Earthquake Vulnerability Study

for the

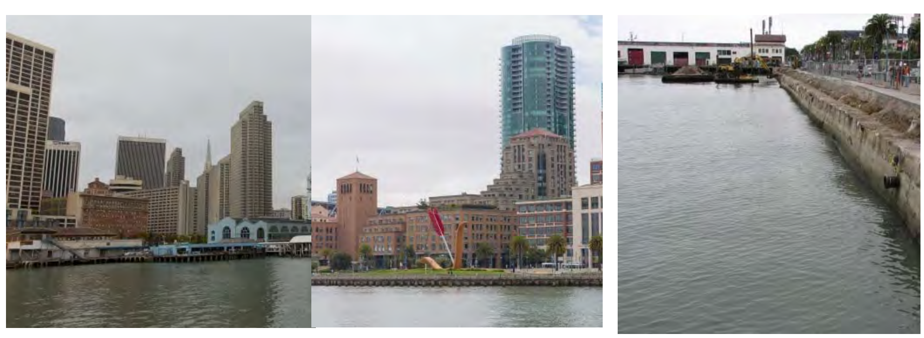
Seawall Vulnerability Study

of the

Northern Seawall
San Francisco, California

Phase 2 Report

July 2016



Prepared for:
Port of San Francisco

Prepared by:
GHD-GTC Joint Venture

Earthquake Vulnerability Study for the Seawall Vulnerability Study
PHASE 2 REPORT

Northern Seawall,
San Francisco, CA

July 2016

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List of Preparers

A consultant team headed by GHD-GTC Joint Venture prepared this report under the guidance of the Port of San Francisco, Engineering Division. A list of the consultant team personnel that worked on the Earthquake Vulnerability Study of the Northern Waterfront Seawall is provided below.

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1. Executive Summary

1.1. Project Description and Scope of Work

The Port of San Francisco ("Port") is a self-supporting, municipal enterprise agency overseeing 7-1/2 miles of waterfront property along the San Francisco Bay. The Port has initiated a program to identify and upgrade portions of the waterfront vulnerable to earthquakes, flooding, and climate change.

As such, the Port authorized an earthquake vulnerability study of the Northern Waterfront Seawall which extends approximately 3 miles from Fisherman's Wharf to Pier 46. Components of the study included: assessment of available information and condition, engineering analysis to determine likely damage to the seawall and infrastructure within an inferred "zone of influence", economic impacts resulting from multiple earthquake scenarios, development of conceptual level retrofits/costs, and recommendations for implementation of improvements and/or further study.

The overall study consisted of three phases: 1) research, data collection and synthesis, 2) earthquake vulnerability study, and 3) recommendations for mitigation of earthquake hazards. The Phase 1 report presented our findings, conclusions and recommendations regarding the research, data collection and synthesis phase of this study.

For Phase 2, the GHD/GTC Joint Venture (JV) scope of work focused on the evaluation of earthquake performance of the seawall, bulkhead wall/wharf, and other infrastructure within the estimated zone of influence of the seawall section. Flooding vulnerability was assessed for intact and damaged seawall conditions associated with seismic events. The assessment considered existing and higher future sea levels. This Phase 2 report presents the JV's findings and conclusions concerning the vulnerability of the seawall and related structures.

1.2. Geotechnical

The GHD/GTC team performed an extensive search of geotechnical data within the project study area, and compiled and catalogued the data in a GIS database. The project study area of the Earthquake Vulnerability Study of the Northern Waterfront Seawall was defined as the areal extent of land, piers and building structures, and other important infrastructure including the Embarcadero Promenade and Roadway, the Muni light rail line, BART facilities, and major utilities including SFPUC pipelines, PG&E, and telecommunications lines that may be impacted by the movement of the seawall in the event of an earthquake (i.e. zone of influence). A conservative boundary of the seawall zone of influence was defined during Phase 1 in order to define the project limits for the purpose of compiling relevant data including geotechnical reports and boring logs, construction drawings of the potentially-affected structures, condition surveys, rapid structural evaluations, utility information, and economic data. Our conservative zone of influence, or project study area, was defined as the study area limits in the 1992 liquefaction study report (HLA et al., 1992) but also further limited to within 1,200 feet of the seawall structures. This project study area is shown graphically on Figure 1-1**– *Seawall Project Study Area Map***. The exploration locations mapped in the GIS database are represented in Figure 1-2**– *Historical Exploration Location Map***.

From this data, the GHD/GTC team prepared geologic cross sections at selected seawall locations, performed site-specific ground motion studies along the 3 miles of waterfront on The Embarcadero, and evaluated the permanent ground deformations (magnitude and areal extent) that may affect the waterfront. Based on the seismic microzonation from the site-specific ground motion study, the waterfront can be represented by three typical, yet generalized, subsurface profiles dependent on the thickness of young bay mud and the depth to bedrock. Each of the seawall sections were evaluated and a zonation assigned (A1, A2, B1, B2, C1, C2) based on subsurface conditions. Seismically-induced horizontal ground deformations, or lateral spread, were estimated based on evaluating the yield acceleration of the seawall using General Limit Equilibrium (GLE) procedures implemented in the software Slope-W, and using practice-oriented, Newmark-type sliding block analyses. The ground motions and permanent ground deformations were evaluated as a function of the intensity of shaking as represented by the Average Return Period (ARP) of the earthquake event.

The variation of horizontal ground deformation and ground surface settlement associated with lateral movement of the seawall were estimated based on case histories. The majority of the data used was from horizontal deformations of the port facilities during the 1995 Kobe Earthquake, which is considered a reasonable analog (with adjustments) for estimating the anticipated behavior of the Northern Waterfront Seawall. This data was also supplemented with observed deformations for several sites associated with the 1989 Loma Prieta Earthquake and the 1993 Guam Earthquake. It is interesting and important to note that the trends of displacement from these case histories indicate a more restricted zone of influence than would be predicted from free-field, lateral spread procedures such as developed by Youd et al. (2002) and Zhang et al. (2004). We also estimated the ground surface settlement associated with post-liquefaction, re-consolidation of sandy fill within the man-made land behind the seawall. The settlements associated with post-liquefaction, re-consolidation of sandy fill will occur regardless of lateral movement of the seawall. Total vertical settlement is a sum of the ground surface settlement associated with lateral movement of the seawall and post-liquefaction, re-consolidation settlement. The GHD/GTC team prepared maps showing the zone of influence and magnitude of permanent horizontal ground deformation, vertical deformation as a result of seawall movement, and total vertical settlement for four seismic hazard levels – the median estimate of a M8.0 San Andreas seismic event, 475-year return period, 975-year return period, and Maximum Considered Earthquake.

The scope of this assessment was limited to a review of existing geotechnical information, much of which was completed by others. No additional geotechnical investigations were performed as part of this project. Therefore, all of the analyses are the products of desktop studies using existing data and reflect uncertainties inherent in this type of study. For several of the selected seawall sections, the base of geologic and geotechnical data is limited, necessitating the use of local trends in stratigraphy and geotechnical soil properties.

The scope of the seismic and geotechnical analyses was tailored with input from the project team, in consultation with the Port of San Francisco. The level of analytical rigor was commensurate with the primary goals of assessing the seismic performance of the seawall at a “screening-level” of evaluation and in support of subsequent cost-benefit relationships for implementation of mitigation strategies at the selected seawall sections. As such, approximations and estimates are inherent in an “advanced screening” study of this type. The resulting seismically-induced ground deformations are presented as “index” values that reflect necessary approximations and assumptions. While regionally accepted methods and standards of practice were used, in many cases the level of geotechnical site investigation was not adequate at a specific location to prepare any more than an approximate estimate of the index ground deformation. The primary goal of this preliminary level of seismic analysis is to contribute to the subsequent structural performance assessments and overall seismic vulnerability assessment leading to the identification of key Port structures that would benefit from further, and more refined, site-specific evaluations. For these critical assets, it is anticipated that additional site investigations and engineering analyses would be conducted in support of potential mitigation and retrofit strategies, and more detailed cost-benefit analyses of individual facilities. These supplementary efforts are recommended as possible goals of a subsequent phase of work on specific assets, the scope of which would be guided using the results of this investigation.

