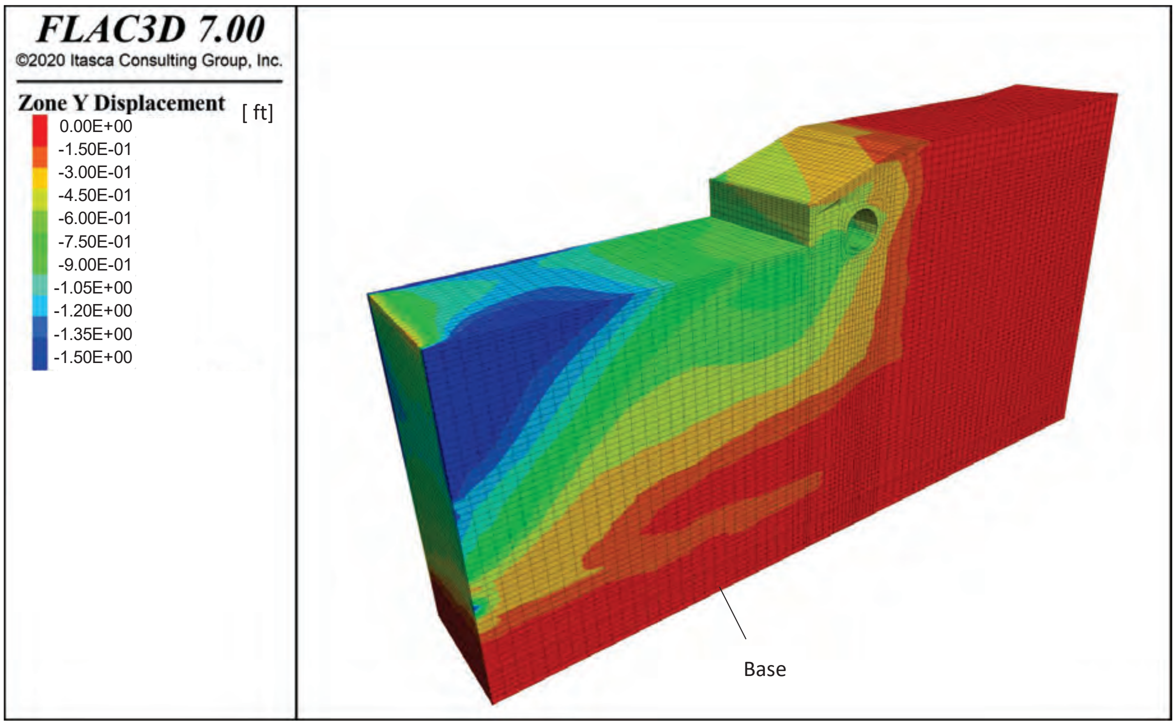


Note 1: Depicted displacements are only due to seismic simulation for the subject SCENARIO and are in addition to the gravitational loading.
Note 2: dY and dZ are relative displacement (relative to bottom of the pile) in Y and Z (elevation) directions, respectively.
Note 3: Displacements are exaggerated. Exaggeration factor in both Z and Y direction is 2.
Note 4: Positive direction in Y direction is toward the slope (away from the ocean).

TUNNEL AND PILE DISPLACEMENT
(AT t = 40.0 SEC. - EARTHQUAKE SCENARIO A BLUFF REACH)
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA

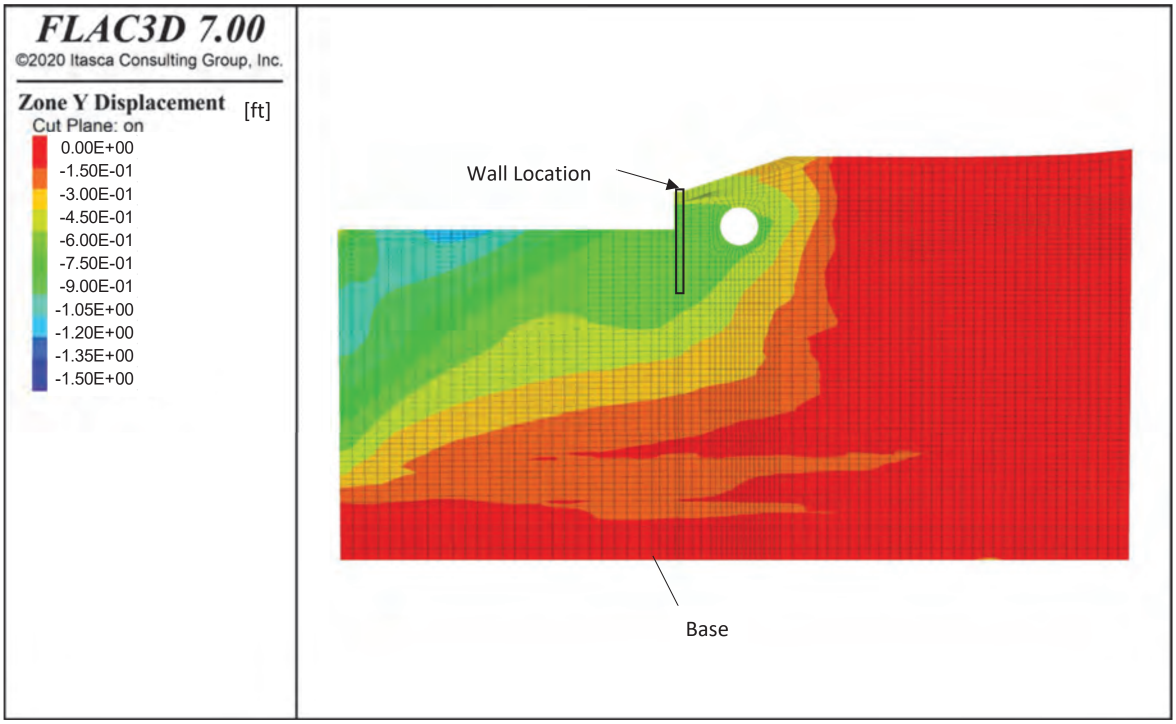
AGS

JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-11

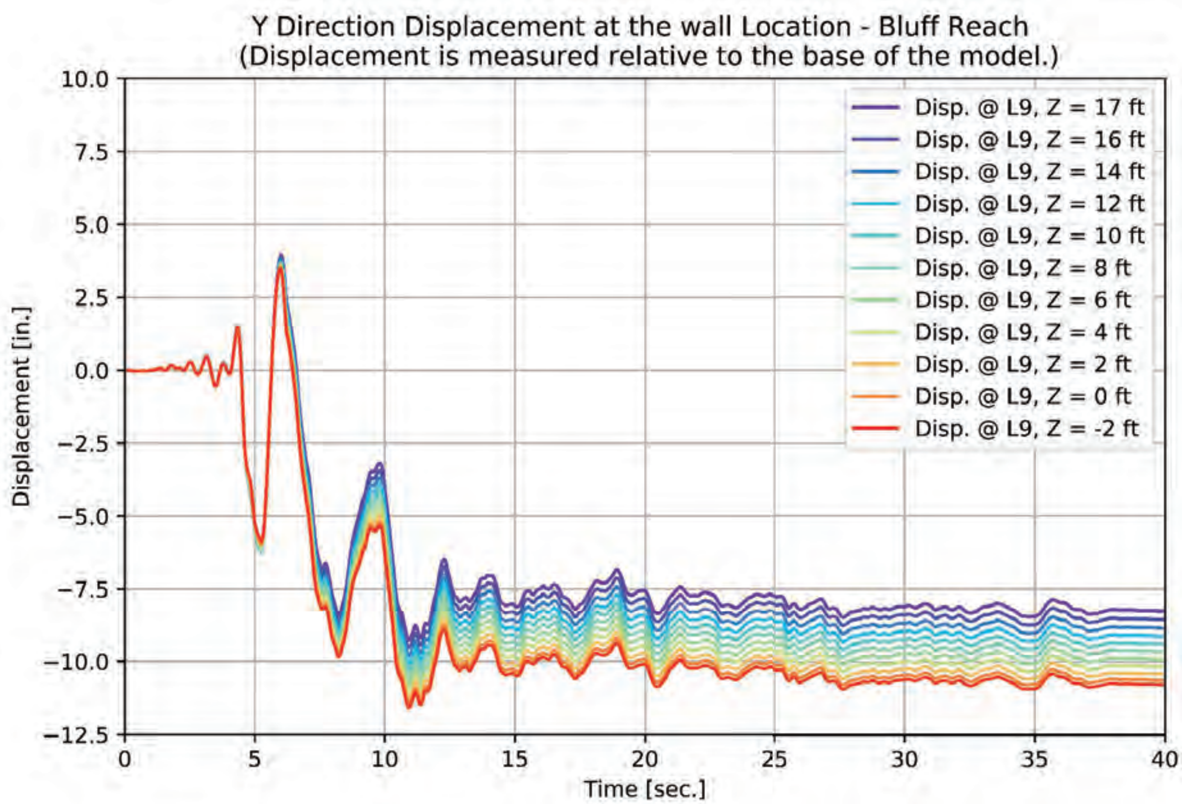


RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO B - BLUFF REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA	AGS

JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-13

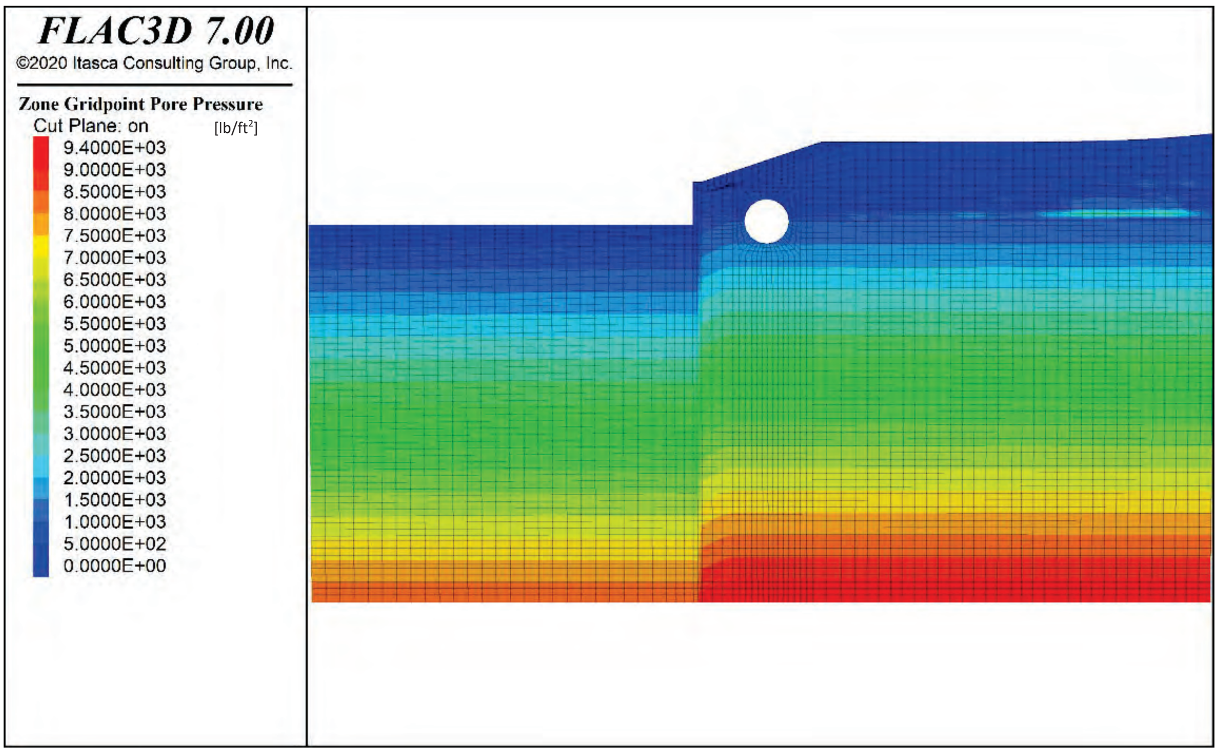


RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO B - BLUFF REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-14

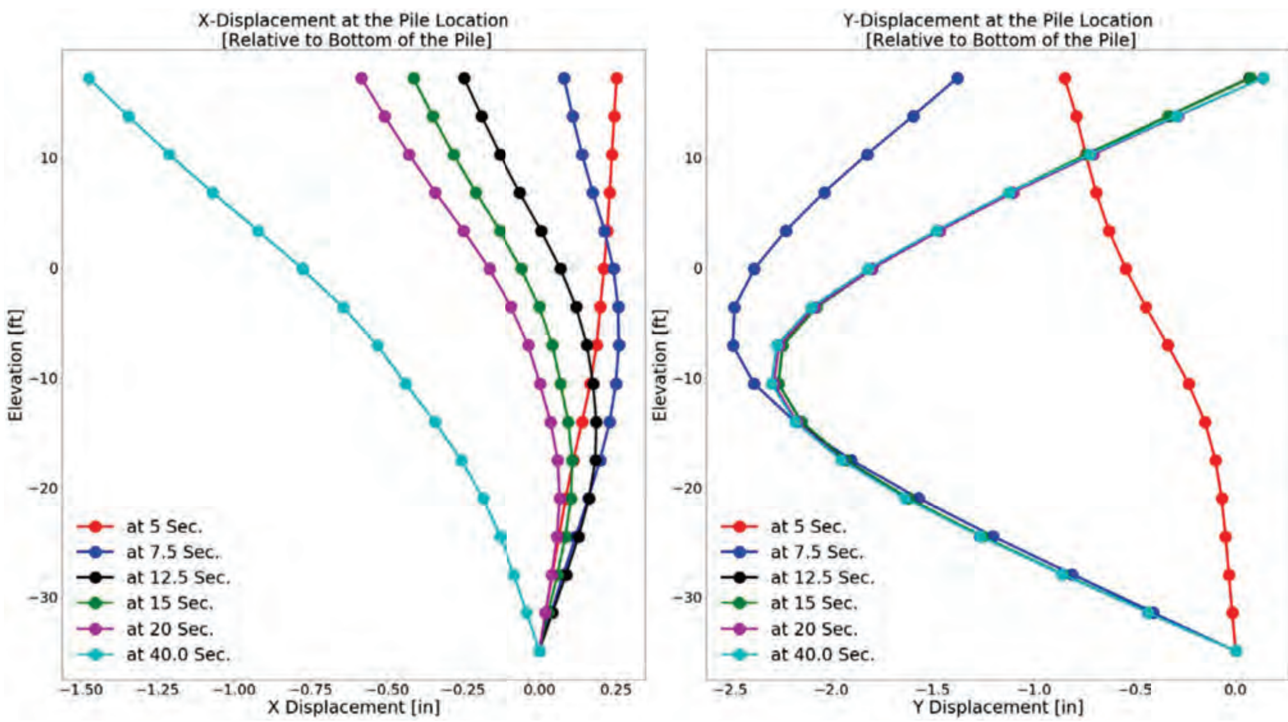


Note: Positive direction in Y direction is toward the slope (away from the ocean).

Y-DIRECTION DISPLACEMENT AT DIFFERENT ELEVATIONS OF THE WALL – MIDDLE SECTION **(EARTHQUAKE SCENARIO B - BLUFF REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-15

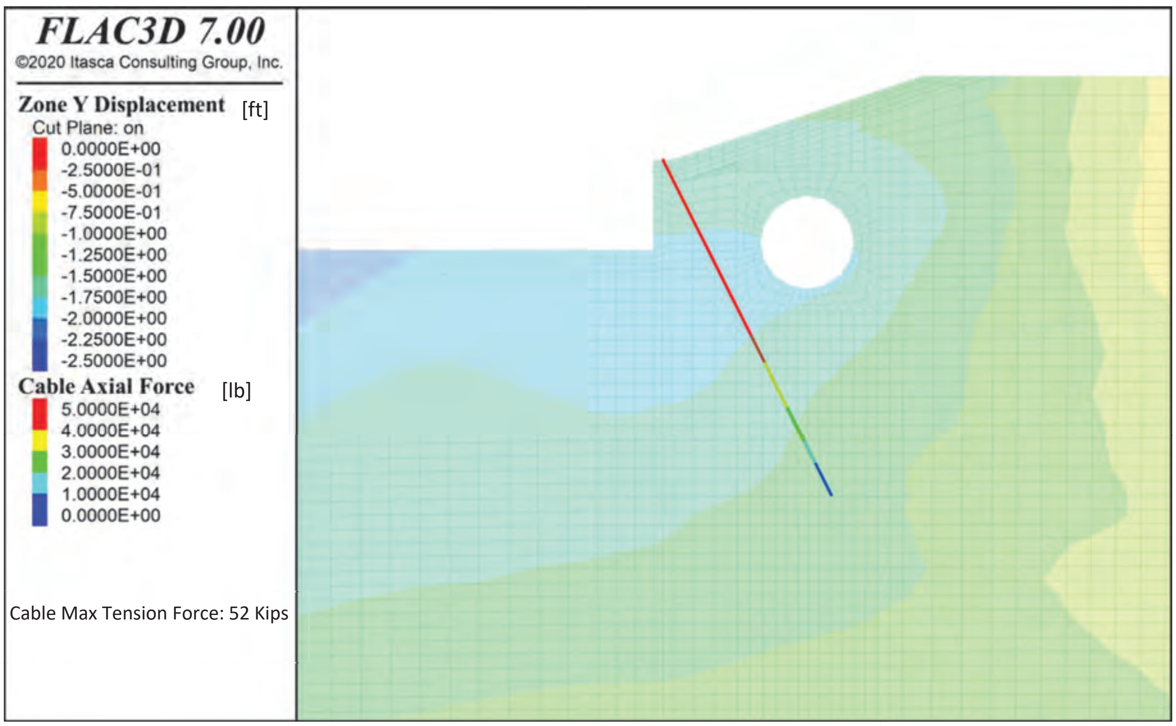


PORE WATER PRESSURE **(EARTHQUAKE SCENARIO-B AT t= 6.8 SEC. - BLUFF REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-16



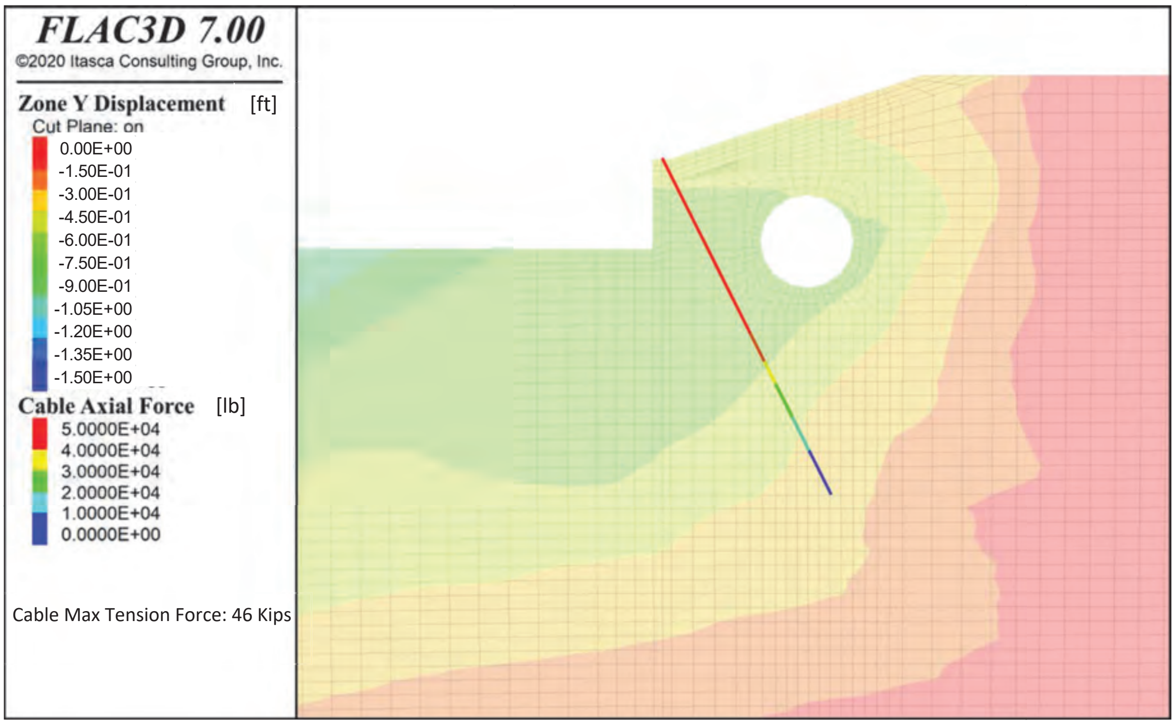
Note 1: Positive direction in Y direction is toward the slope (away from the ocean).
Note 2: Displacements are shown up to fixity depth of the pile.