1.3. Seismic Hazard Levels

The GHD/GTC team and the Port of San Francisco discussed quite extensively the selection of seismic hazard levels for the Earthquake Vulnerability Study of the Northern Waterfront Seawall. The selection weighed several factors including; (i) the historical occurrence of earthquakes (e.g. 1989 M 6.9 Loma Prieta Earthquake, 1906 M 7.9 San Francisco Earthquake), (ii) code-mandated requirements (e.g. Risk-Adjusted Maximum Considered Earthquake [MCE_R] and Design Earthquake [DE] per ASCE 7-10), (iii) Uniform Hazard Response Spectra based on probabilistic seismic hazard assessments (e.g. 5% in 50 year return period seismic event), and (iv) scenario earthquakes (e.g. M8.0 San Andreas) because of their use in similar recent seismic hazard studies. Each of these approaches provides valid ground motions for seismic performance assessment of existing waterfront structures. In the end, a combination of historical earthquakes, scenario earthquakes, probabilistic seismic events and code-based response spectra were used to evaluate certain aspects of the project which are further discussed below. ***Table 1-1*** lists the selected scenario earthquakes and uniform hazard levels with the estimated peak ground accelerations (PGA) at the ground surface that were used in the vulnerability study.

Table 1-1: Seismic Hazard Levels Used in Earthquake Vulnerability Study

Seismic Event	Estimated PGA at Ground Surface (g)	Average Return Period (years)	Probability of Exceedance in 50 Years [1] (%)
1906 San Francisco Earthquake	0.35 – 0.40	200 - 225	20 - 22
1989 Loma Prieta Earthquake	0.14 – 0.20	40 - 50	63 - 71
M8.0 San Andreas (median)	0.35 – 0.40	200 - 225	20 - 22
MCE_R	0.42 – 0.53	1500	3.3
DE	0.35 – 0.46	350 - 475	10 - 13
72-Year Return Period	0.29 – 0.34	72	50
475-Year Return Period	0.39 – 0.46	475	10
975-Year Return Period	0.41 – 0.51	975	5
2475-Year Return Period	0.42 – 0.58	2475	2

Notes:

1. Based on time-independent Poisson model.

The seawall performance was primarily assessed in terms of the permanent horizontal ground deformations (PGDs) estimated on the basis of standard of practice, “advanced screening” level of engineering evaluation for the ground motions indicated in ***Table 1-1***. As such, these estimates should be considered “Index PGDs” that are considered representative for the overall seismic performance assessment of the waterfront, but which should be refined for site-specific analysis leading to design of geotechnical and/or structural mitigation options. The engineering analyses were calibrated using observed performance during the 1906 and 1989 earthquakes, and best estimates for the mean-level ground motions during both events. Because no ground motion records are available from 1906, the estimated spectral accelerations are based on a M8.0 event on the San Andreas Fault located approximately 13 to 14 km from the site. The five attenuation relationships developed in the 2014

NGA-West2 project were used to establish spectral accelerations at bedrock level with the following weighting factors, per USGS NSHMP (2014): Abrahamson et al. 0.22, Boore et al. 0.22, Campbell and Bozorgnia 0.22, Chiou and Youngs 0.22, and Idriss 0.12. Site response analyses for generalized waterfront soil profiles were performed to evaluate the spectral accelerations at the ground surface. For the 1989 Loma Prieta earthquake, acceleration time histories recorded on rock at local sites were used for evaluation of the seawall. After calibration of the seismic performance of the seawall during these historic earthquakes, ground motions determined from regional PSHA investigations (USGS 2008, 2014) for the seismic hazard levels provided in ***Table 1-1*** were used in supplementary analyses. Again, attenuation relationships from the 2014 NGA-West2 project and site-specific site response were used in the analyses. Permanent ground deformations were evaluated at selected seismic hazard levels for subsequent structural analysis of Port assets. These seismic hazard levels were the median estimate of a M8.0 San Andreas seismic event, 475-year return period, 975-year return period, and Maximum Considered Earthquake. The structural performance of the Bulkhead Wall structures was assessed for the following seismic hazard levels: the median estimate of a M8.0 San Andreas seismic event, Design Earthquake, 475-year return period, 975-year return period, and Maximum Considered Earthquake. Structural performance of selected Bulkhead Wharf structures were assessed for: 72-year return period, the median estimate of a M8.0 San Andreas seismic event, and 975-year return period. ***Table 1-2*** summarizes the seismic hazard levels that were used for each aspect of the seawall vulnerability study.

Table 1-2: Matrix of Seismic Hazard Levels Used in Various Analyses

	Permanent Ground Deformation (PGD) Analysis	PGD Maps	Bulkhead Wall Structural Analysis	Analysis of Bulkhead Wharf Structures
1906 San Francisco Earthquake	✓			
1989 Loma Prieta Earthquake	✓			
M8.0 San Andreas (median) [1]	✓	✓	✓	✓
MCE_R		✓ [2]	✓	
DE			✓	
72-Year Return Period	✓			✓
475-Year Return Period	✓	✓	✓	
975-Year Return Period	✓	✓	✓	✓
2475-Year Return Period	✓			

Notes:

1. Equivalent to 1906 San Francisco Earthquake seismic event for this study.
2. Estimated to be a 1500-year return period seismic event for PGD estimation.

1.4. Structural Assessment Results

General

Bulkhead Wall and Bulkhead Wharf structures were assessed for earthquake vulnerability. Bulkhead and retaining wall structures were also assessed for sliding and overturning stability. This assessment was performed for static only and static plus five different levels of seismic loading. A structural criticality rating was assigned to each structure type as a function of demand to capacity ratio (DCR) and/or factor of safety (FOS). A positive rating indicates a structural deficiency..

Five bulkhead wharf sections were assessed for structural capacity under design basis seismic inertial and soil lateral sliding loads.

Bulkhead Walls

All Bulkhead Wall types were found to be acceptable under static load, however many were found to be inadequate under varying levels of seismic load. Forty (40) distinct bulkhead wall types were assessed. The number of deficient bulkhead walls (DCR > 1.0) as a function of design loading is as follows:

- Static load only – 1 (maximum DCR = 1.04)
- Static plus M8.0 San Andreas earthquake loading - 13 (maximum DCR = 2.24)
- Static plus DE earthquake loading - 13 (maximum DCR = 2.34)
- Static plus 475 year earthquake loading - 15 (maximum DCR = 2.66)
- Static plus 975 year earthquake loading - 17 (maximum DCR = 2.88)
- Static plus MCE earthquake loading - 20 (maximum DCR = 3.19)

For the most severe seismic loading considered in this study, about half the total length of bulkhead assessed in this study has some type of structural deficiency. The relative criticality is shown graphically on Figure 4-47 through Figure 4-52 and by seawall location on Figure 4-53 and Figure 4-54.

Various types of data needed for the structural analysis were collected during Phase 1 of this study. The types of data collected for each seawall section were divided by seawall section component, namely, rock dike, bulkhead wall, bulkhead/marginal wharf and finger pier. The rock dike represents a common component that has geotechnical and structural implications. The bulkhead structures were assessed for their stability and design basis load capacity. The marginal wharf structures were assessed to ascertain their contribution to design basis load resistance of the bulkhead wall structures.