DISPLACEMENT IN THE PILE OF THE WALL (MIDDLE PILE) **(BLUFF REACH - EARTHQUAKE SCENARIO B - BLUFF REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-17



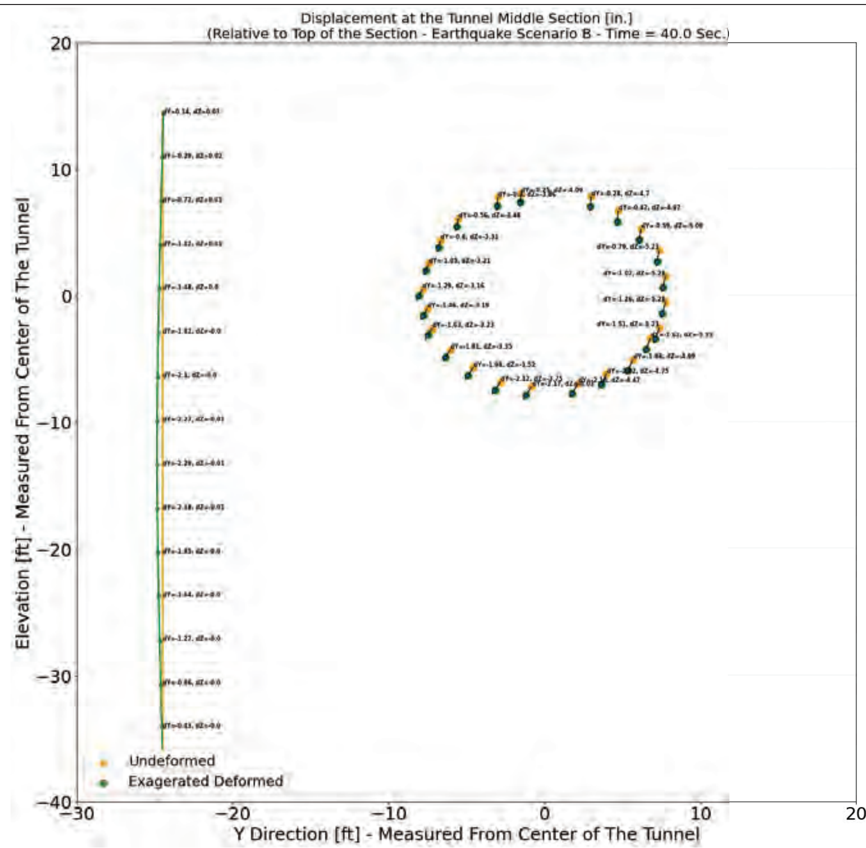
Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 11 SEC. - EARTHQUAKE SCENARIO B - BLUFF REACH) AND TENSILE FORCE IN THE TIE-BACK SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-18



Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 40 SEC. - EARTHQUAKE SCENARIO B - BLUFF REACH) AND TENSILE FORCE IN THE TIE-BACK SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-19

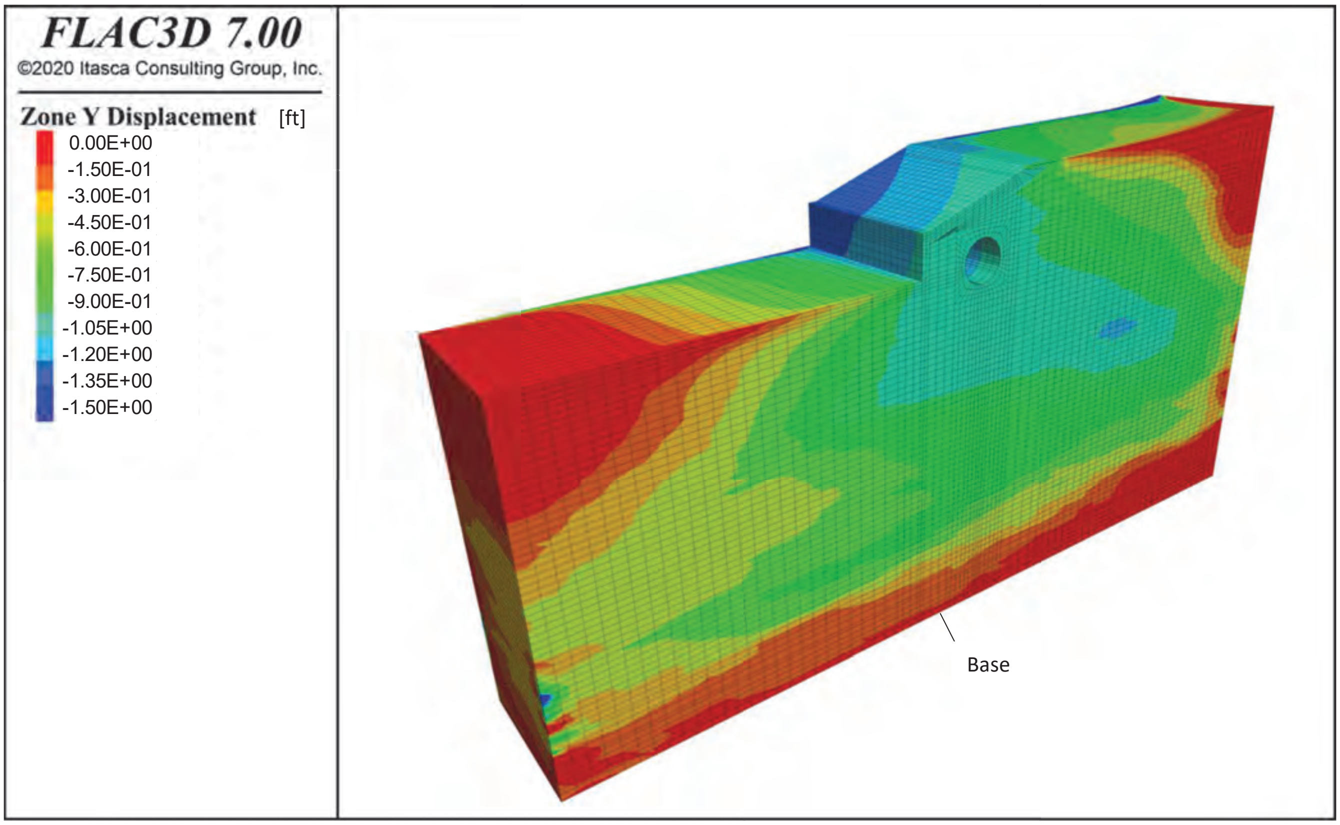


Note 1: Depicted displacements are only due to seismic simulation for the subject SCENARIO and are in addition to the gravitational loading.
Note 2: dY and dZ are relative displacement (relative to bottom of the pile) in Y and Z (elevation) directions, respectively.
Note 3: Displacements are exaggerated. Exaggeration factor in both Z and Y direction is 2.
Note 4: Positive direction in Y direction is toward the slope (away from the ocean).

TUNNEL AND PILE DISPLACEMENT
(AT t = 40.0 SEC. - EARTHQUAKE SCENARIO B BLUFF REACH)
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA

AGS

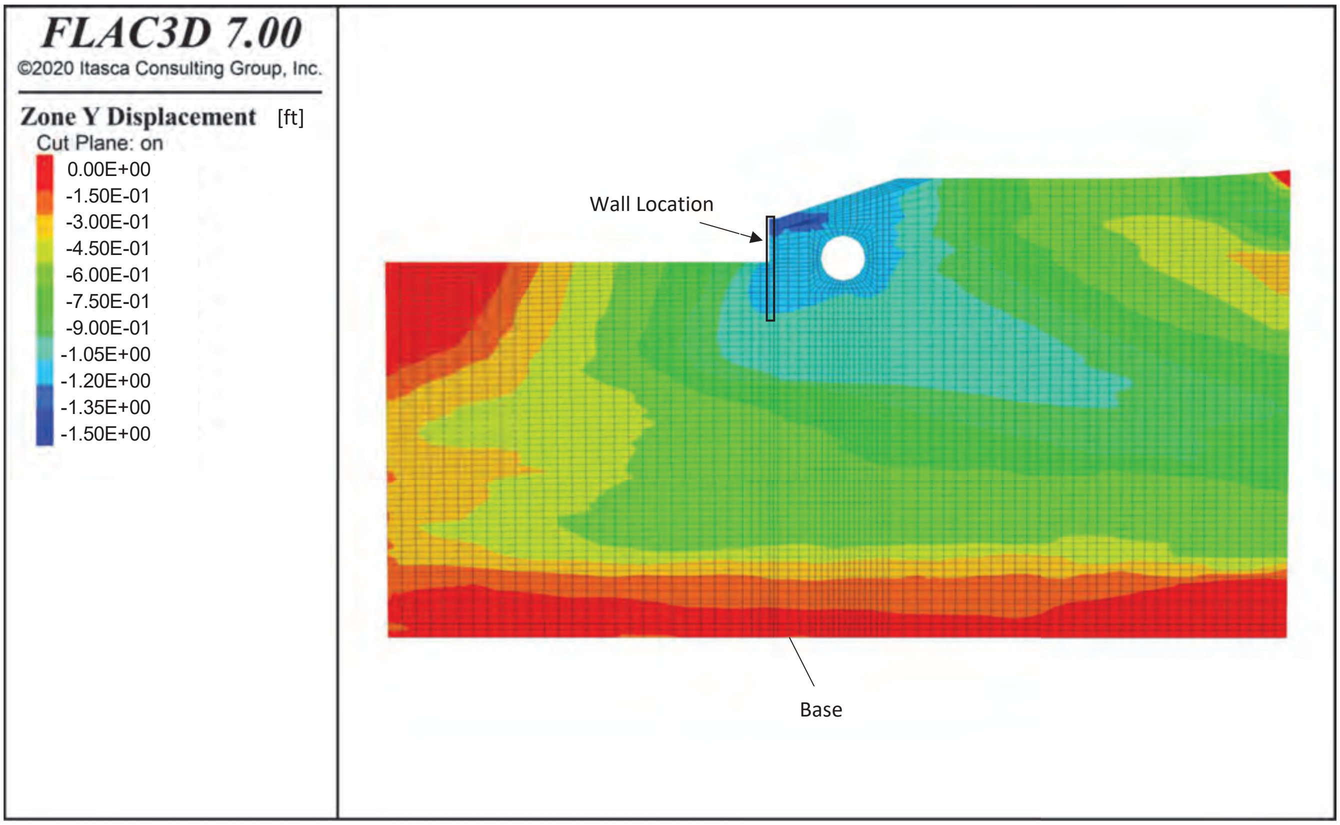
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-20



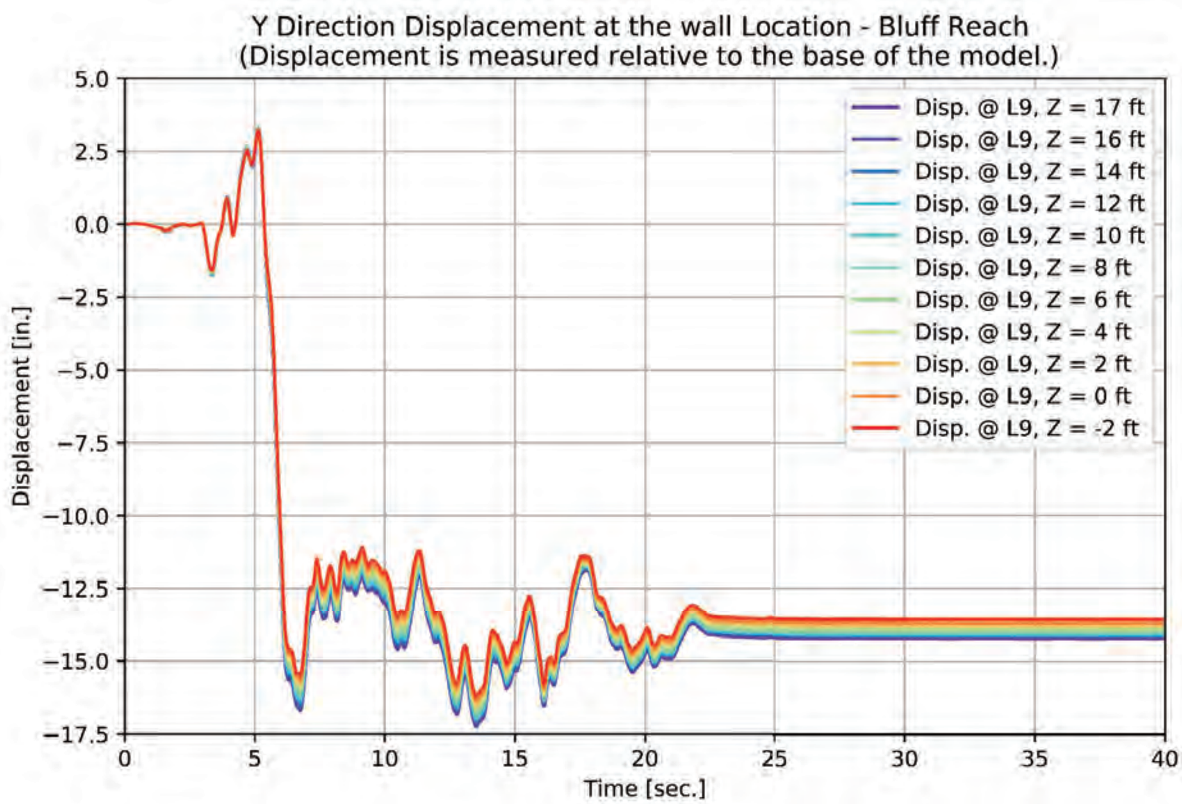
RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION
(AT t = 40 SEC. - EARTHQUAKE SCENARIO C - BLUFF REACH)
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-21

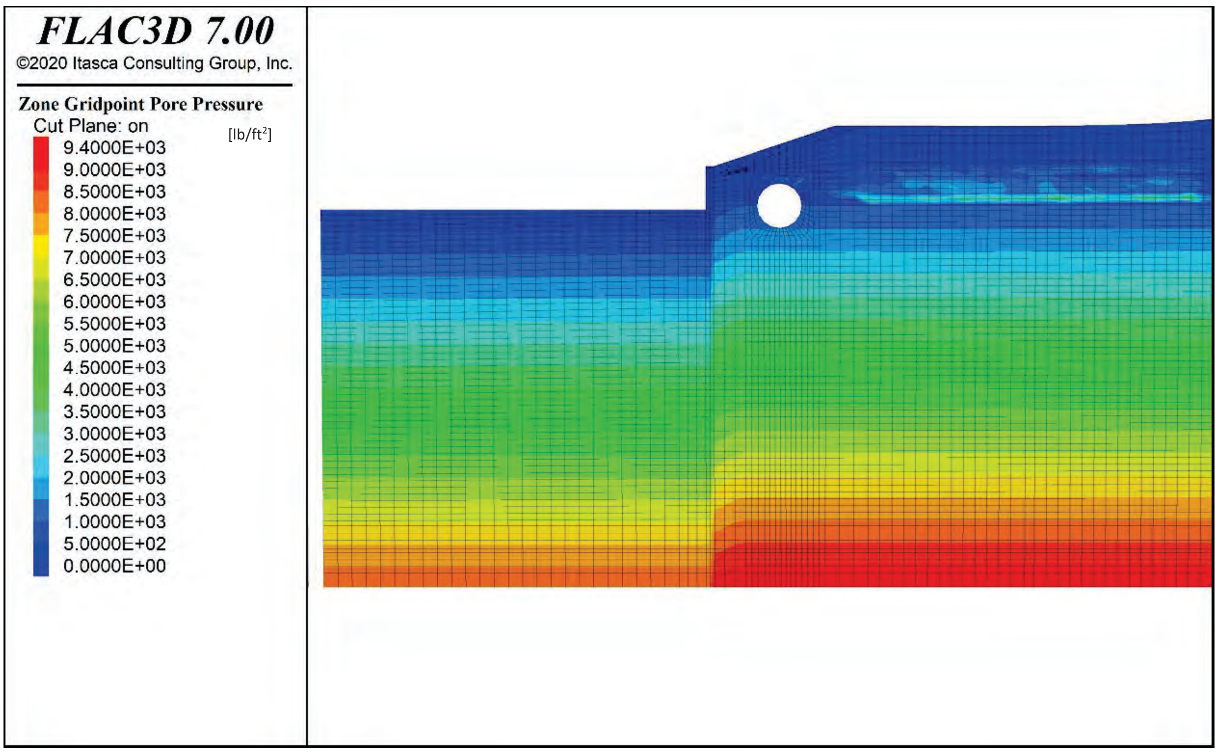


RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO C - BLUFF REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-22