Figure 4-1 summarizes the data obtained for these various seawall sections and structural types. The summary is sorted by seawall section and structure type, with data listed for the rock dike, bulkhead wall, marginal wharfs, and finger piers. The individual blocks are color-coded to represent the data item status (green for data in-hand through red for data that is unavailable). Where a data item is in-hand, the data value is indicated in the block.

Where data are not available, data was assumed based on seawall sections of similar construction period and design, or by other criteria appropriate for the particular structural data item under consideration. These assumptions are reflected on Figure 4-1.

<u>Bulkhead Wharfs</u>

Analyses of five bulkhead wharfs and extrapolation of analyses results to other bulkhead wharfs indicate the following:

1. Timber piles are insufficient for seismic inertial load.
2. Bulkhead wharf beam connections to seawalls should be inspected and assessed for structural adequacy. Deficient connections should be retrofitted to accommodate the expected seismic loads and/or relative displacements.
3. All bulkhead wharfs are susceptible to damage due to soil lateral deformation and sliding. The percentage of wharf damage that may be expected due to soil lateral deformation is a function of the amount of pile skin friction available (i.e., rock dike material) to a specific pile above the location of pile failure. Piles located nearer the rock dike toe or outside of the rock dike location are likely to have insufficient vertical load capacity.

The finger piers were not part of this seismic assessment. Finger piers are generally more flexible than the bulkhead wharves. Additional seismic impacts on bulkhead wharfs should be expected due to the seismic response of finger piers where the piers are rigidly connected or separated by inadequate seismic joints.

1.5. Utilities Assessment Results

The infrastructure utility systems study consisted of compiling existing utility information within the zone of influence (Figure 1-1), identifying critical utilities and studying their vulnerability due to earthquake, settlement and flooding as defined by the project.

Through the Notice of Intent (NOI) process and the Lifeline Council, we have gathered existing utility system maps from some of the participating utility agencies.

Some of the system maps are in GIS format and contain useful GIS data. We created AutoCAD drawings for the system maps that are received in pdf format. Data are grouped and color coded for each utility system in individual exhibits.

We met with individual utility agencies to discuss and further understand and identify the critical elements of their system. The team presented the findings for the studied events to some of the utility agencies to understand the impact and vulnerability to their critical utility and structures.

1.6. Flooding Vulnerability Assessment Results

The vulnerability of the San Francisco waterfront to flooding and inundation was evaluated for different seismic scenarios that could occur for both existing and future conditions with sea level rise. The assessment utilized prior studies completed for the Port and the City and County of San Francisco, as well as recently adopted guidance for incorporating sea level rise into planning in San Francisco, to define the existing and future flood elevations, extents and pathways. Refinements to these data were performed to best represent the potential impacts of different combinations of sea level rise, storms, and intact or deformed seawall and its zone of influence. Flood vulnerability was measured on a semi-quantitative basis using criteria that was developed during the study in collaboration with the project team and the Port of San Francisco.

The approach to evaluating the flood vulnerability along the San Francisco waterfront comprised selecting flood elevations and estimating the approximate extents of flooding for the conditions of an intact seawall and a damaged seawall associated with seismic scenarios. The increase in risk over time was assessed by considering sea level rise amounts consistent with City guidance at 2050 and 2100. Still water level (SWL) elevations and wave runup heights along the study area were derived using the SFPUC (2014) and URS and AGS (2012) mapping and tabulations of values.

This flooding vulnerability assessment covers the following items:

- Key Terminology, Datums, and Extreme Values: presents a summary of the tidal elevations and extreme water levels along the San Francisco waterfront, as well as defining terminology that is used in coastal flooding and vulnerability assessments;
- Jurisdiction, Policy, and Sea Level Rise Guidance: presents a description of pertinent policies and guidance for incorporating sea level rise into planning, sea level rise projections, and defines vulnerability and risk terminology;
- Available Maps and Data Products: summarizes available coastal flood maps and data for existing and future conditions with sea level rise along the San Francisco waterfront;

- Approach to Assessing Flooding Vulnerability: describes the approach that was used to evaluate the vulnerability of the San Francisco waterfront to flooding for existing and future conditions with sea level rise.
- Flooding vulnerability maps presented for 5-Year still water level inundation and potential wave hazard zones for the years 2010, 2050 and 2100.

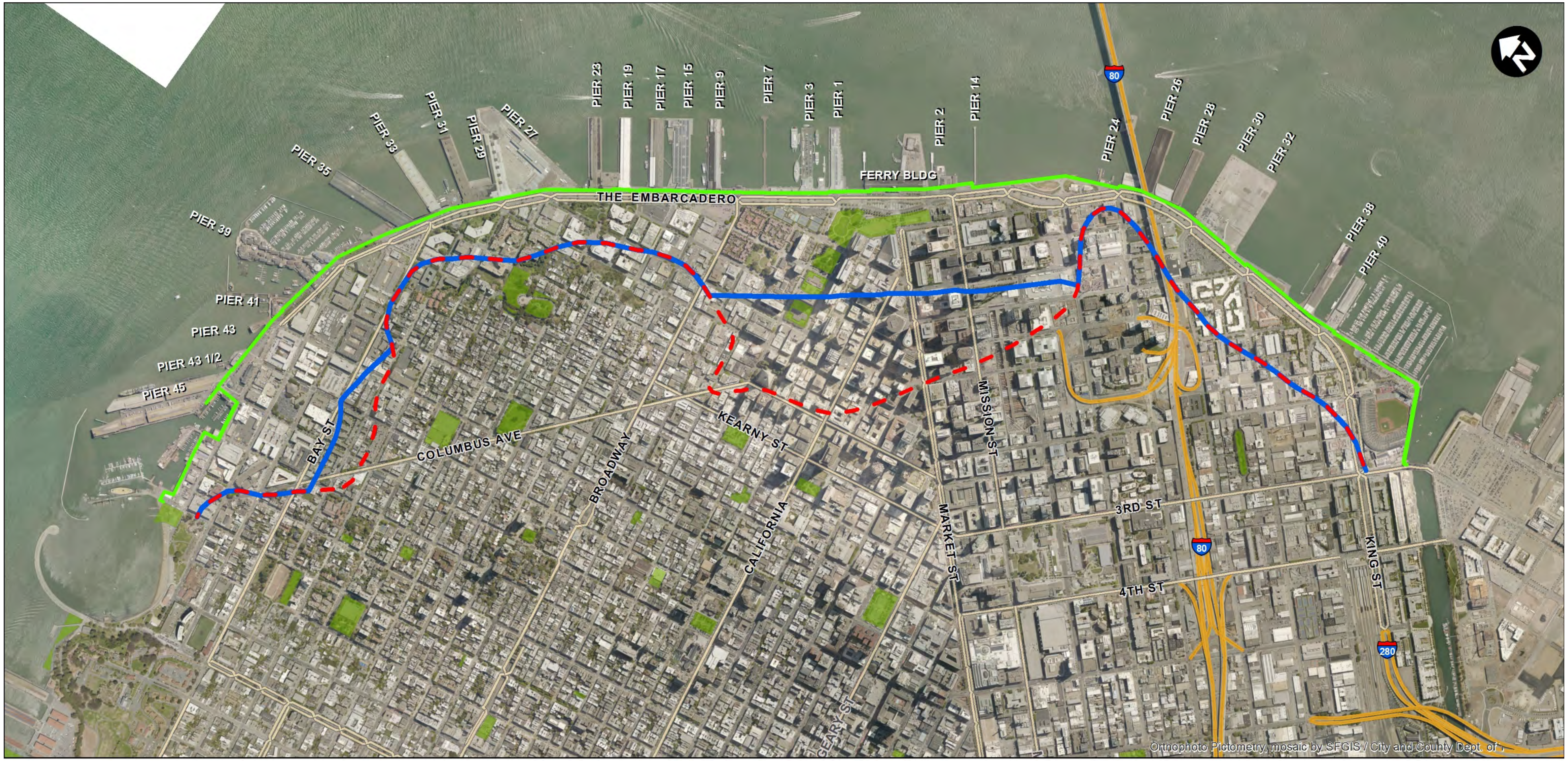
1.7. Economics Research and Data Results

The approach and methodology used in assessing the economic impacts of potential damage to the San Francisco seawall in the event of a major seismic event is presented in *Section 7* of this report. The data may be adapted for use with the HAZUS modeling framework in a subsequent phase of work.

1.8. Peer Review

At the request of the Port, COWI performed a peer review of this Phase 2 report. COWI's comments and the associated JV responses were documented and are presented in Appendix A.

All peer review comments were addressed either by explanatory response or earlier revision of this final report by the JV. The JV actions with respect to the peer review were deemed appropriate for the scope and objectives of this study.



LEGEND

- Seawall Bulkhead
- Lateral Spread Hazard Boundary (HLA et al., 1992)
- Project Study Area, within 1200 feet of the Seawall Bulkhead and within the Lateral Spread Hazard Zone

0 500 1,000 2,000 Feet

Lateral Spread Boundary Source: Harding Lawson Associates (HLA), Dames & Moore, Kennedy/Jenks/Chilton, EQE Engineering, 1992. Final Report, Liquefaction Study, North Beach, Embarcadero Waterfront, South Beach, and Upper Mission Creeek Area, San Francisco, California.

Figure 1-1: Seawall Project Study Area Map

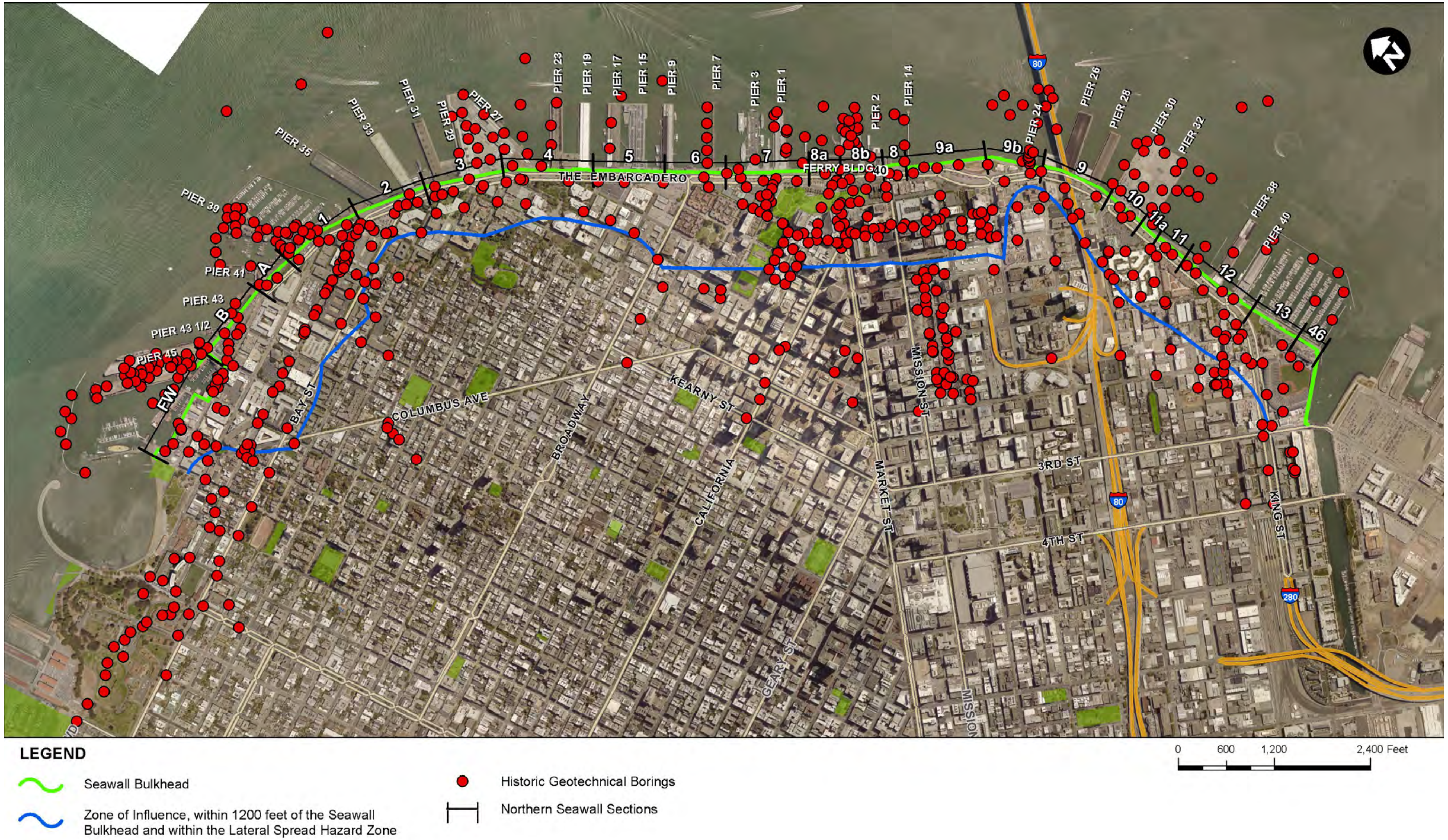


Figure 1-2: Historical Exploration Location Map

2. Introduction

2.1. Background

The Port of San Francisco (“Port”) is a self-supporting, municipal enterprise agency overseeing 7-1/2 miles of waterfront property along the San Francisco Bay. The Port has initiated a program to identify and upgrade portions of the waterfront vulnerable to earthquakes, flooding, and climate change.

As such, the Port authorized an earthquake vulnerability study of the Northern Waterfront Seawall, which extends approximately 3 miles from Fisherman’s Wharf to Pier 46. Components of the study included: assessment of available information and condition, engineering analysis to determine likely damage to the seawall and infrastructure within the zone of influence, economic impacts resulting from multiple earthquake scenarios, development of conceptual level retrofits/costs, and recommendations for implementation of improvements and/or further study.

The overall study consisted of three phases: 1) research, data collection and synthesis, 2) earthquake vulnerability study, and 3) recommendations for mitigation of earthquake hazards. Our Phase 1 report presented our findings, conclusions and recommendations regarding the research, data collection and synthesis phase of this study.

For the Phase 2 work, the GHD/GTC Joint Venture (JV) evaluated earthquake performance of the seawall, bulkhead wall/wharf, and other infrastructure within the estimated zone of influence of the seawall section. The results of the geotechnical and structural assessments served as a basis for the Phase 3 development of mitigation alternatives for earthquake hazards to the seawall sections, adjacent marginal wharf structure, and utilities. The geotechnical and structural assessment is quite specific for each seawall section delineated in this study, since the specific data was used to ascertain site-specific hazards and their potential effects on geotechnical and structural damage or failure for each seawall section. Other study disciplines, specifically utilities, flooding and economics are more global in their coverage and do not necessarily lend themselves to such site-specific consideration. The assessments for these other disciplines are also summarized in this Phase 2 report.

3. Geotechnical

3.1. Introduction

The Port of San Francisco ("Port") is conducting a comprehensive seismic vulnerability assessment of roughly three miles of waterfront development. The primary purpose of the Northern Waterfront Seawall Earthquake Vulnerability Study was to provide a screening-level evaluation of the seismic performance of the seawall and adjacent infrastructure (piers, buildings, buried utilities, and lifelines) at various seismic hazard levels for the sake of identifying key vulnerabilities, providing preliminary recommendations for possible mitigation of seismic risks, and highlighting possible needs for more focused seismic performance assessment of key assets. For this broad waterfront evaluation it should be emphasized that the study team *focused on a level of geotechnical analysis that was deemed sufficient to provide a preliminary assessment of seismic performance and to identify structures and facilities that likely warrant additional, more refined investigation* in subsequent phases of the Port's seismic risk reduction program.