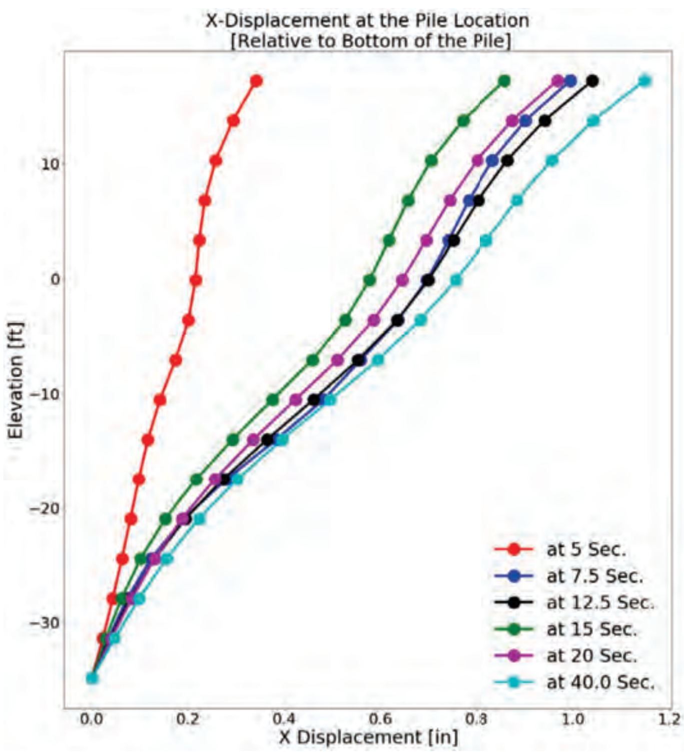


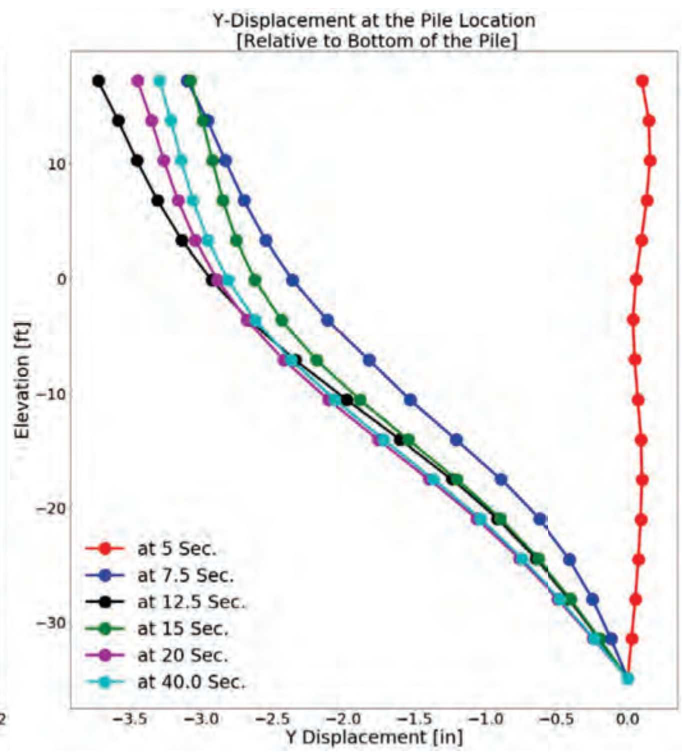
Note: Positive direction in Y direction is toward the slope (away from the ocean).

Y-DIRECTION DISPLACEMENT AT DIFFERENT ELEVATIONS OF THE WALL – MIDDLE SECTION **(EARTHQUAKE SCENARIO C - BLUFF REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-23



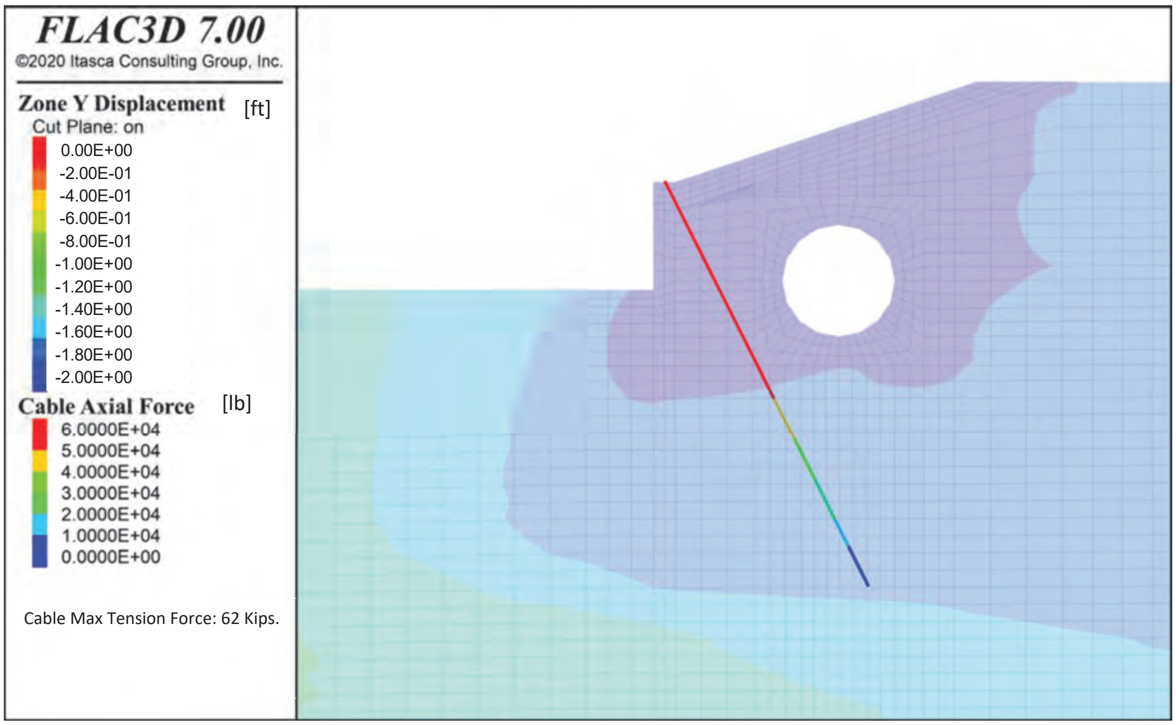
	PORE WATER PRESSURE **(EARTHQUAKE SCENARIO-C AT t= 6.0 SEC- BLUFF REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA	AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-24





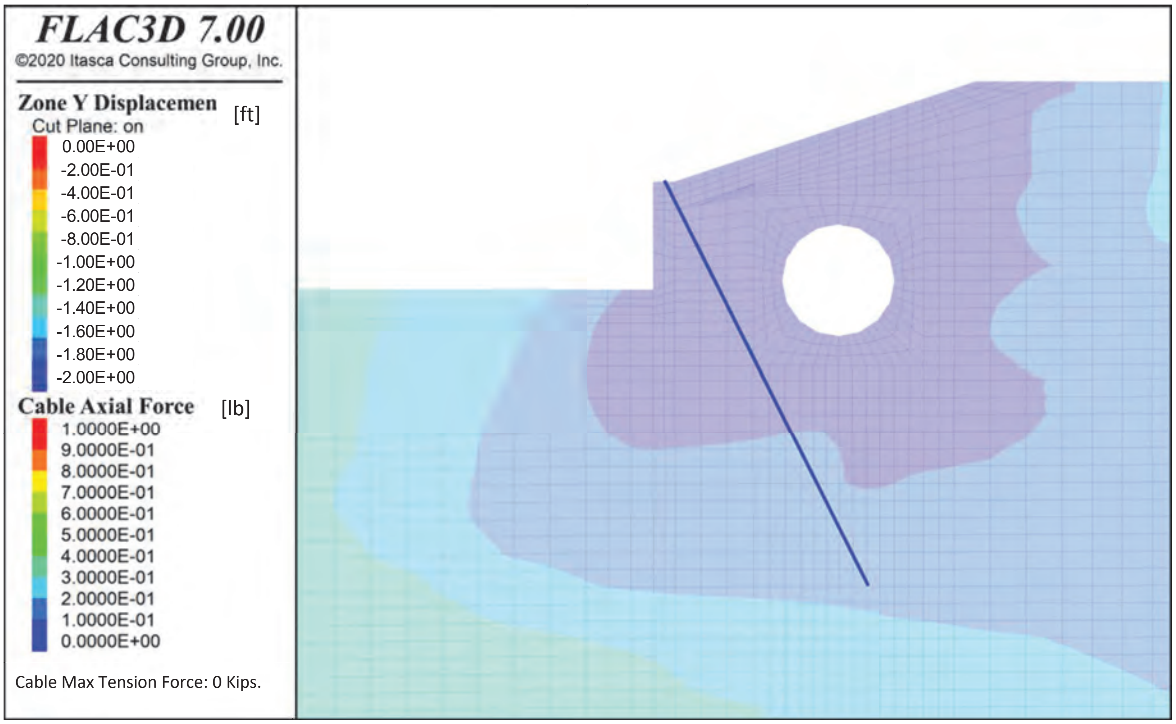
Note 1: Positive direction in Y direction is toward the slope (away from the ocean).
Note 2: Displacements are shown up to fixity depth of the pile.

DISPLACEMENT IN THE PILE OF THE WALL (MIDDLE PILE) **(BLUFF REACH - EARTHQUAKE SCENARIO C - BLUFF REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-25



Note: Y Displacement contours shown in transparent.

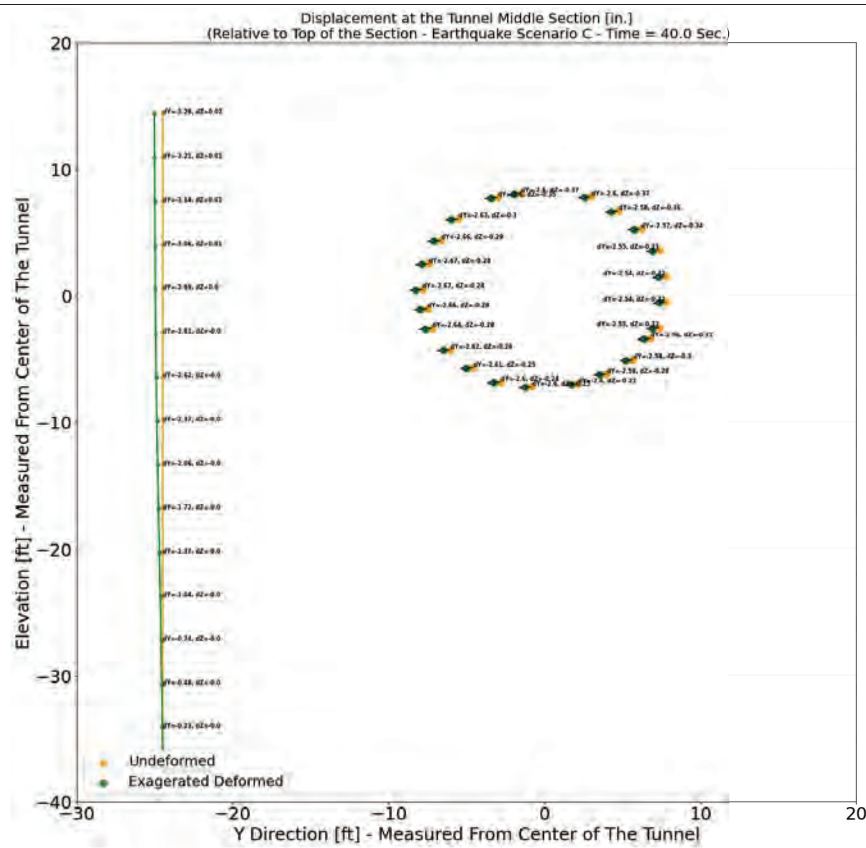
RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 6.4 SEC. - EARTHQUAKE SCENARIO C - BLUFF REACH) AND TENSILE FORCE IN THE TIE-BACK SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-26



Note: Y Displacement contours shown in transparent.

RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40.0 SEC. - EARTHQUAKE SCENARIO C - BLUFF REACH)** **AND TENSILE FORCE IN THE TIE-BACK** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS

JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-27

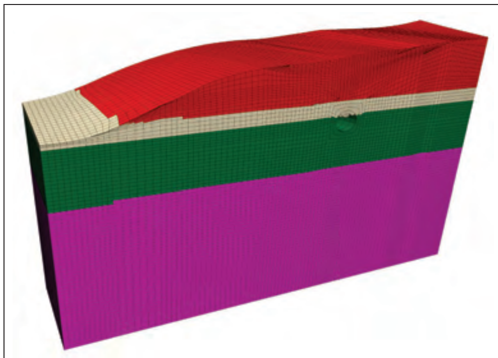


Note 1: Depicted displacements are only due to seismic simulation for the subject SCENARIO and are in addition to the gravitational loading.
Note 2: dY and dZ are relative displacement (relative to bottom of the pile) in Y and Z (elevation) directions, respectively.
Note 3: Displacements are exaggerated. Exaggeration factor in both Z and Y direction is 2.
Note 4: Positive direction in Y direction is toward the slope (away from the ocean).

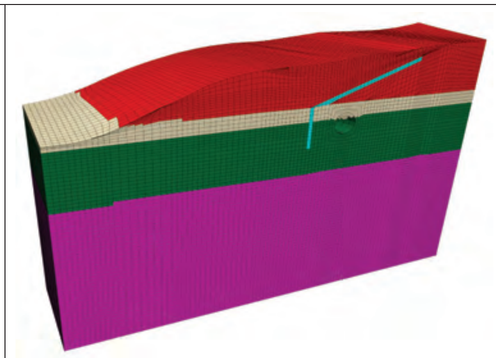
TUNNEL AND PILE DISPLACEMENT **(AT t = 40.0 SEC. - EARTHQUAKE SCENARIO C BLUFF REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE E-28

Appendix F

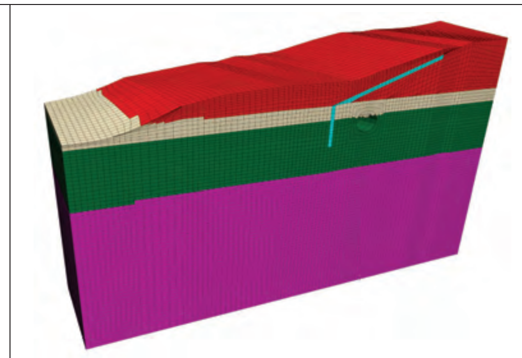
FLAC3D NUMERICAL SSSI DYNAMIC ANALYSES RESULTS FOR NORTH REACH



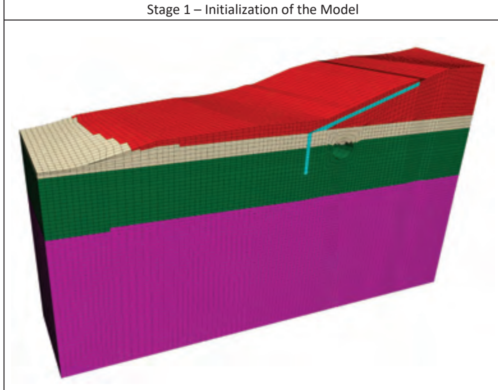
Stage 1 – Initialization of the Model



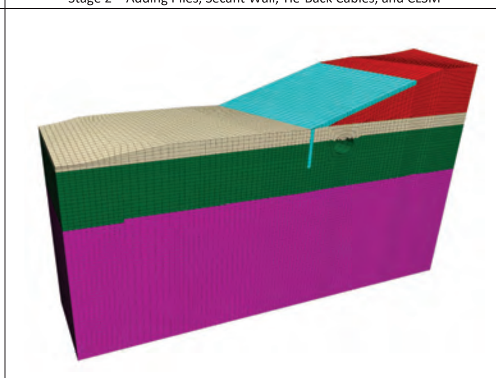
Stage 2 – Adding Piles, Secant Wall, Tie-Back Cables, and CLSM