The seismic vulnerability assessment required an integrated project approach that addressed the interrelated evaluation of seismic hazards, geotechnical earthquake engineering, and structural analysis and seismic performance assessment. Geotechnical Consultants, Inc. (GTC) and New Albion Geotechnical, Inc. (NA) provided seismic and geotechnical input (ground motions, ground deformations, ground treatment strategies and approximate costs) to the GHD/GTC project team members focused on addressing structural-related seismic performance issues. The results of these ground motion and ground deformation analyses were used directly by all members of the team to evaluate seismic loading on the structures at the seismic hazard levels (or Average Return Periods, ARP) of interest for the specific asset.

The Phase 1 report provided an overview of the geologic and geotechnical conditions and seismicity within the project study area. This Phase 2 report presents the findings, conclusions and recommendations regarding geologic cross sections, seismic ground motions, slope stability assessment, seismically-induced ground deformations, and additional geotechnical design parameters for structural assessment. Generally speaking, the project study area was defined as the area of man-made land offshore of the original shoreline (also coinciding with the study area limits in the 1992 liquefaction study report (HLA et al., 1992)), but was also further limited to within 1,200 feet of the seawall structures. As will be discussed in *Section 3.7*, the "Zone of Influence" of the seawall will be a smaller area of the waterfront.

3.2. Site and Subsurface Conditions

The site and subsurface conditions anticipated at each of the 23 seawall sections based on available subsurface boring information is provided in the Phase 1 report. Geologic cross sections were developed at Sections B, 1, 3, 7, 8b, 9a, 12 and 46 based on available information at the locations shown on ***Figure 3-6 – Geologic Cross Section Location Map***. The site and subsurface conditions for these eight seawall sections are repeated and expanded upon in the following sections. Contour maps of the thicknesses of artificial fill and young bay mud, and the elevations at the top of the young bay mud, bottom of young bay mud and top of bedrock are included as ***Figures 3-2 through 3-6***.

<u>Section 1 – 1000 Feet between Stockton and Kearny Streets</u>

Section 1 of the seawall is located in an area that was once offshore of North Beach and Telegraph Hill with almost the entire project study area for this section located within the former offshore area. The edge of the project study area for this section is located along the former 1800's shoreline which coincides with the bedrock ridge for Telegraph Hill. Borings indicate that young geologic units within the project study area for this section include: artificial fill, Young Bay Mud, and Upper Layered Sediments. Artificial fill was found to range from 0 to 36 feet thick and Young Bay Mud landward of the seawall ranges from 14 to 37 feet thick in the borings. Offshore borings along this section and along Pier 39 indicate Young Bay Mud with thicknesses ranging from at least 8 to 48 feet. Young Bay Mud in the vicinity of Section 1 is generally underlain by Upper Layered Sediments consisting of alternating layers of dense to very dense yellowish brown to grayish brown poorly graded to clayey sand and stiff to very stiff greenish gray to brown to yellowish brown lean to sandy clay. Franciscan Complex bedrock of reddish-brown shale and sandstone were noted at about 40 to 50 feet depth near North Point Street. The bedrock surface dips bayward with the bedrock approximately 95 feet below ground surface at the seawall. A geologic cross section through Section 1 is provided on ***Figure 3-8 – Geologic Profile Through Seawall Section 1***. The seawall rock dike profile at Section 1 is based on an interpretation of limited subsurface data from nearby borings and on typical seawall construction details as no specific rock dike profile was available for this section. The width of the seawall was assumed to be approximately 100 feet at its base with a 1 to 1 (horizontal to vertical) slope on the bayward side and approximately 4 ½ to 1 slope on the landward side. The fill soils are underlain by soft to medium stiff marine silts and clays and in turn by marine silty sands.

Section 3 – 1000 Feet between Francisco and Lombard Streets

Section 3 of the seawall is located in an area that was once offshore of Telegraph Hill with the project study area for this section located within the former offshore and historic wharf areas, and the hills adjacent to Telegraph Hill. The edge of the project study area for this section is located within and along the former 1800's shoreline which coincides with the Telegraph Hill bedrock high. The borings indicate that young geologic units within the project study area for this section include: artificial fill, Young Bay Mud, and Upper Layered Sediments. Landward of the seawall, artificial fill was found to range from 21 to 46 feet thick and Young Bay Mud approximately 20 feet thick in the borings. Young Bay Mud was found to be 29 to 69 feet thick in the offshore borings at Piers 27 and 29. Young Bay Mud in the vicinity of Section 3 was found to be underlain by Upper Layered Sediments consisting of layers of dense to very dense light brown to gray poorly graded to clayey sand and stiff to very stiff brown to gray lean to sandy clay. Gray Franciscan Complex shale, sandstone, and serpentinite were noted in the onshore borings at depths of 75 to 104 feet. The bedrock surface dips bayward with the bedrock approximately 130 feet below ground surface at the seawall. A geologic cross section through Section 3 is provided on ***Figure 3-9 – Geologic Profile Through Seawall Section 3***.

Section 7 – 980 Feet between Pacific and Clay Streets

Section 7 of the seawall is located in an area that was once offshore of Yerba Buena Cove, with the seawall and project study area for this section located within the former offshore and historic wharf areas and on made land. The edge of the project study area for this section is located within the former 1800's shoreline across a portion of the filled Yerba Buena Cove. The borings indicate that this area is underlain by young geologic units of: artificial fill, Young Bay Mud, and Upper Layered Sediments. The borings located landward of the seawall were noted to have artificial fill to depths ranging from 18 to 54.5 feet and Young Bay Mud at thicknesses ranging from 62 to 99 feet beneath the fill. Young Bay Mud was found to be 97.5 to 116 feet thick offshore and along Piers 1 and 3. Young Bay Mud in the vicinity of Section 7 was found to be underlain by Upper Layered Sediments consisting of layers of dense to very dense gray poorly graded to clayey sand and stiff to very stiff gray lean to sandy clay. Brown to gray Franciscan Complex sandstone and serpentinite were noted in the onshore borings at depths of 143 to 221 feet. The bedrock surface dips downward toward the east to a low off the shore of the Ferry Building with the bedrock approximately 210 feet below ground surface at the seawall. A geologic cross section through Section 7 is provided on ***Figure 3-10 – Geologic Profile Through Seawall Section 7***. The seawall rock dike profile at Section 7 is based on Plans for Pier 3 Sub Structure dated November 1916 which indicates the rock dike slopes downward between approximately 3 to 1 (horizontal to vertical) (“High Wall”) to 1 ½ to 1 (“Low Wall”) on the bayward side and approximately 3 to 1 on the landward side. The width of the seawall was assumed to be 149 feet (“High Wall”) to 86 feet (“Low Wall”) based on the plans indicating a seawall depth of approximately 51.5 feet. The seawall is underlain by approximately 70 feet of Young Bay Mud.

<u>Section 8b – 450 Feet between Market and Mission Streets</u>

Section 8b of the seawall is located in an area that was once offshore of Yerba Buena Cove, with the seawall and project study area for this section located within the former offshore and historic wharf areas and on made land. The edge of the project study area for this section is located within the former 1800’s shoreline across a portion of the filled Yerba Buena Cove. The borings indicate that this area is underlain by young geologic units of: artificial fill, Young Bay Mud, and Upper Layered Sediments. The borings located landward of the seawall were noted to have artificial fill to depths ranging from 20.5 to 45 feet and Young Bay Mud at thicknesses ranging from 60 to 89 feet beneath the fill. Offshore and along the Ferry Plaza/Ferry Terminal Pier, Young Bay Mud was found to be 50 to 100 feet thick, with artificial fill noted beneath the pier ranging from 20.5 to 45 feet thick. The thicker sequences of offshore artificial fill are associated with construction of the BART tunnels and ventilation structure. Young Bay Mud in the vicinity of Section 8b was found to be underlain by Upper Layered Sediments consisting of layers of dense to very dense gray to greenish gray to brown poorly graded sand to clayey sand and stiff to very stiff gray to brown to greenish gray lean to sandy clay. Franciscan Complex bedrock was noted in the onshore borings at a depths of 216.5 to 252.5 feet. The bedrock dips downward off of Telegraph Hill, located to the northwest, and downward off of Rincon Hill, located to the south, to a bedrock trough off the shore of the Ferry Building. The bedrock is approximately 230 feet below ground surface at the seawall. A geologic cross section through Section 8b is provided on ***Figure 3-11 – Geologic Profile Through Seawall Section 8b***. According to plans from the Board of State Harbor Commissioners and described in the 1992 liquefaction study (HLA et al., 1992), the design and construction of the seawall at Section 8b, along with Section 8a, is unique. The excavated trench for the seawall was filled with sand rather than quarry stone. According to the HLA report, a pile-supported concrete wall was constructed after partial filling of the trench. Riprap was placed over the sand fill at the toe of the seawall for erosion protection. A pile-supported relieving platform was then constructed from the concrete wall to the old 1867 seawall at the elevation of approximately Mean Lower Low Water. Sand fill was then placed to reach street grade. The sand fill is likely liquefiable. The seawall cross section at Section 8b is depicted on ***Figure 3-4***. The seawall is underlain by approximately 70 feet of Young Bay Mud.

<u>Section 9a – 990 Feet South of Mission to Folsom Street</u>

Section 9a of the seawall is located in an area that was once offshore of Yerba Buena Cove, with the seawall and project study area for this section located within the former offshore and historic wharf areas and on made land. The edge of the project study area for this section is located within the former 1800’s shoreline across a portion of the filled Yerba Buena Cove. The borings indicate that this area is underlain by young geologic units of: artificial fill, Young Bay Mud, and Upper Layered Sediments. Borings were noted to have artificial fill to depths ranging from 9.5 to 61 feet and Young Bay Mud at thicknesses ranging from 77 to 123 feet beneath the fill. Young Bay Mud in the vicinity of Section 9a was found to be underlain by Upper Layered Sediments consisting of layers of dense to very dense gray to grayish brown poorly graded sand to clayey sand and stiff to very stiff gray to brown lean to sandy clay. Franciscan

Complex bedrock was noted in one of the borings near the seawall at a depth of 154 feet. The bedrock surface dips offshore towards the north with the bedrock ranging from approximately 120 to 180 feet below ground surface at the seawall. A geologic cross section through Section 9a is provided on ***Figure 3-12 – Geologic Profile Through Seawall Section 9a***. The seawall rock dike profile at Section 9a is based on plans from the Board of State Harbor Commissioners. The rock dike slopes downward at 10 to 3 (horizontal to vertical) for approximately 23 feet, and then more steeply at 1 to 1 on the bayward side. On the landward side, the rock dike slope is approximately 1 to 1. The seawall rock dike bottom is approximately 50 feet below existing grade with a base width of 72 feet. The seawall is underlain by approximately 50 feet of Young Bay Mud.

Section 12 – 1167 Feet between Fremont and King Streets

Section 12 of the seawall is located in a former offshore area near to and northeast of Steamboat Point, with the seawall and project study area for this section located almost entirely within the former offshore area. The western edge of the project study area for this section is primarily located crossing through the former cove between Rincon and Steamboat Points with about 230 feet of it crossing the northern end of Steamboat Point. Both the onshore and offshore borings indicate that this area is underlain by young geologic units of artificial fill, Young Bay Mud, and Upper Layered Sediments. The borings landward of the seawall have artificial fill ranging from 15 to 39.5 feet deep below ground surface. Young Bay Mud was noted beneath the fill ranging from 1.5 to 17 feet thick. Young Bay Mud was observed in the offshore borings to be between 24.5 and 43 feet thick. The Young Bay Mud in the vicinity of Section 12 is underlain by Upper Layered Sediments consisting of layers of dense to very dense gray to yellowish brown to brown poorly graded sand to clayey sand and stiff to very stiff gray to grayish brown lean to sandy clay and dark gray silt. Franciscan Complex shale was encountered in two of the onshore borings at depths of 55.5 and 88.5 feet. The bedrock surface dips bayward with the bedrock approximately 180 feet below ground surface at the seawall. A geologic cross section through Section 12 is provided on ***Figure 3-13 – Geologic Profile Through Seawall Section 12***. The seawall rock dike profile at Section 12 is based on plans from the Board of State Harbor Commissioners dated May 1907 and recent geotechnical borings. The seawall is shown on the plans as being approximately 130 feet wide at its base with 1 to 1 slopes on both the bayward and landward sides. Recent geotechnical borings performed for Brannan Street Wharf (GTC, 2010), Pier 30/32 (Earth Mechanics, 2012) and AWSS Pump Station No. 1 Tunnel (AECOM AGS JV, 2013) along Seawall Sections 10, 11a, 11 and 12 indicate that the seawall rock dike bottom is approximately 38.5 feet below existing grade. Although the plans indicate the seawall is founded on a “hard bottom”, all six recent geotechnical borings encountered Young Bay Mud ranging from 1.5 to 8 feet thick underlying the rock dike.

Section 46 - AT&T Park – 1240 Feet between Berry Street and Third Street Bridge (China Basin Channel)

The Pier 46 Section of the seawall is located in a former offshore area east of Steamboat Point, with the seawall and project study area for this section located entirely within the former offshore area. The western edge of the project study area for this section is located crossing just offshore of the former Steamboat Point shoreline. Borings indicate that this area is underlain by young geologic units of artificial fill, Young Bay Mud, and Upper Layered Sediments. The borings landward of the seawall have artificial fill ranging from 13.5 to 38 feet deep below ground surface. Young Bay Mud was noted beneath the fill ranging from 2 to 19.5 feet thick. No artificial fill was observed in the offshore borings and Young Bay Mud was observed to be between 23.5 and 34.5 feet thick offshore. The Young Bay Mud in the vicinity of the Pier 46 Section is underlain by Upper Layered Sediments consisting of layers of dense to very dense gray to yellowish brown to olive brown poorly graded sand to clayey sand and stiff to very stiff gray to grayish brown and olive brown lean to sandy clay. Franciscan Complex shale was encountered in several of the onshore borings at depths ranging from 33 to 70.5 feet below ground surface. The bedrock surface dips bayward with the bedrock approximately 125 feet below ground surface at the seawall. A geologic cross section through Section 46 is provided on ***Figure 3-14 – Geologic Profile Through Seawall Section 46***. The seawall at Section 46 is also unique to this portion of the waterfront. The seawall section is based on Plans for Substructure – China Basin Terminal dated October 1921. As in Sections 8a and 8b, the trench for Section 46 was backfilled with sand fill. The sand fill is likely liquefiable. The sand fill bottom is approximately 46.5 feet below existing grade and the sand fill top is approximately 19.5 feet below grade. Rock fill was placed on top of the sand fill to form the upper portion of the seawall and to provide erosion protection. Plans indicate the width of the seawall is approximately 54 feet at its base with a 3 to 1 (horizontal to vertical) slope on the bayward side and 1 to 1 slope on the landward side. The seawall is shown to be founded on Upper Layered Sediments.