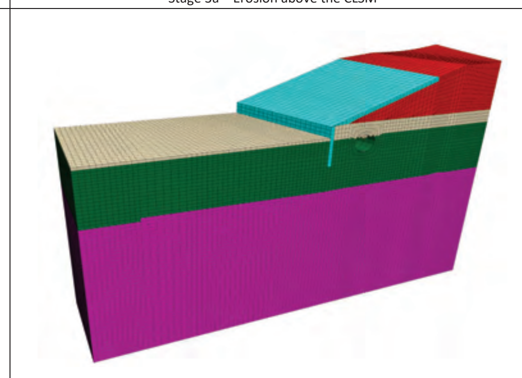
Stage 3a – Erosion above the CLSM



Stage 3b – Erosion above the CLSM



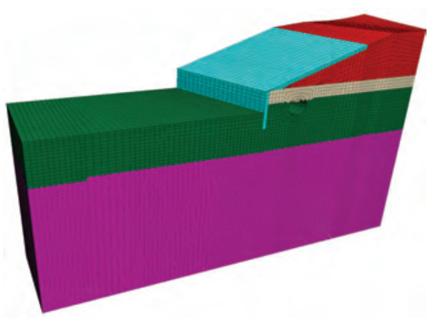
Stage 3c – Erosion above the CLSM



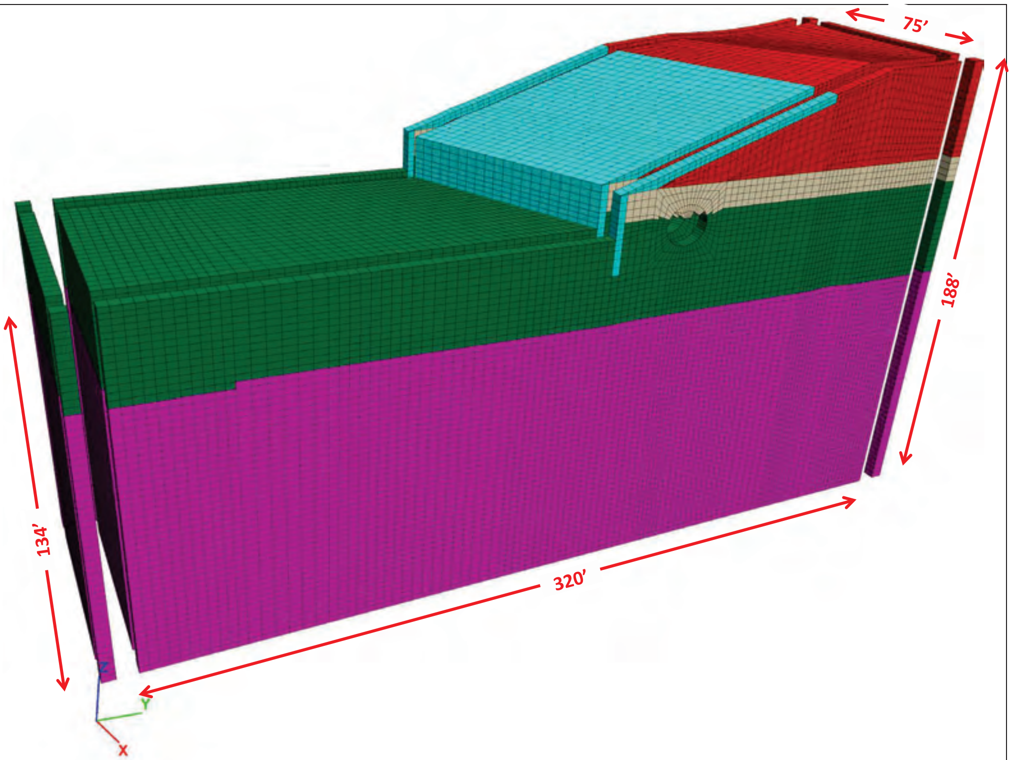
Stage 4 – Erosion in front of the Secant Wall

CLSM and Wall
Fill
Liquefiable Layer
Colma Formation
Merced Formation

SOUTH REACH MESH – DIFFERENT STAGES OF THE NUMERICAL SIMULATION SOUTH OCEAN BEACH C OASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE F-1 a



Stage 5 – Erosion in front of the Secant Wall



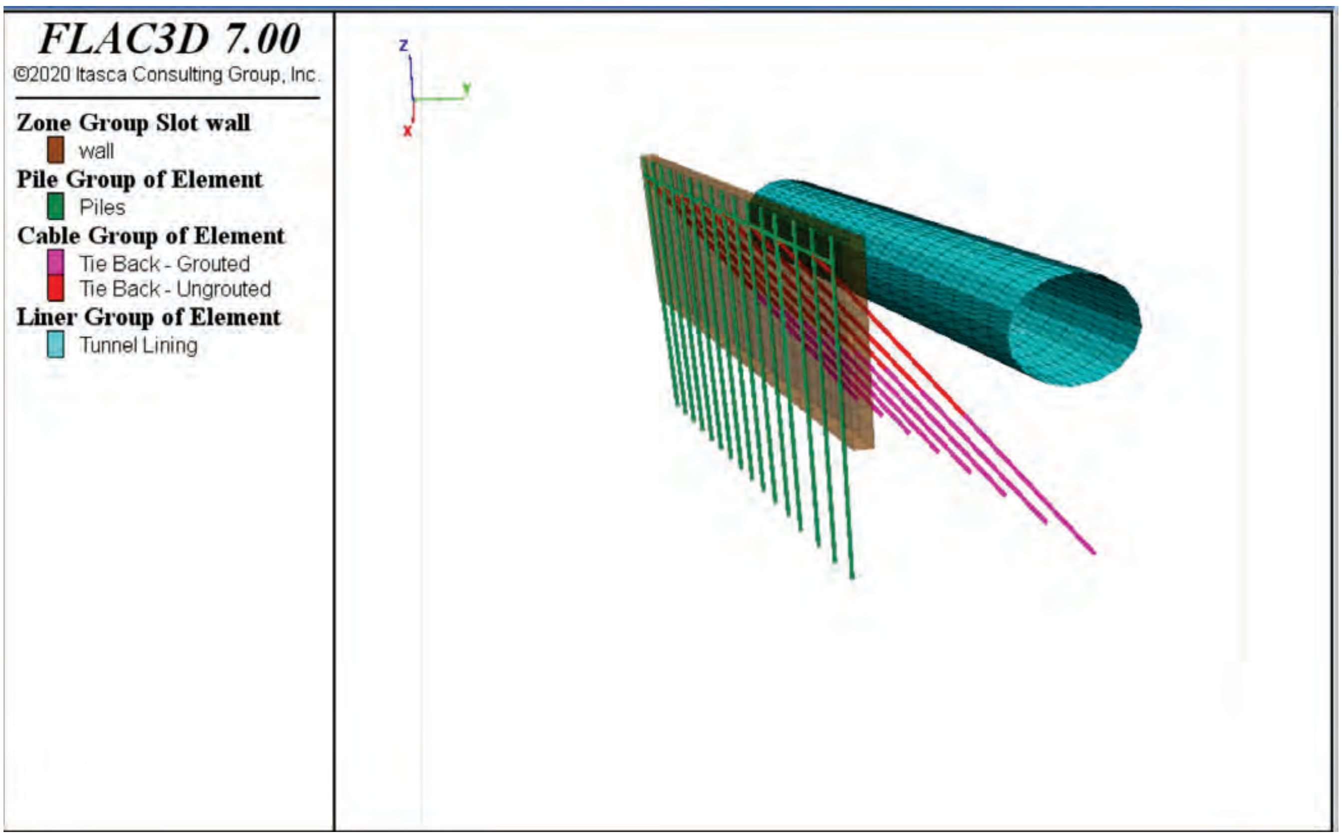
Mesh Used to Seismic Analysis – Includes Free Field Boundary Conditions

CLSM and Wall	Liquefiable Layer
Fill	Colma Formation
	Merced Formation

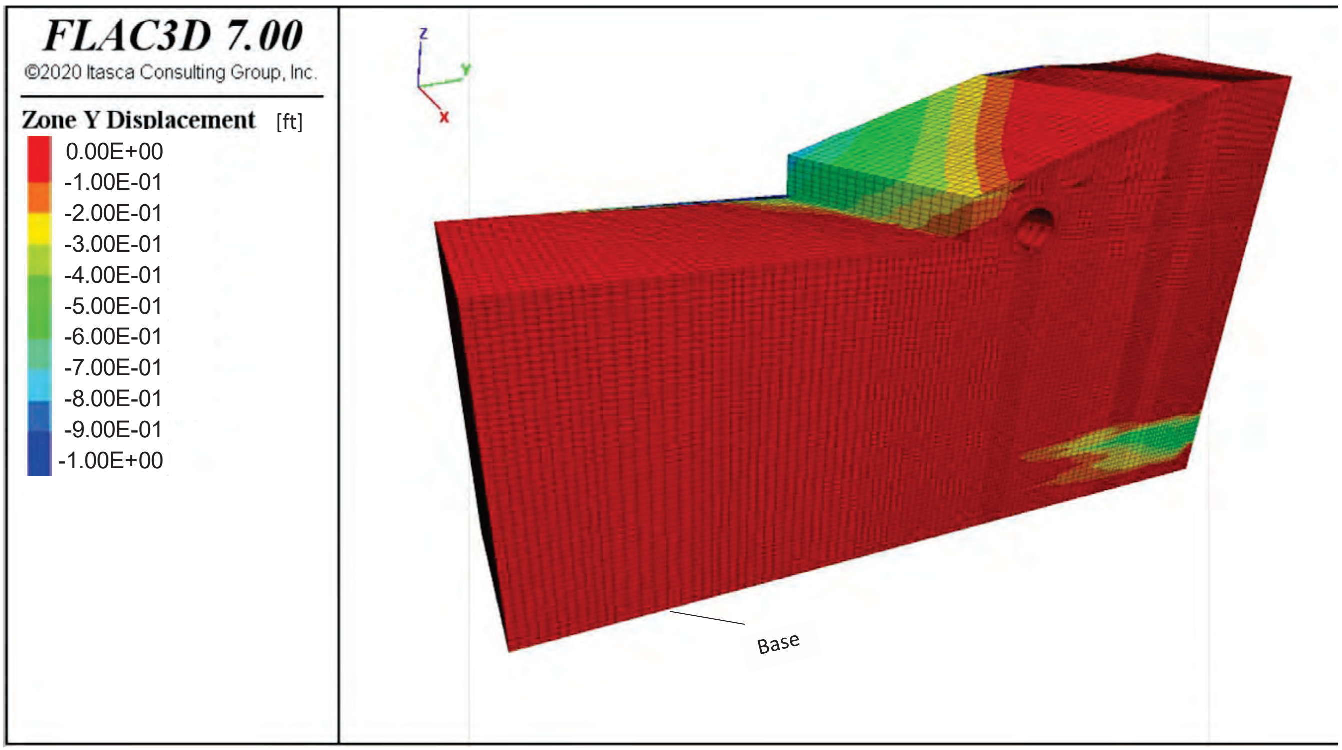
SOUTH REACH MESH – DIFFERENT STAGES OF THE NUMERICAL SIMULATION
SOUTH OCEAN BEACH C OASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION
SAN FRANCISCO, CALIFORNIA

AGS

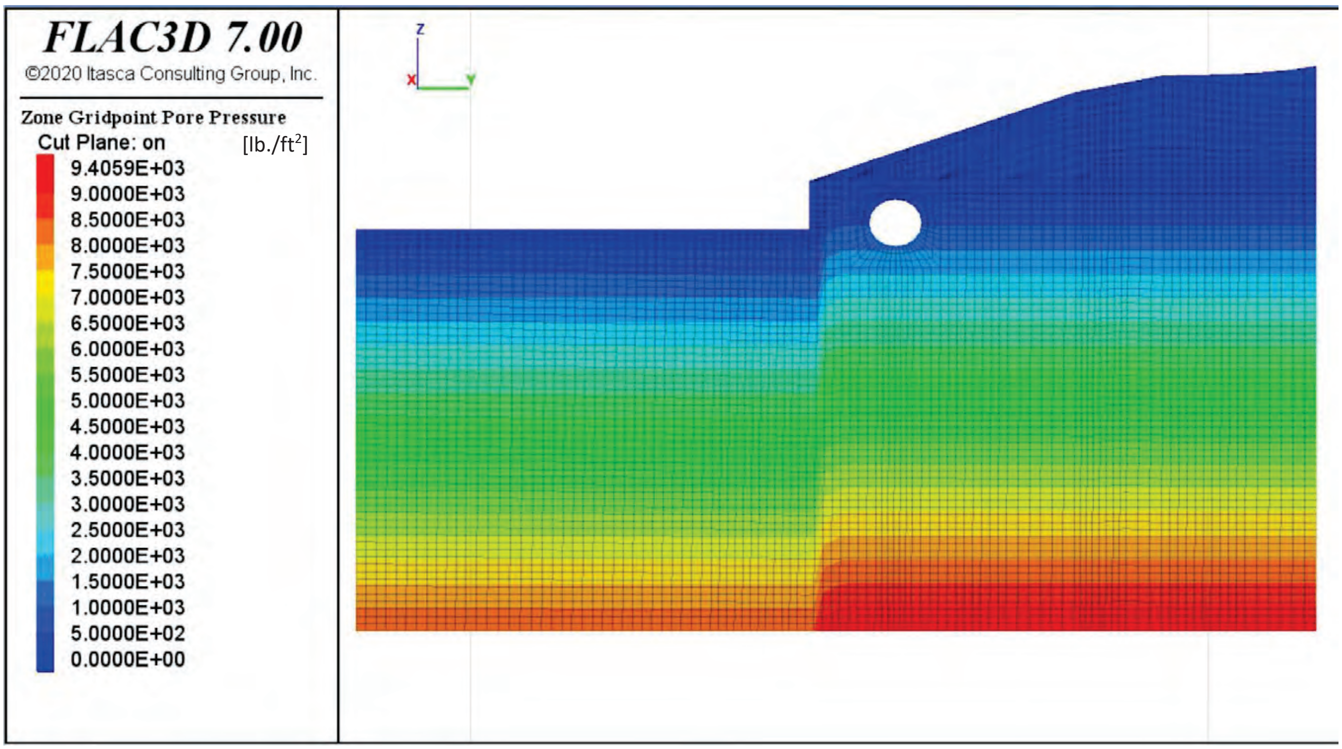
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE F-1 b



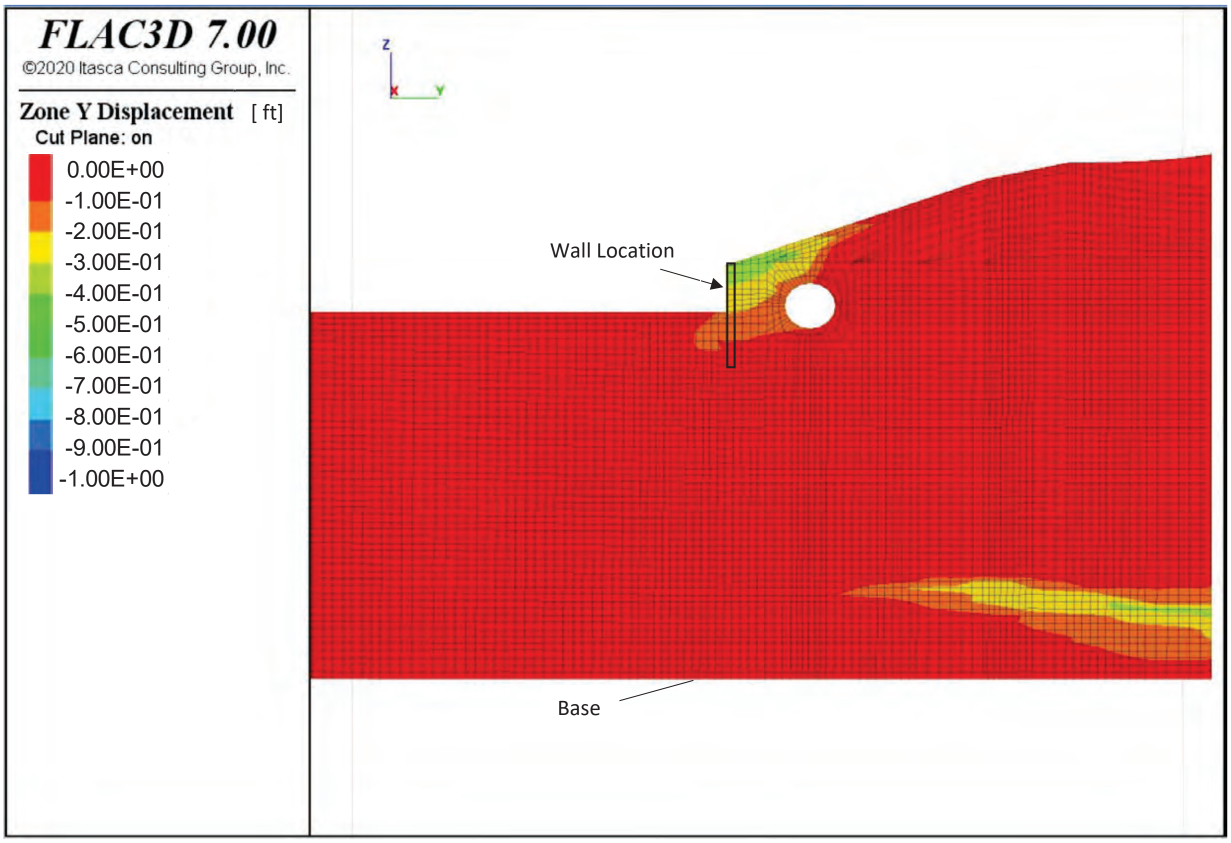
STRUCTURAL ELEMENTS OF THE MODEL **(EARTHQUAKE SCENARIO A - SOUTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE F-2



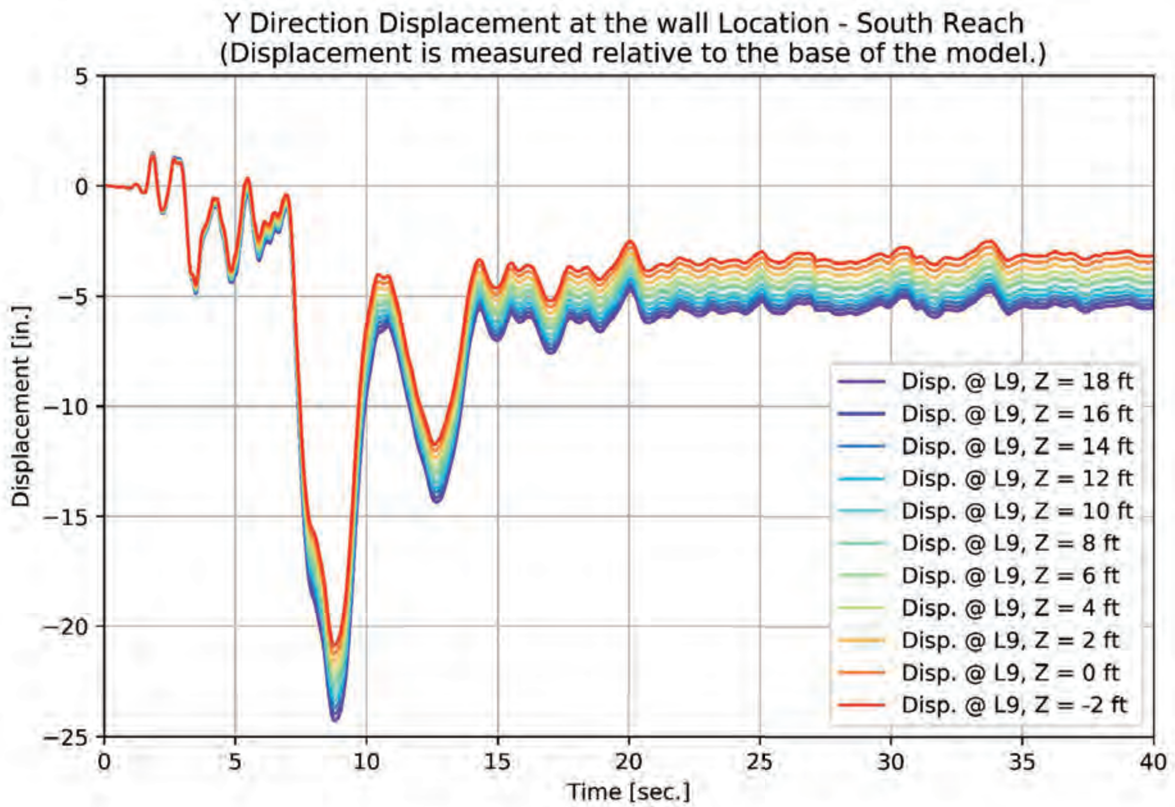
RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO A - SOUTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE F-3