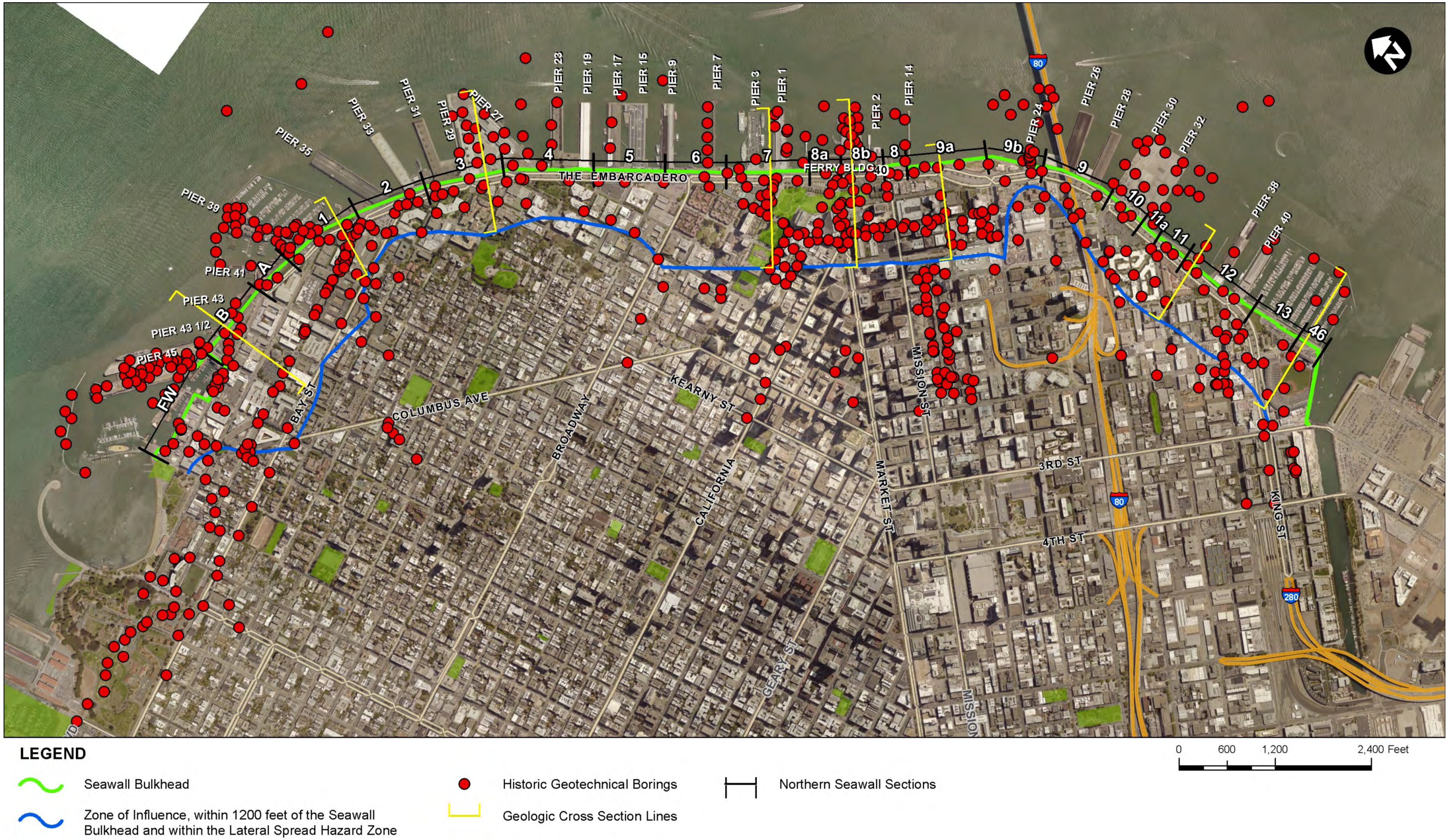


Figure 3-1: Geologic Cross Section Location Map

LEGEND

Seawall Bulkhead

Zone of Influence, within 1200 feet of the Seawall Bulkhead and within the Lateral Spread Hazard Zone

HIstoric Geotechnical Borings

5 foot Contour of Thickness of Artificial Fill

Geologic Cross Section Lines

0 600 1,200 2,400 Feet

Figure 3-2: Thickness of Artificial Fill

Figure 3-3: Thickness of Young Bay Mud

Figure 3-4: Elevation of Top of Young Bay Mud

LEGEND

Seawall Bulkhead

Zone of Influence, within 1200 feet of the Seawall Bulkhead and within the Lateral Spread Hazard Zone

HIstoric Geotechnical Borings

10 foot Contours of Elevation of Bottom of Young Bay Mud (NAVD88)

Geologic Cross Section Lines

Figure 3-5: Elevation of Bottom of Young Bay Mud

LEGEND

Seawall Bulkhead

Zone of Influence, within 1200 feet of the Seawall Bulkhead and within the Lateral Spread Hazard Zone

Historic Geotechnical Borings

20 foot Contours of Elevation of Top of Bedrock (NAVD88)