HYDROSTATIC PORE-WATER PRESSURE PRIOR TO SHAKING (SOUTH REACH) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS
JOB NO. AGS-18-003 DATE: MAY 2021	PLATE F-4

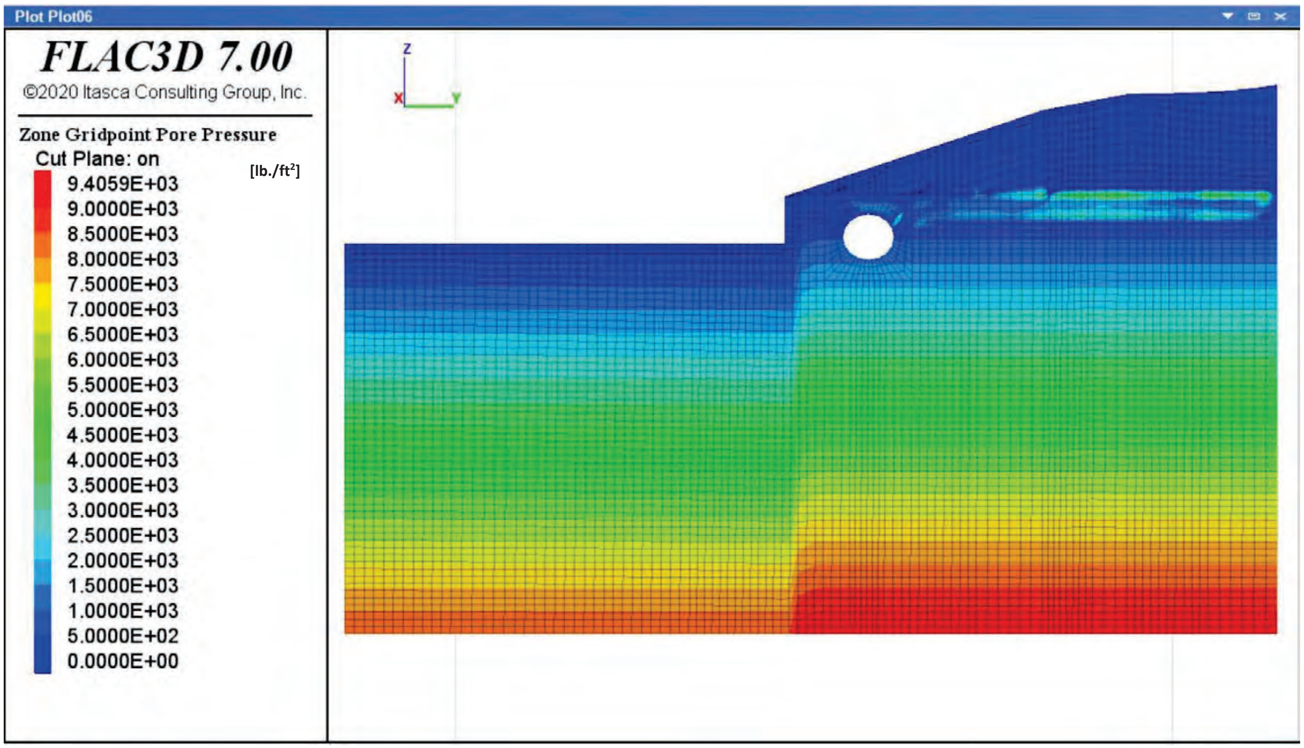


RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 40 SEC. - EARTHQUAKE SCENARIO A - SOUTH REACH) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE F-5

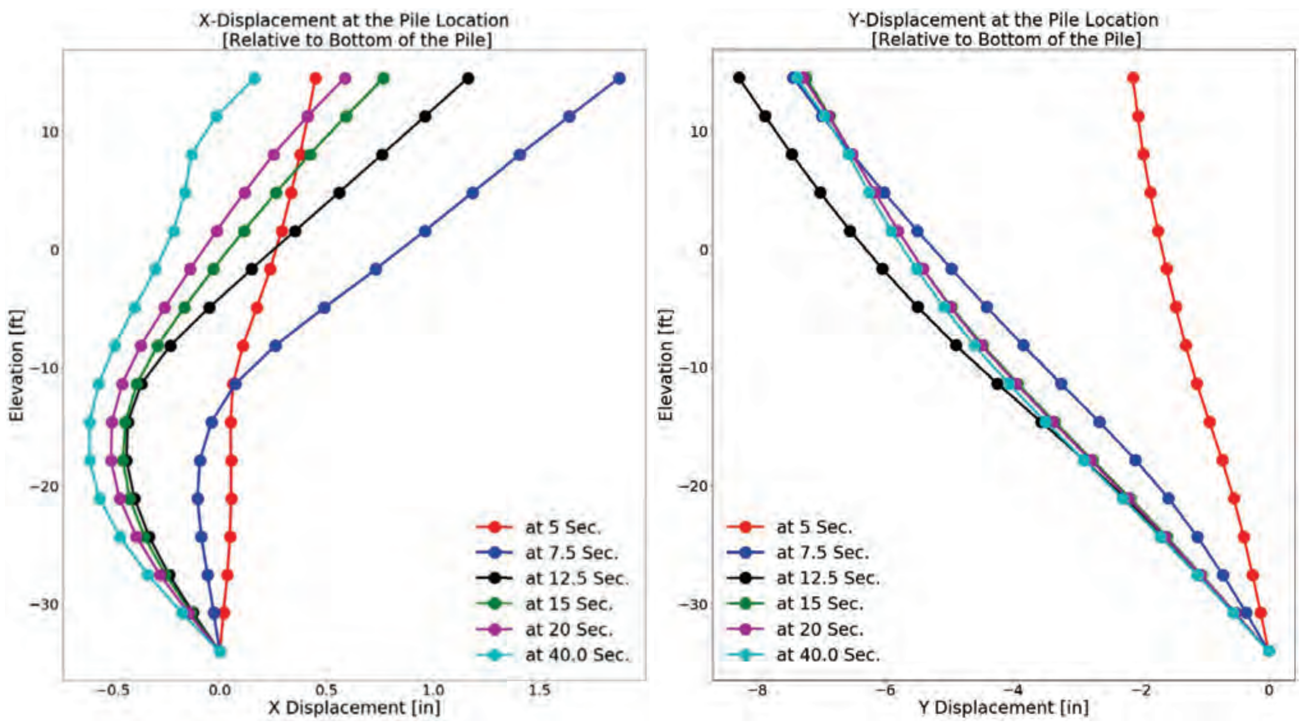


Note: Positive direction in Y direction is toward the slope (away from the ocean).

Y-DIRECTION DISPLACEMENT AT DIFFERENT ELEVATIONS– MIDDLE SECTION (EXPOSED PORTION OF THE SECANT WALL- EARTHQUAKE SCENARIO A - SOUTH REACH) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE F-6

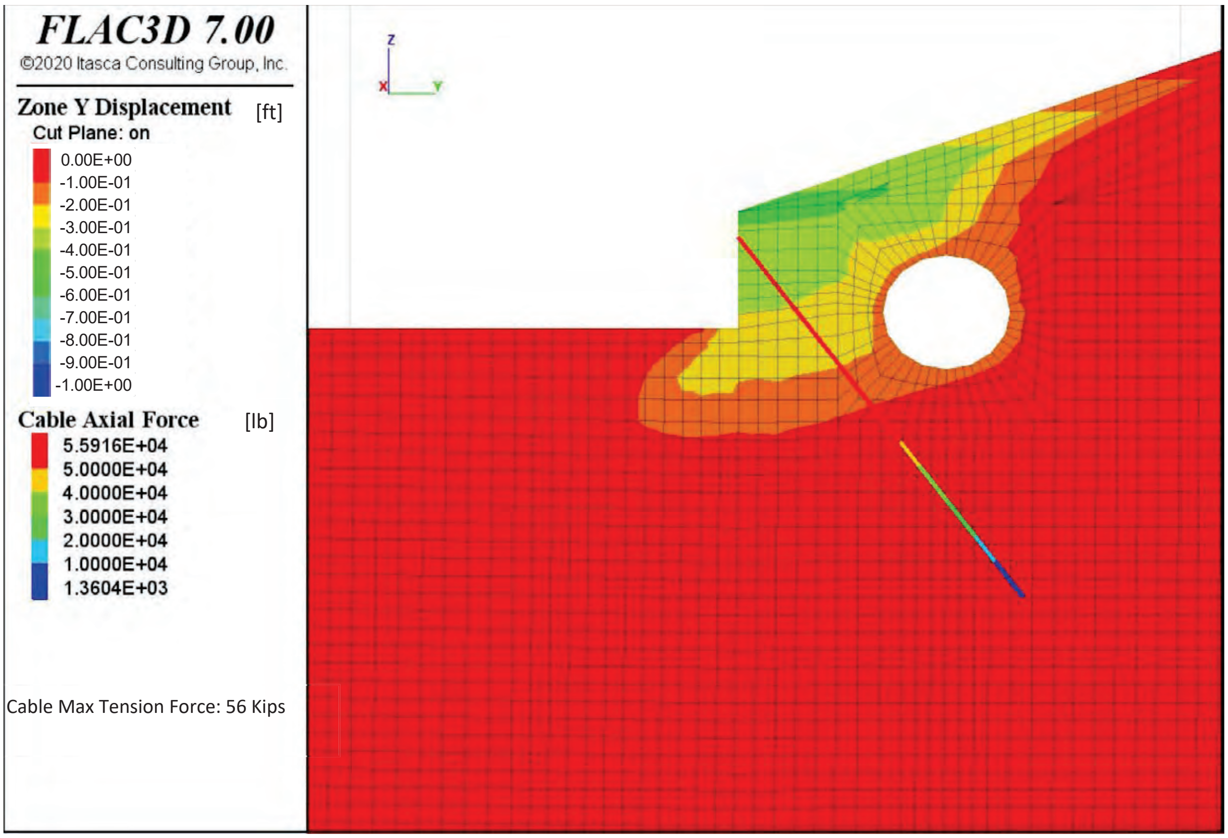


	PORE WATER PRESSURE **(EARTHQUAKE SCENARIO-A AT t= 7.4 SEC.- SOUTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE F-7



Note 1: Positive direction in Y direction is toward the slope (away from the ocean).
Note 2: Displacements are shown up to fixity depth of the pile.

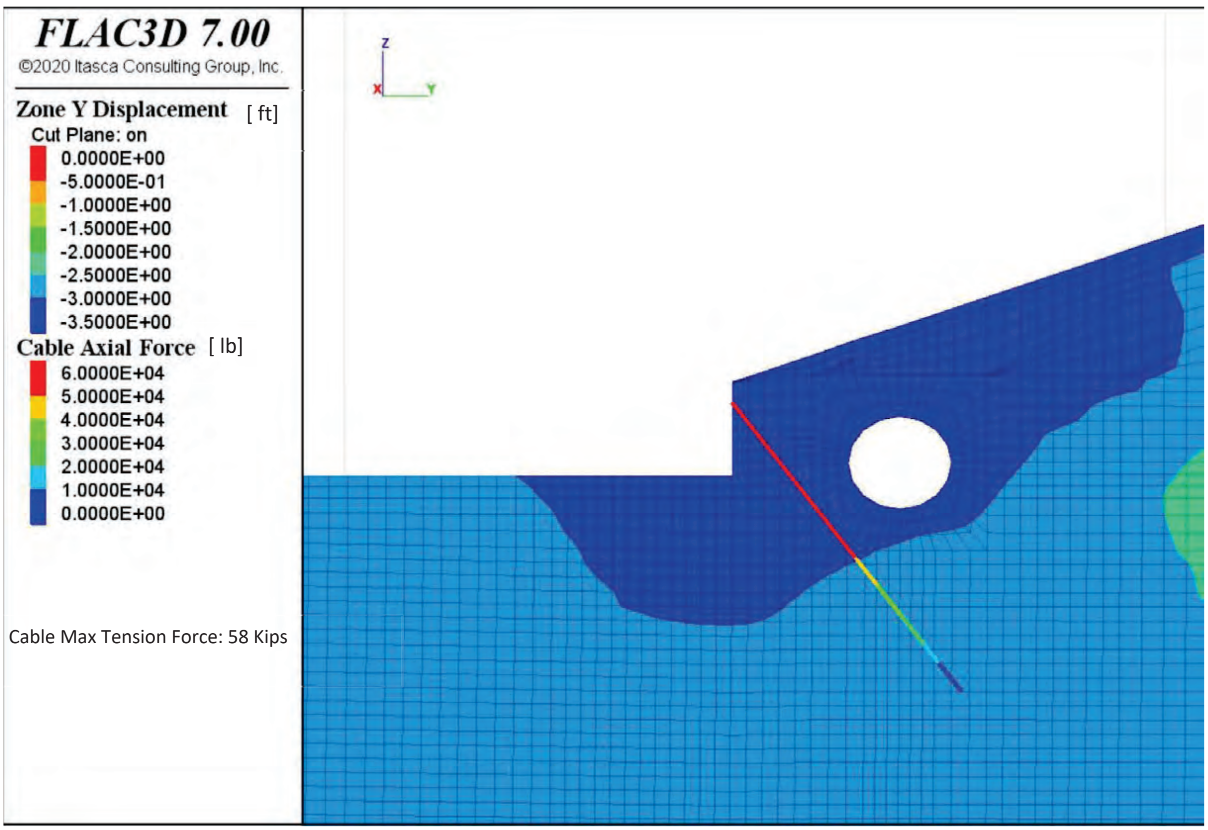
DISPLACEMENT IN THE PILE OF THE WALL (MIDDLE PILE) **(SOUTH REACH - EARTHQUAKE SCENARIO A - SOUTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE F-8



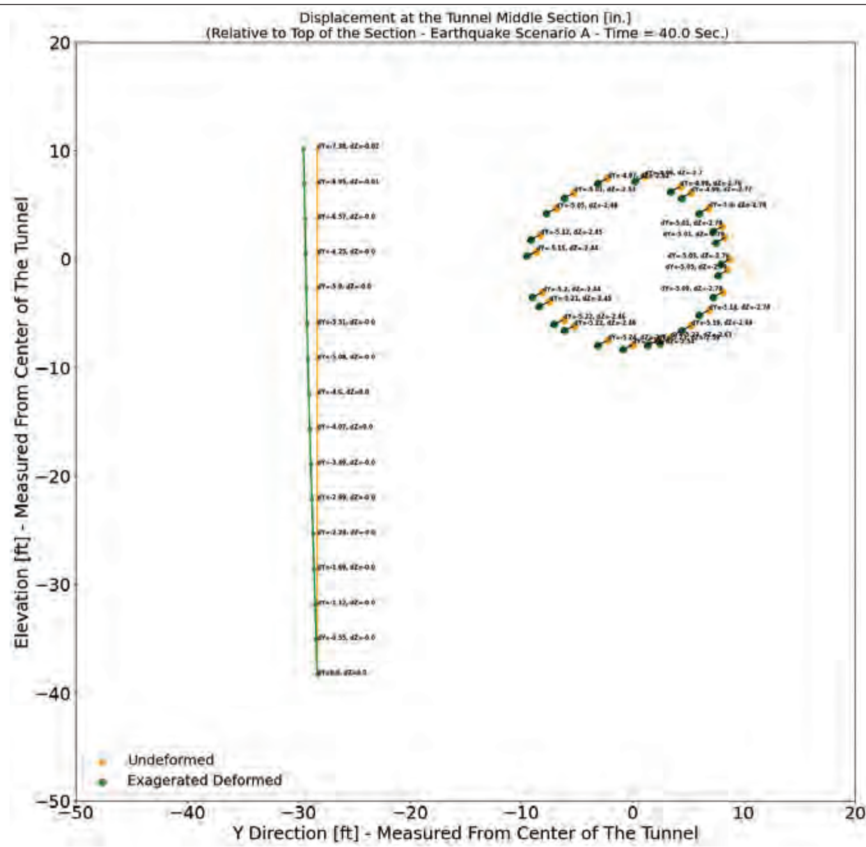
RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION
(AT t = 40 SEC. - EARTHQUAKE SCENARIO A - SOUTH REACH)
AND TENSILE FORCE IN THE TIE-BACK
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: MAY 2021	PLATE F-10



RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 12.5 SEC. - EARTHQUAKE SCENARIO A - SOUTH REACH) AND TENSILE FORCE IN THE TIE-BACK SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE F-9

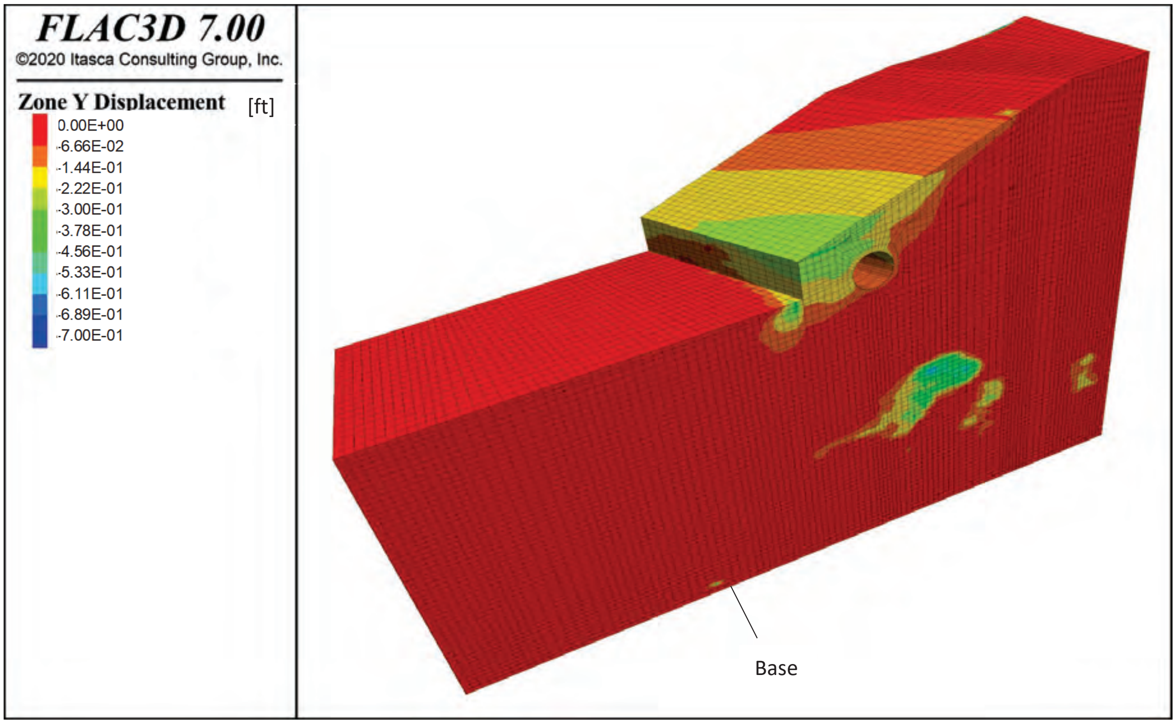


Note 1: Depicted displacements are only due to seismic simulation for the subject SCENARIO and are in addition to the gravitational loading.
Note 2: dY and dZ are relative displacement (relative to bottom of the pile) in Y and Z (elevation) directions, respectively.
Note 3: Displacements are exaggerated. Exaggeration factor in both Z and Y direction is 2.
Note 4: Positive direction in Y direction is toward the slope (away from the ocean).

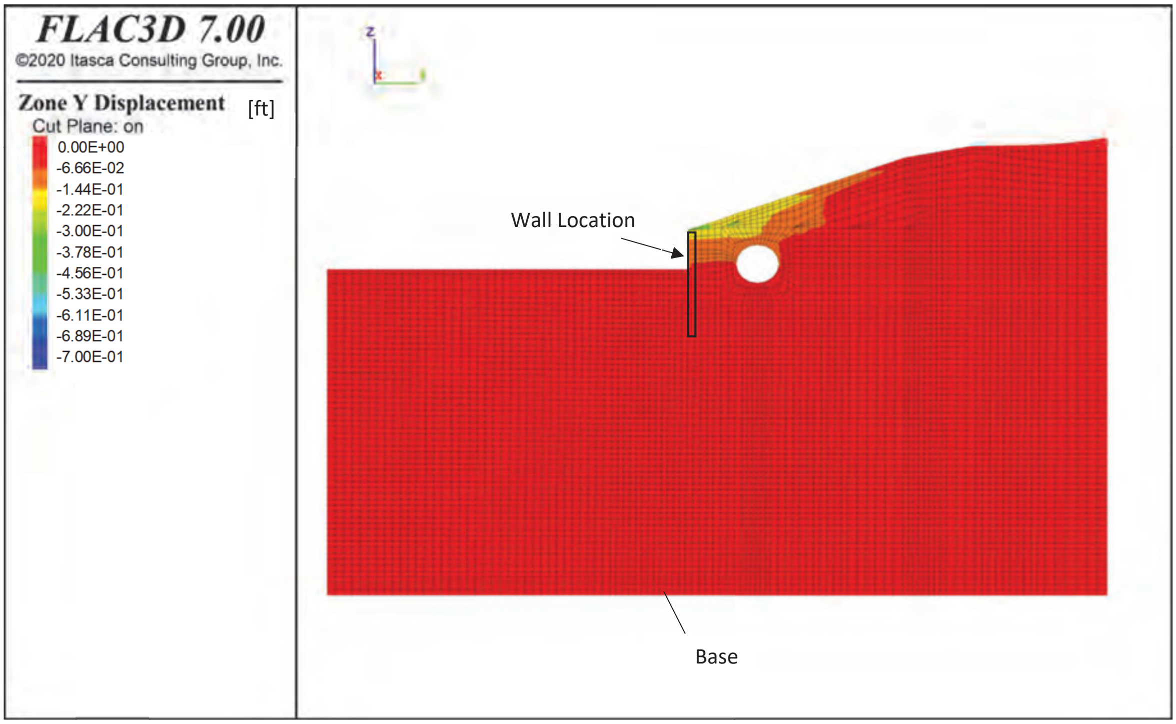
TUNNEL AND PILE DISPLACEMENT
(AT t = 40.0 SEC. - EARTHQUAKE SCENARIO A SOUTH REACH)
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA

AGS

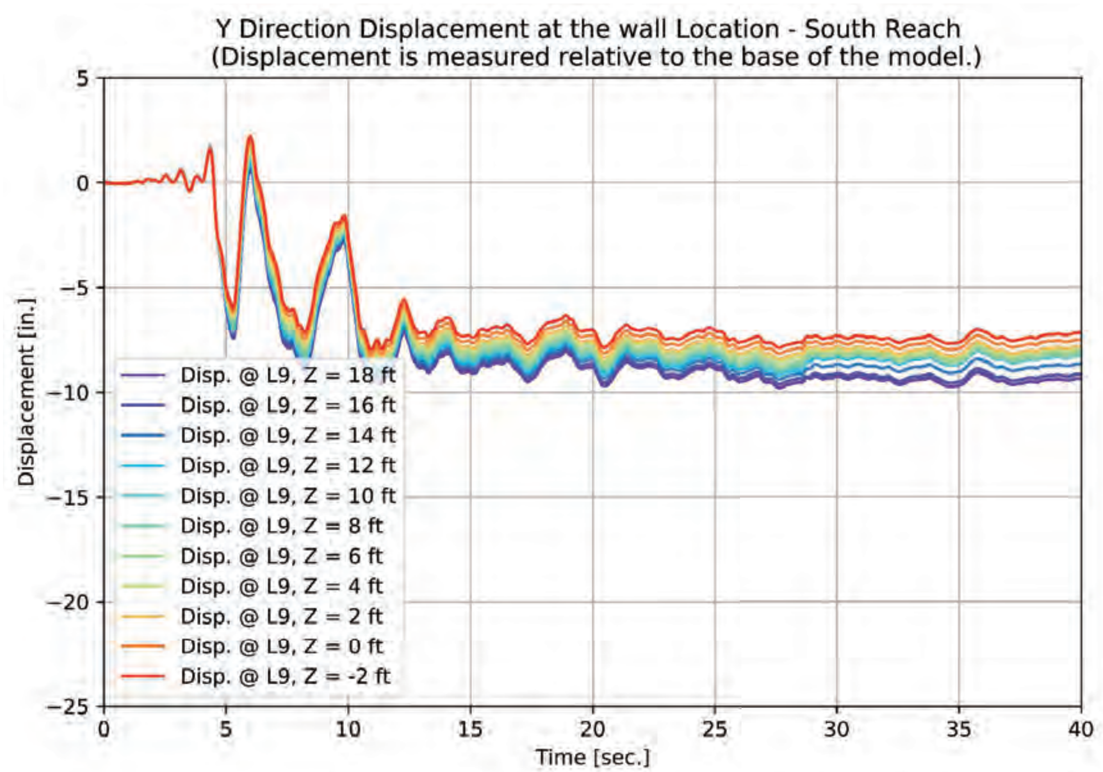
JOB NO. AGS-18-003	DATE: MAY 2021	PLATE F-12



RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 40 SEC. - EARTHQUAKE SCENARIO B - SOUTH REACH) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE F-13

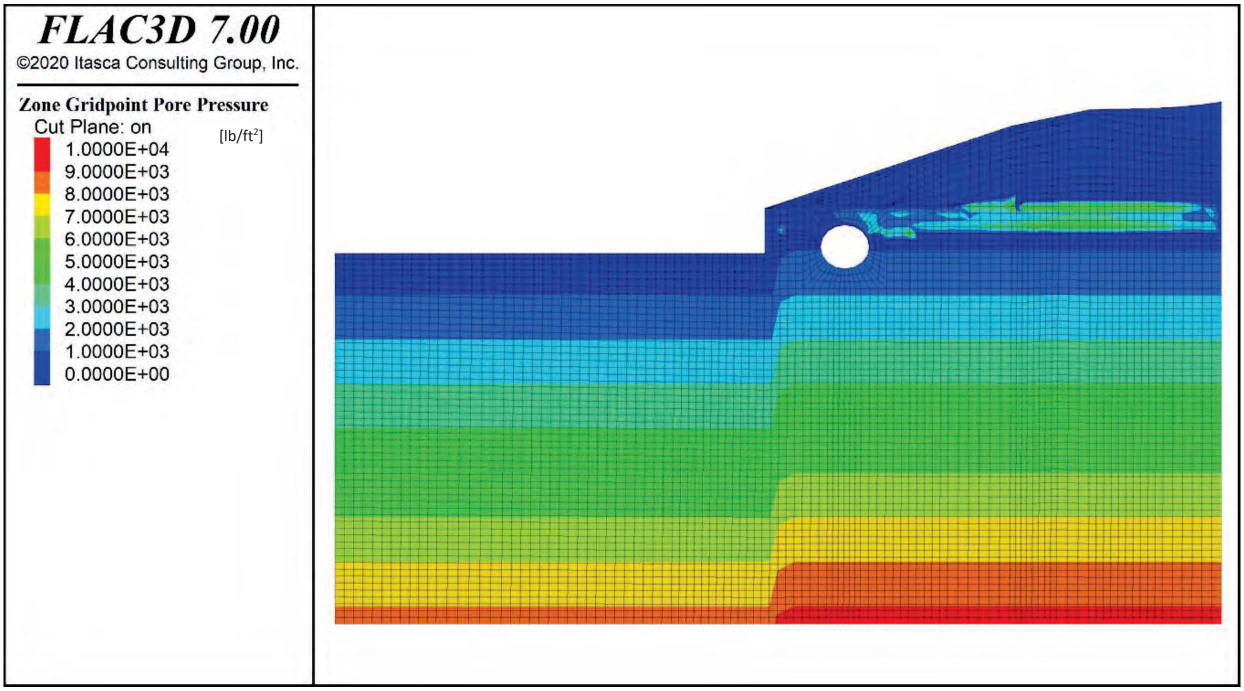


RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO B - SOUTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE F-14

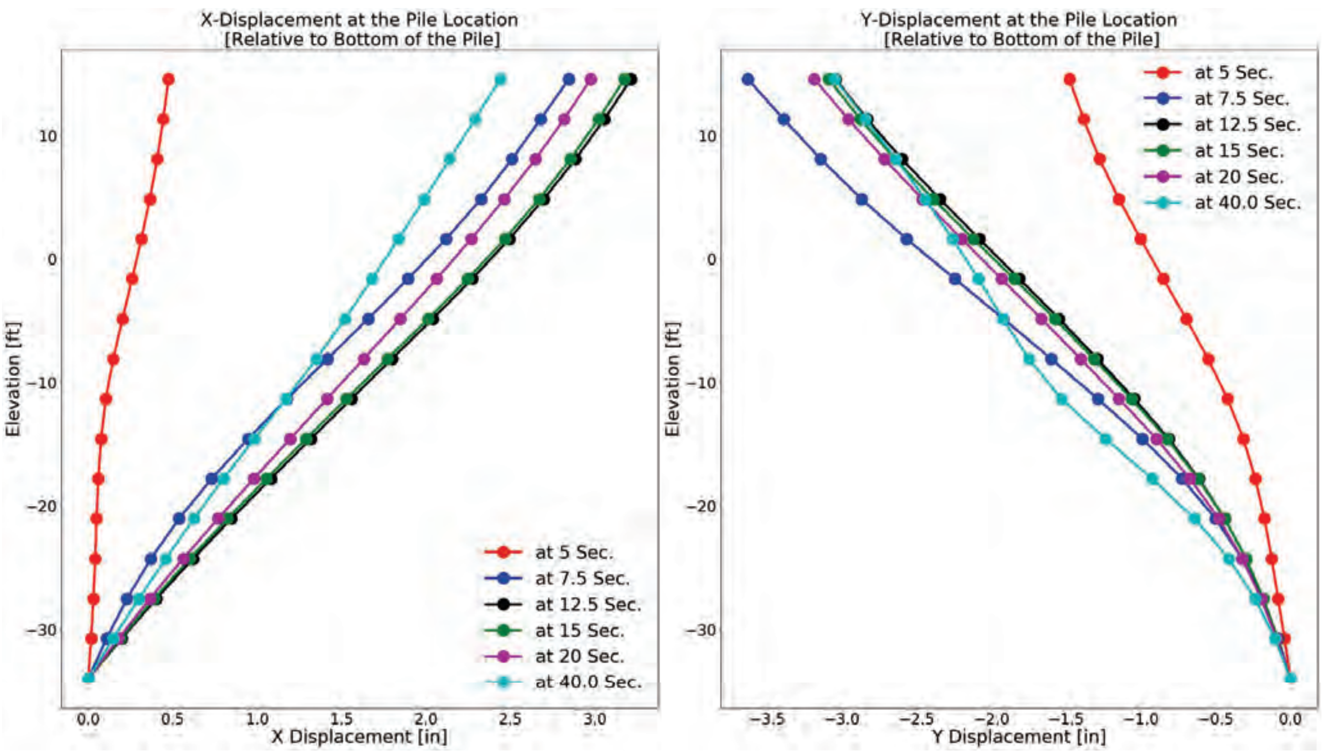


Note: Positive direction in Y direction is toward the slope (away from the ocean).

Y-DIRECTION DISPLACEMENT AT DIFFERENT ELEVATIONS– MIDDLE SECTION (EXPOSED PORTION OF THE SECANT WALL- EARTHQUAKE SCENARIO B - SOUTH REACH) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE F-15

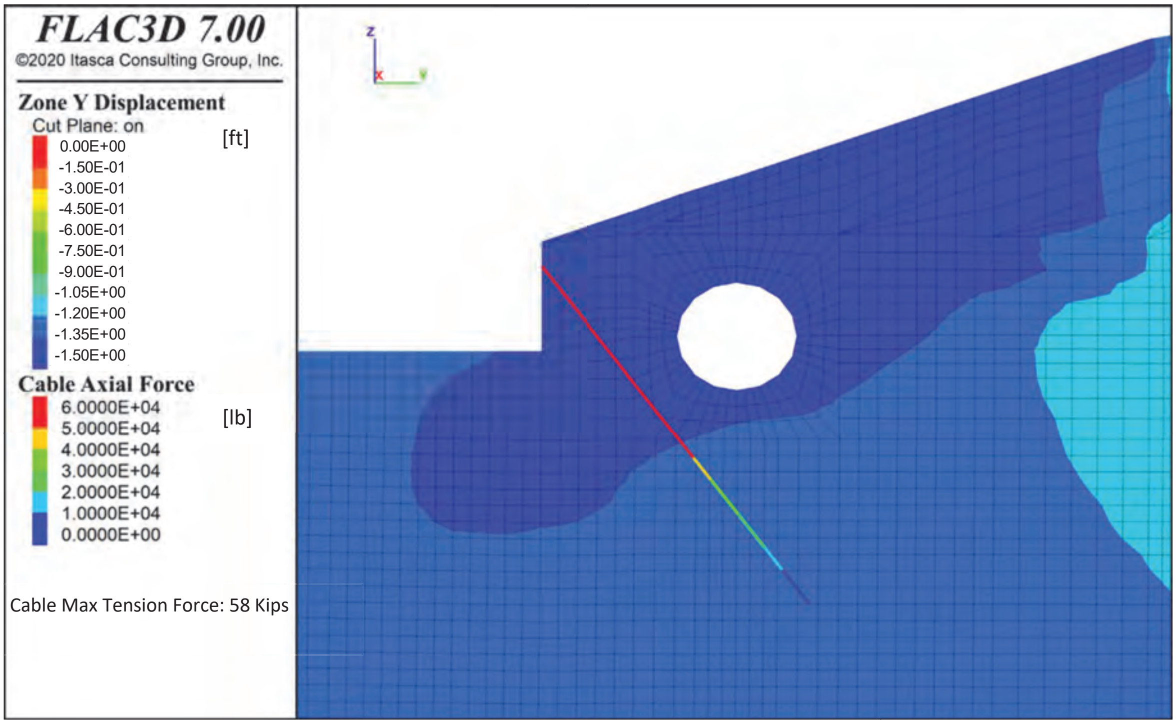


	PORE WATER PRESSURE **(EARTHQUAKE SCENARIO-B AT t= 11.0 SEC. - SOUTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA	AGS
	JOB NO. AGS-18-003 DATE: JUNE 2021	PLATE F-16



Note 1: Positive direction in Y direction is toward the slope (away from the ocean).
Note 2: Displacements are shown up to fixity depth of the pile.

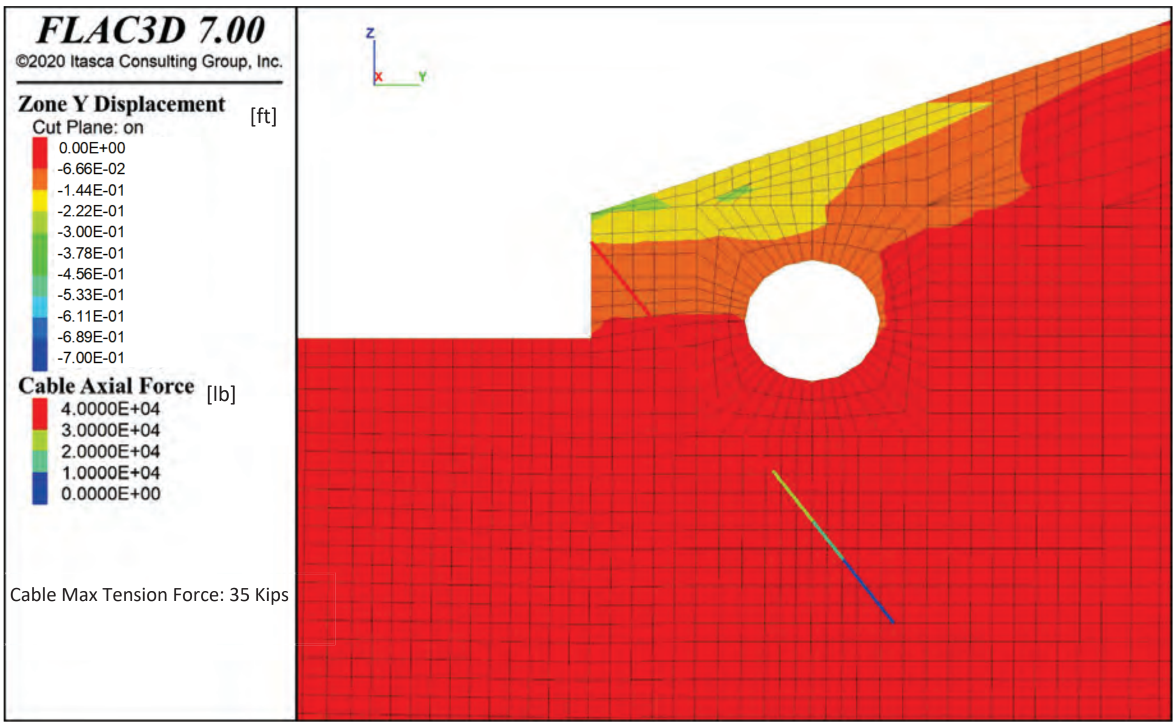
DISPLACEMENT IN THE PILE OF THE WALL (MIDDLE PILE) **(SOUTH REACH - EARTHQUAKE SCENARIO B - SOUTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE F-17



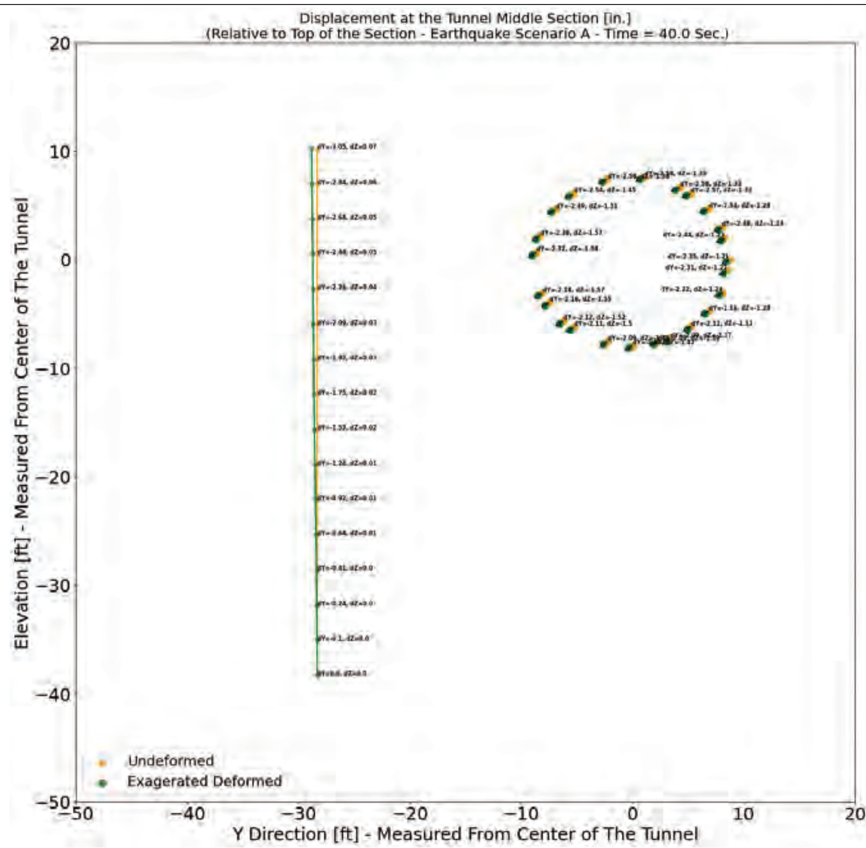
RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION
(AT t = 11 SEC. - EARTHQUAKE SCENARIO B - SOUTH REACH)
AND TENSILE FORCE IN THE TIE-BACK
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE F-18

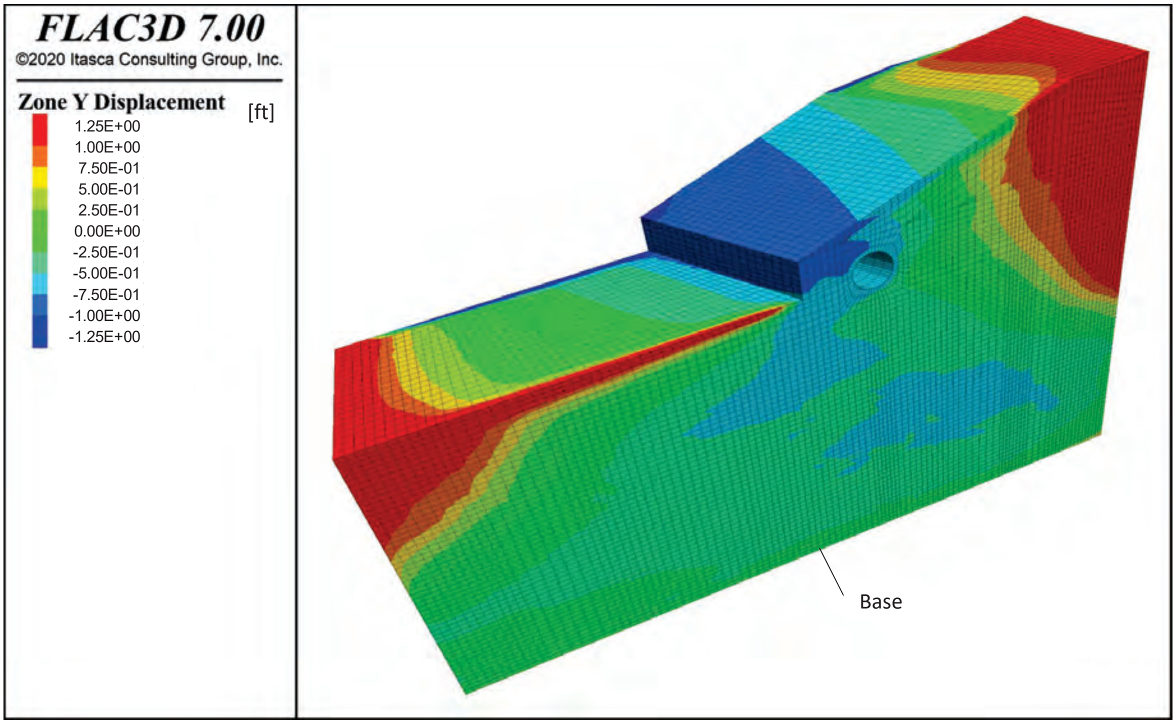


RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 40 SEC. - EARTHQUAKE SCENARIO B - SOUTH REACH) AND TENSILE FORCE IN THE TIE-BACK SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE F-19

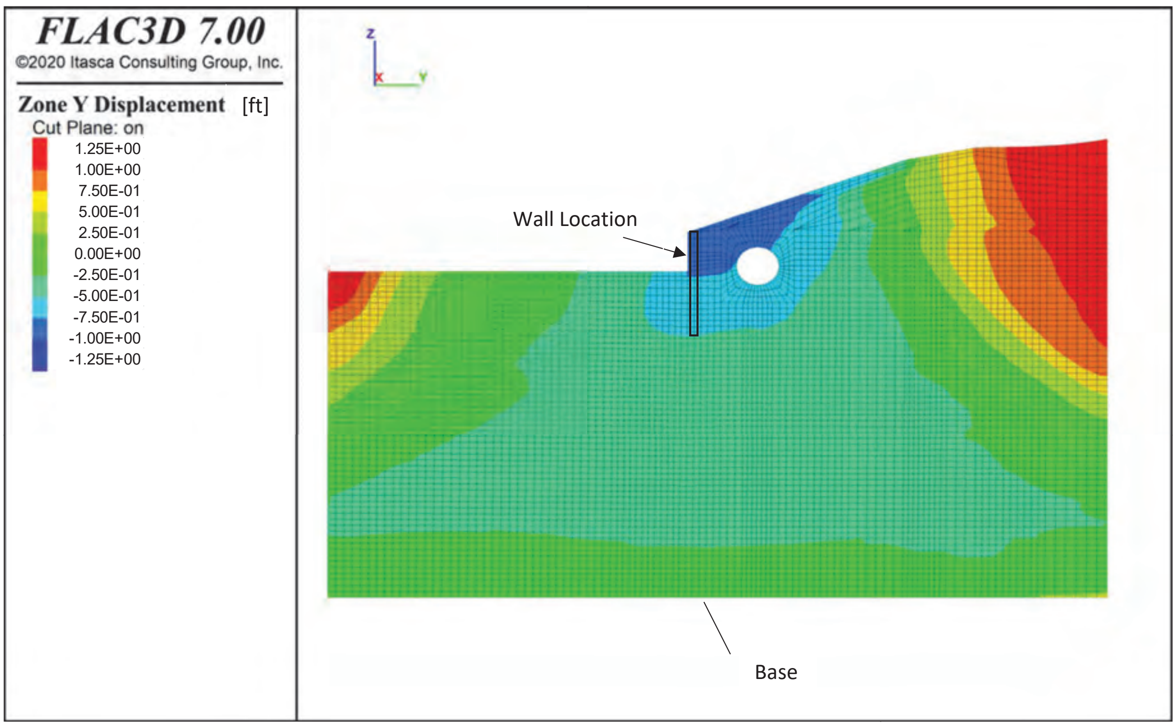


Note 1: Depicted displacements are only due to seismic simulation for the subject SCENARIO and are in addition to the gravitational loading.
Note 2: dY and dZ are relative displacement (relative to bottom of the pile) in Y and Z (elevation) directions, respectively.
Note 3: Displacements are exaggerated. Exaggeration factor in both Z and Y direction is 2.
Note 4: Positive direction in Y direction is toward the slope (away from the ocean).

TUNNEL AND PILE DISPLACEMENT **(AT t = 40.0 SEC. - EARTHQUAKE SCENARIO B SOUTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE F-20

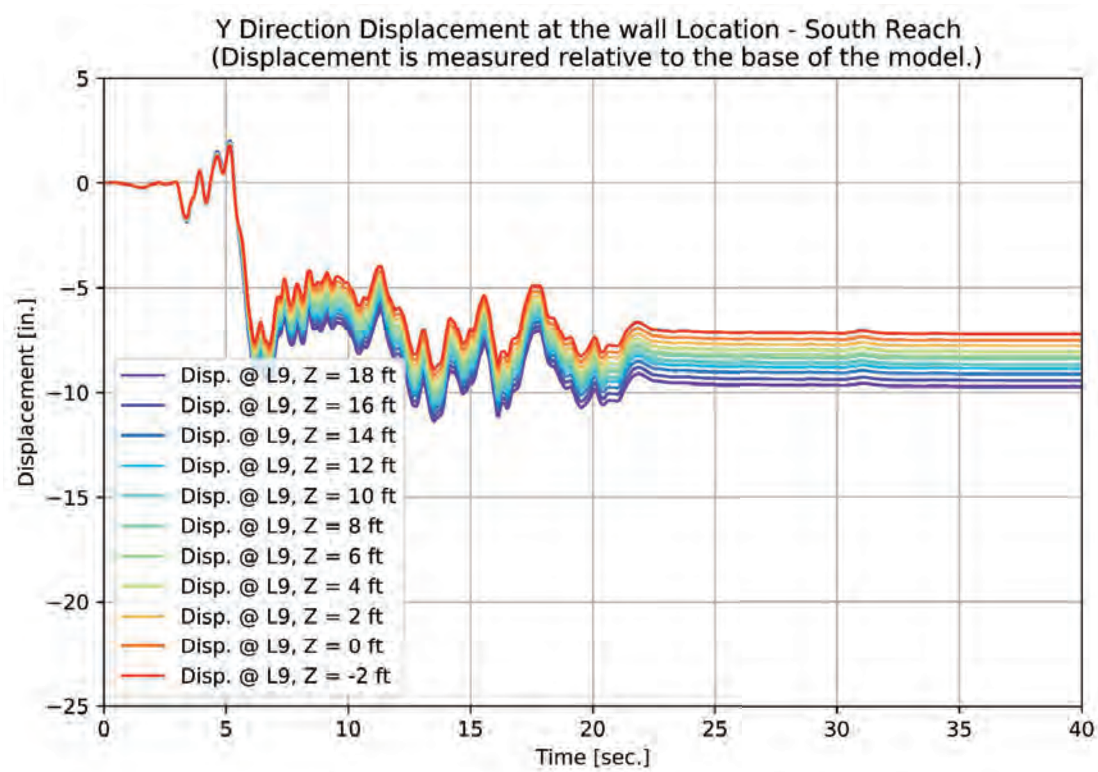


RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO C - SOUTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE F-21



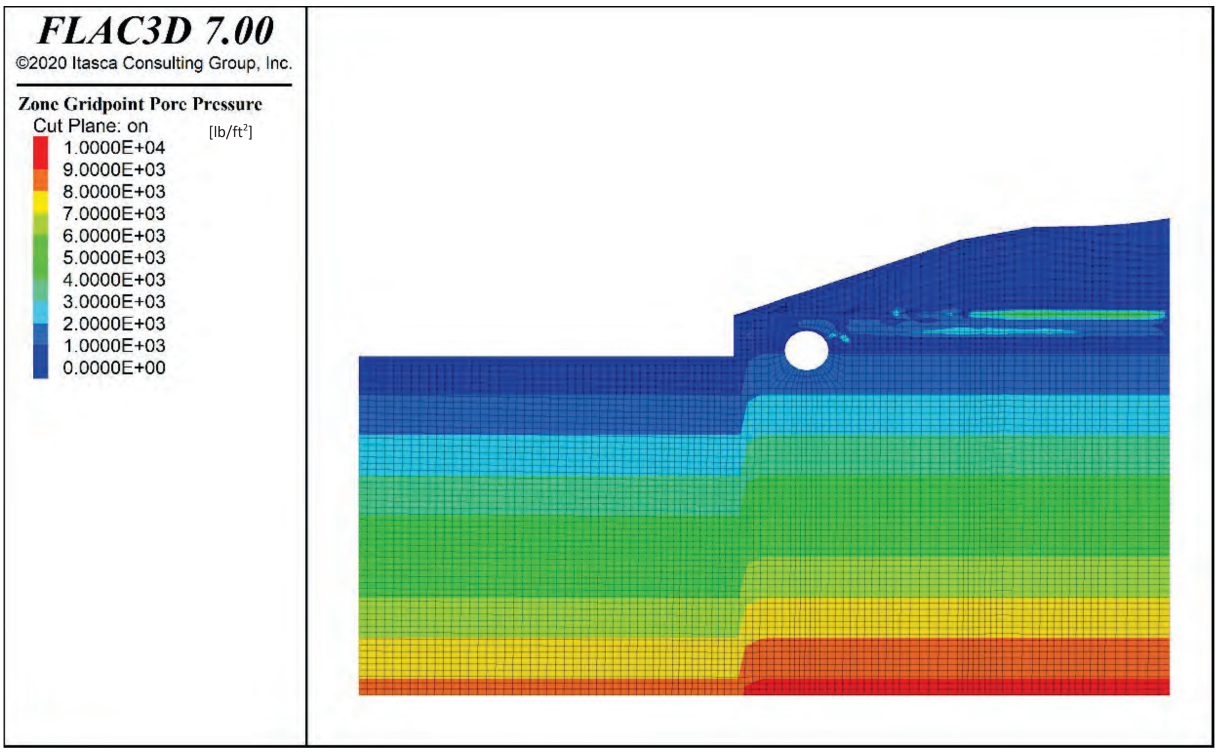
RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO C - SOUTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIASAN FRANCISCO, CALIFORNIA	AGS

JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE F-22

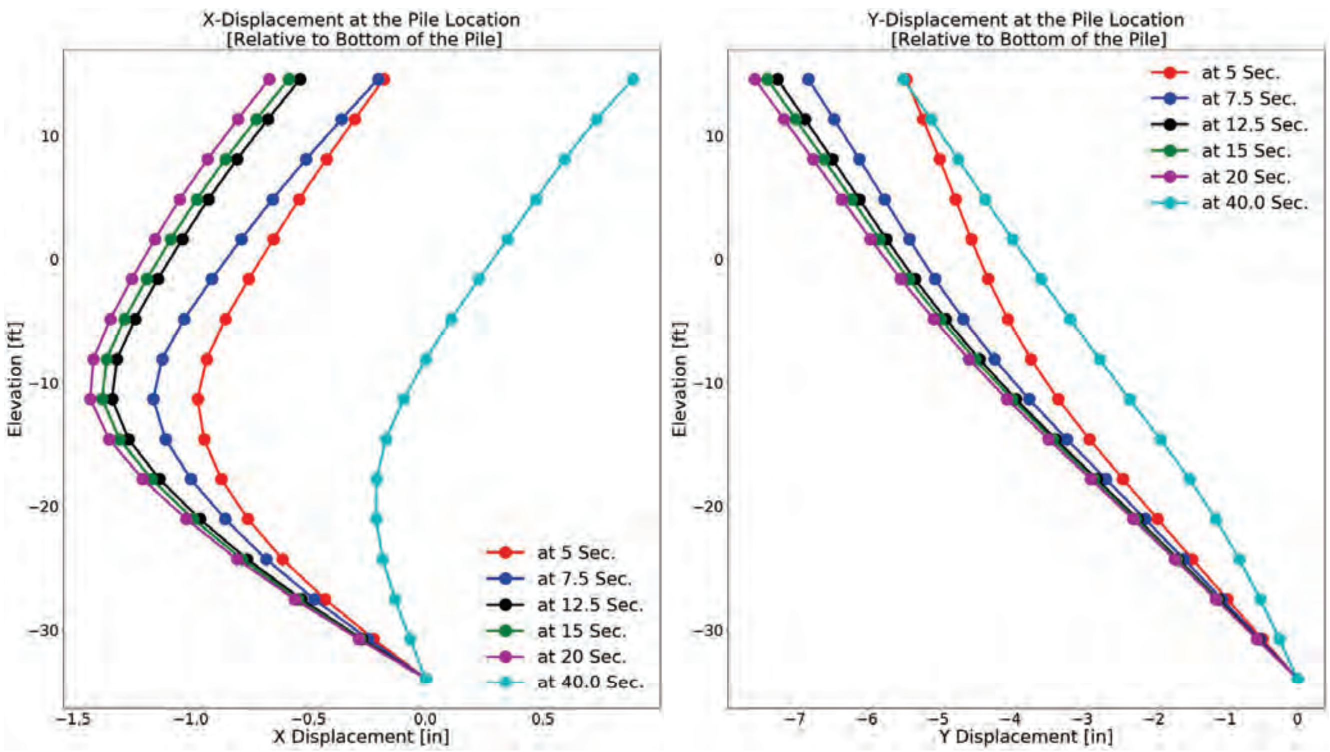


Note: Positive direction in Y direction is toward the slope (away from the ocean).

Y-DIRECTION DISPLACEMENT AT DIFFERENT ELEVATIONS– MIDDLE SECTION (EXPOSED PORTION OF THE SECANT WALL- EARTHQUAKE SCENARIO C - SOUTH REACH) SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE F-23

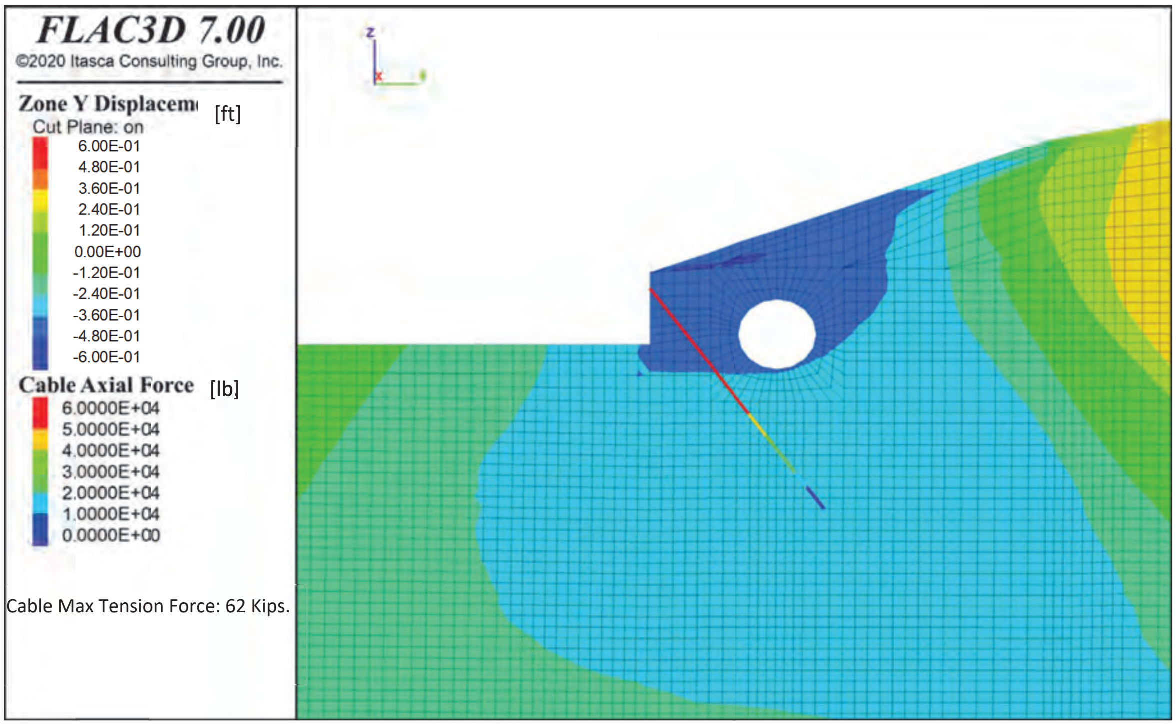


	PORE WATER PRESSURE **(EARTHQUAKE SCENARIO-C AT t= 6.0 SEC- SOUTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIA	AGS
	JOB NO. AGS-18-003 / DATE: JUNE 2021	PLATE F-24

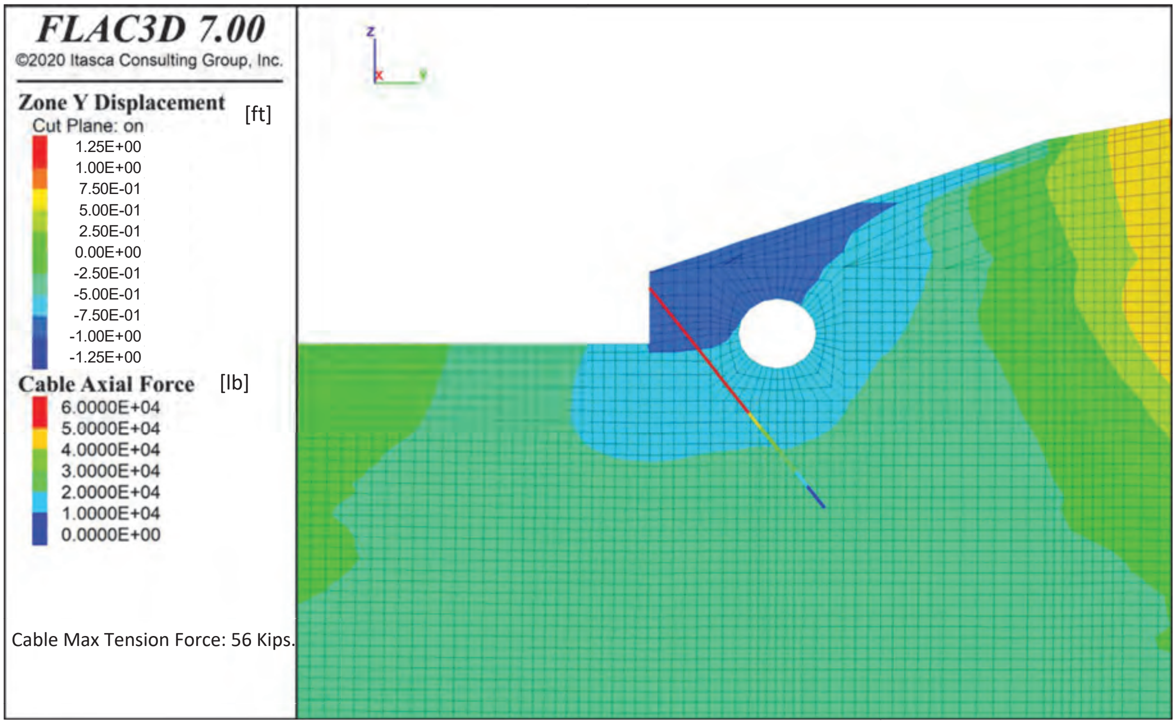


Note 1: Positive direction in Y direction is toward the slope (away from the ocean).
Note 2: Displacements are shown up to fixity depth of the pile.

DISPLACEMENT IN THE PILE OF THE WALL (MIDDLE PILE) **(SOUTH REACH - EARTHQUAKE SCENARIO C - SOUTH REACH)** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION SAN FRANCISCO, CALIFORNIA SAN FRANCISCO, CALIFORNIASAN		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE F-25

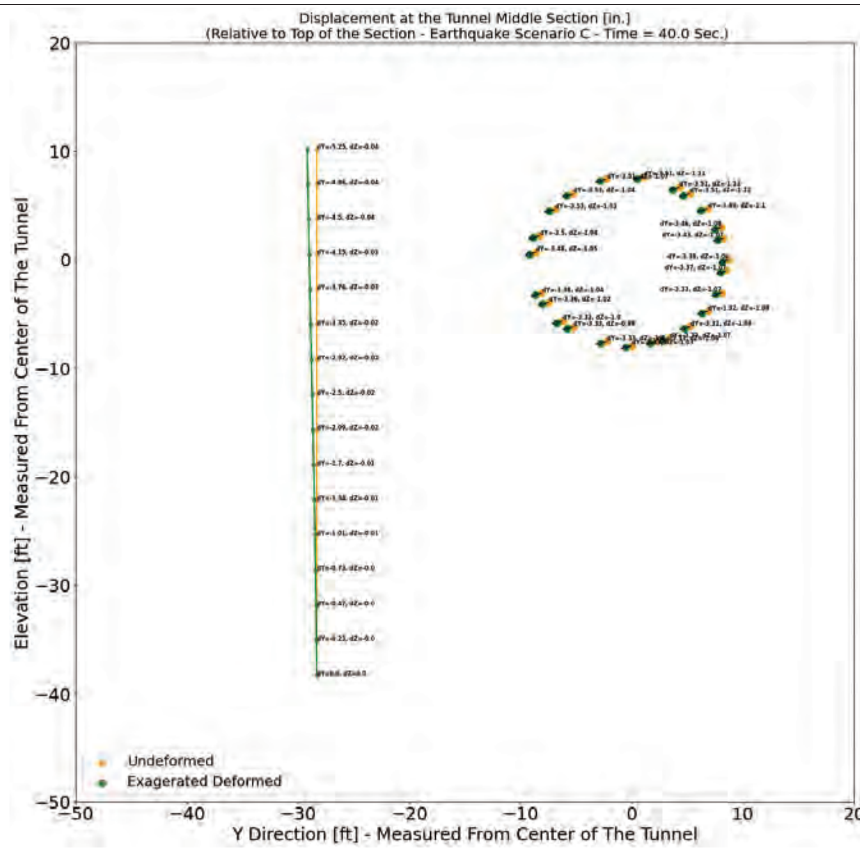


RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION (AT t = 6.0 SEC. - EARTHQUAKE SCENARIO C - SOUTH REACH) AND TENSILE FORCE IN THE TIE-BACK SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA		AGS
JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE F-26



RELATIVE (TO THE BASE) DISPLACEMENT in Y-DIRECTION **(AT t = 40 SEC. - EARTHQUAKE SCENARIO C - SOUTH REACH)** **AND TENSILE FORCE IN THE TIE-BACK** SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA	AGS

JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE F-27



Note 1: Depicted displacements are only due to seismic simulation for the subject SCENARIO and are in addition to the gravitational loading.
Note 2: dY and dZ are relative displacement (relative to bottom of the pile) in Y and Z (elevation) directions, respectively.
Note 3: Displacements are exaggerated. Exaggeration factor in both Z and Y direction is 2.
Note 4: Positive direction in Y direction is toward the slope (away from the ocean).

TUNNEL AND PILE DISPLACEMENT
(AT t = 40.0 SEC. - EARTHQUAKE SCENARIO C SOUTH REACH)
SOUTH OCEAN BEACH COASTAL EROSION AND WASTEWATER INFRASTRUCTURE PROTECTION, SAN FRANCISCO, CALIFORNIA

AGS

JOB NO. AGS-18-003	DATE: JUNE 2021	PLATE F-28

List of Required Construction Best Management Practices (BMPs)

SFPUC Ocean Beach Armoring

Standard Construction Measure (SCM) 7. Biological Resources: All project sites and the immediate surrounding area will be screened to determine whether biological resources may be affected by construction. A qualified biologist will also carry out a survey of the project site, as appropriate, to note the general resources and identify whether habitat for special-status species and/or migratory birds, are present. In the event further investigation is necessary, the SFPUC will comply with all local, State, and federal requirements for surveys, analysis, and protection of biological resources (e.g., Migratory Bird Treaty Act, federal and State Endangered Species Acts, etc.). If necessary, measures will be implemented to protect biological resources, such as installing wildlife exclusion fencing, establishing work buffer zones, installing bird deterrents, monitoring by a qualified biologist, and other such measures. If tree removal is required, the SFPUC would comply with any applicable tree protection ordinance.

Mitigation Measure M-BI-2a: Nesting Bank Swallow Protection Measures. This measure applies to construction activities and small sand placements. Nesting bank swallows, their eggs and their nests, and their young shall be protected during construction and during sand placement events through the implementation of the following measures:

a) If construction or beach nourishment activities within 650 feet of the bluffs used by the Fort Funston bank swallow colony are conducted during bank swallow nesting season (nesting is from April 1 to August 1), a qualified wildlife biologist shall conduct preconstruction surveys for nesting bank swallow within seven days prior to the start of construction, beach nourishment activities, and prior to reinitiating construction at this location after any construction breaks of 14 days or more.
b) If active bank swallow nest sites are located during the preconstruction nesting surveys, a 650-foot no-disturbance buffer shall be established around the burrow nest site and all project work shall halt within the buffer until a qualified biologist determines the nest is no longer in use.

Mitigation Measure M-BI-2b: Worker Environmental Awareness Program Training. This measure applies to construction activities and small sand placements. A project-specific Worker Environmental Awareness Program training shall be developed by a qualified biologist for the project and attended by all construction personnel prior to beginning on site work. As part of the training, brochures may be given to provide reference material to contractors. The training may be provided by the qualified biologist or by designated SFPUC staff trained by the biologist to provide this training, using the materials developed by the qualified biologist, and may be administered via a video-recorded training produced specifically for the project by a qualified biologist. A more in-depth environmental training may be developed and provided for contractor supervisors in leadership roles. The environmental training shall generally include but not be limited to education about the following:

a) Applicable state and federal laws, environmental regulations, project permit conditions, and penalties for non-compliance;
b) Special-status species with potential to occur on or in the vicinity of the project sites, avoidance measures, and a protocol for encountering such species including a communication chain;
c) Preconstruction surveys and biological monitoring requirements associated with each phase of work and at each project site as biological resources and protection measures will vary depending on project component location and the corresponding land managers (see f, below);
d) Known sensitive resource areas in the project vicinity that are to be avoided and/or protected, as well as approved project work areas, access roads, and staging areas;
e) Best management practices and their location at various project sites for erosion control and species exclusion, in addition to general housekeeping requirements; and
f) Specific requirements sanctioned by the National Park Service (NPS) that the project must comply with while working on NPS-managed lands.

Mitigation Measure M-BI-2c: Bank Swallow Educational Signage and Protective Fencing. During the construction period and prior to project completion, the SFPUC, with the oversight of the planning department, shall implement the following:

a. Develop and produce one, permanent educational kiosk or signage to be installed in the Skyline coastal parking lot or along the multi-use trail. Educational content, sign design and structure shall be coordinated with the San Francisco Recreation and Parks Department and the National Park Service (NPS).

b. Develop and produce semi-permanent educational signs that shall be installed on NPS property along bluff top access points at Fort Funston near the bank swallow nesting locations to alert the public of the sensitive nesting area. The SFPUC and NPS shall enter into an agreement for the one-time development and production of the semi-permanent signs that the NPS shall install at its discretion as long as the bank swallow are listed as special-status and nesting within NPS-managed lands.

c. Install semi-permanent fencing at a setback from the bluff edge above suitable nesting habitat to restrict public access above sensitive nesting areas. The SFPUC and NPS shall enter into an agreement for the one-time development and production of the semi-permanent fencing that the NPS shall design and install at its discretion as long as the bank swallow are listed as special-status and nesting within NPS-managed lands.

Mitigation Measure M-BI-9: Avoidance and Minimization Measures for Special-Status Bats and Maternity Roosts. A qualified biologist experienced in the identification of special-status bats shall conduct a preconstruction survey for special-status bat species habitat in advance of any tree trimming or removal to identify signs of potential bat habitat, including maternity colonies and any active roost sites. Identified bat maternity colonies shall be avoided, if possible. Should potential maternity colonies or active bat roosts be found in trees but cannot be avoided, SFPUC shall ensure the following measures are implemented:

a. Trim trees or install bat exclusion devices when bats are active, approximately between the periods of March 1 to April 15 and August 15 to October 15; outside of the bat maternity roosting season (approximately April 15 to August 15) if a maternity roost is present, and outside the months of winter torpor (approximately October 15 to February 28, or as determined by a qualified biologist experienced in the identification of special-status bats).

b. If tree trimming is not feasible during the periods when bats are active, and bat roosts being used for maternity or hibernation purposes are found on or in the immediate vicinity of the tree trimming, a qualified biologist shall delineate a no-disturbance buffer around these roost sites until they are no longer in use as maternity or hibernation roosts or the young are capable of flight.

c. Based on the professional opinion of a qualified biologist, buffer distances may be adjusted around roosts depending on the level of surrounding ambient activity (e.g., if the subject tree is adjacent to a busy road) or if an obstruction, such as a large sand dune, is within the line-of-sight between the roost and construction.

d. A biologist experienced in the identification of special-status bats shall be present during tree trimming and removal if bat roosts are present. Project activities shall disturb trees with roosts only when no rain is occurring or is not forecast to occur for three days and when daytime temperatures are at least 50 degrees Fahrenheit.

e. Under the supervision of the qualified biologist, trim trees containing or suspected to contain roost sites over two days. On the first day, branches and limbs not containing cavities or fissures in which bats could roost shall be cut using chainsaws. The following day, branches or limbs containing roost sites shall be trimmed with chainsaws, under the supervision of the biologist.

Land Valuation

Address	Lot Size (sq. ft.)	Sale Price	Sale Date	Price/Sq. Ft.
2630-2632 Great Highway	2,688	$3,400,000	6/30/2023	$1,264.88
2646-2648 Great Highway	2,674	$1,750,000	5/2/2023	$654.45
2554 Great Highway	2,988	$2,307,000	5/18/2022	$772.09
2542 Great Highway	3,001	$926,000	2/28/2022	$308.56
2538 Great Highway	2,996	$2,540,000	4/9/2021	$847.80

Average Cost/Square Foot: $769.56

Source: Zillow, March 2024