Geologic Cross Section Lines

Figure 3-6: Elevation of Top of Bedrock

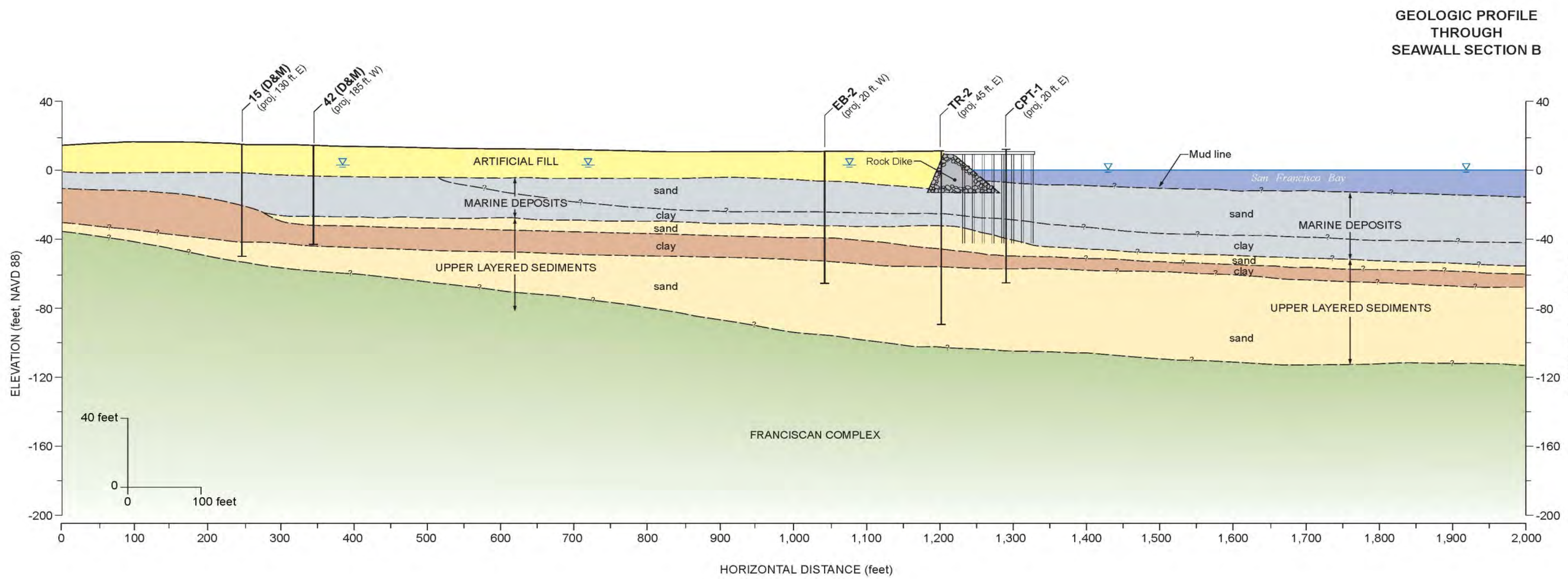


Figure 3-1: Geologic Profile Through Seawall Section B

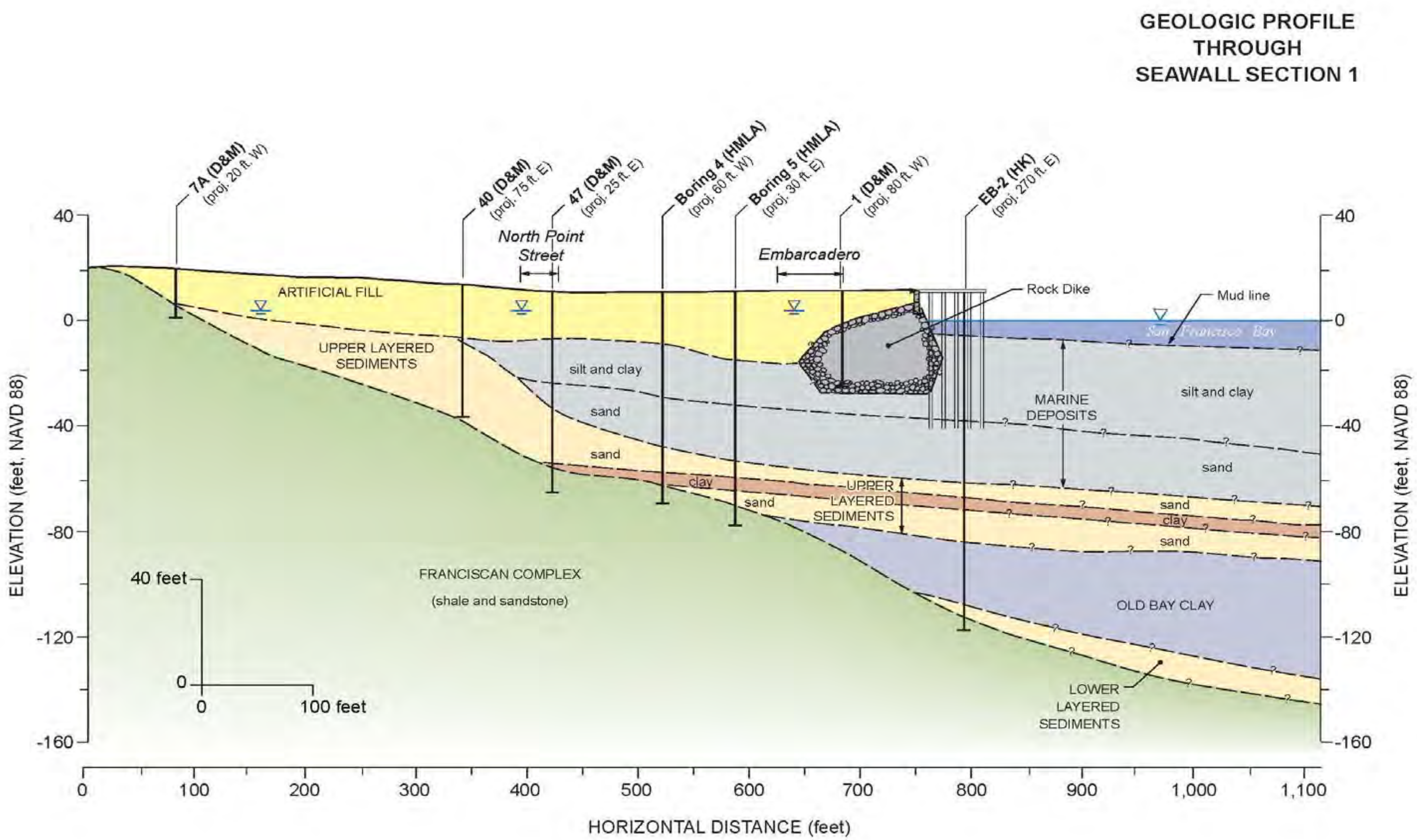


Figure 3-2: Geologic Profile Through Seawall Section 1

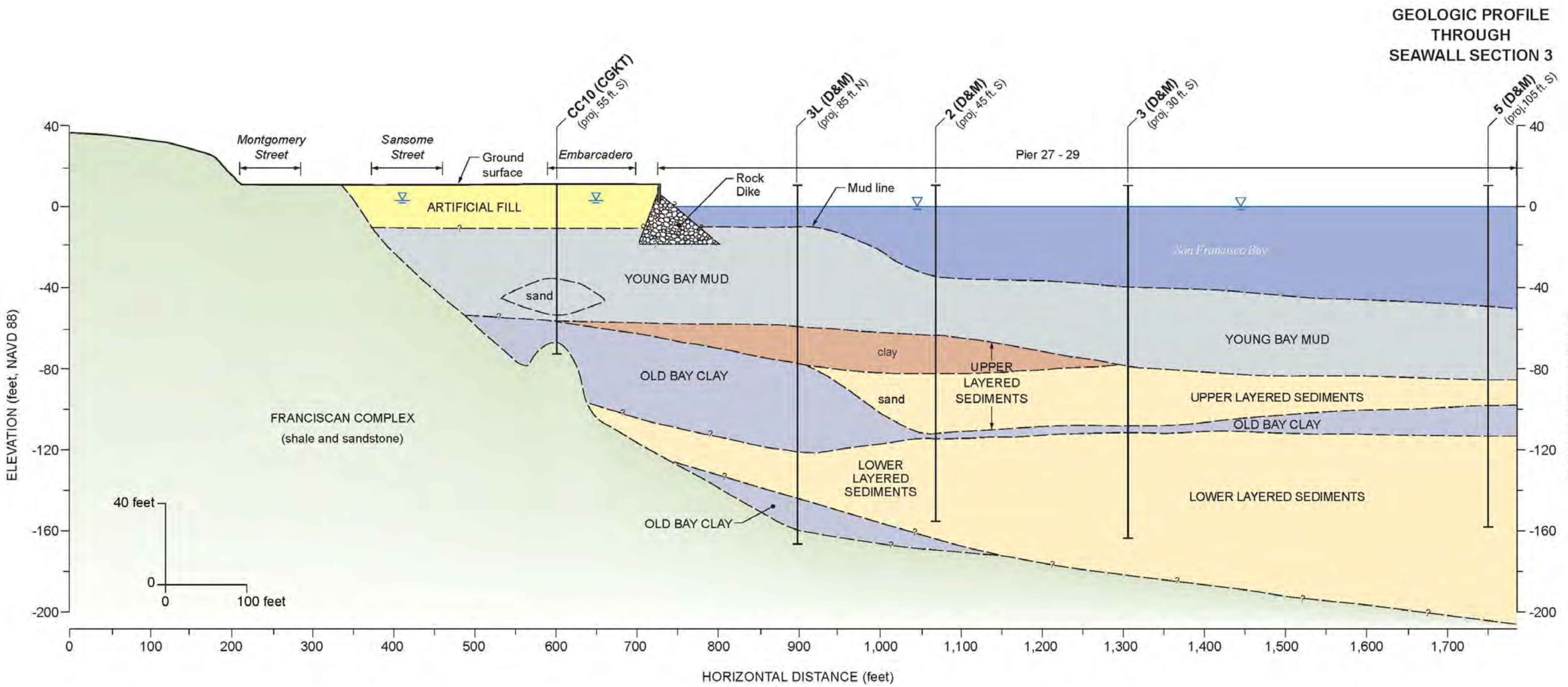


Figure 3-3: Geologic Profile Through Seawall Section 3

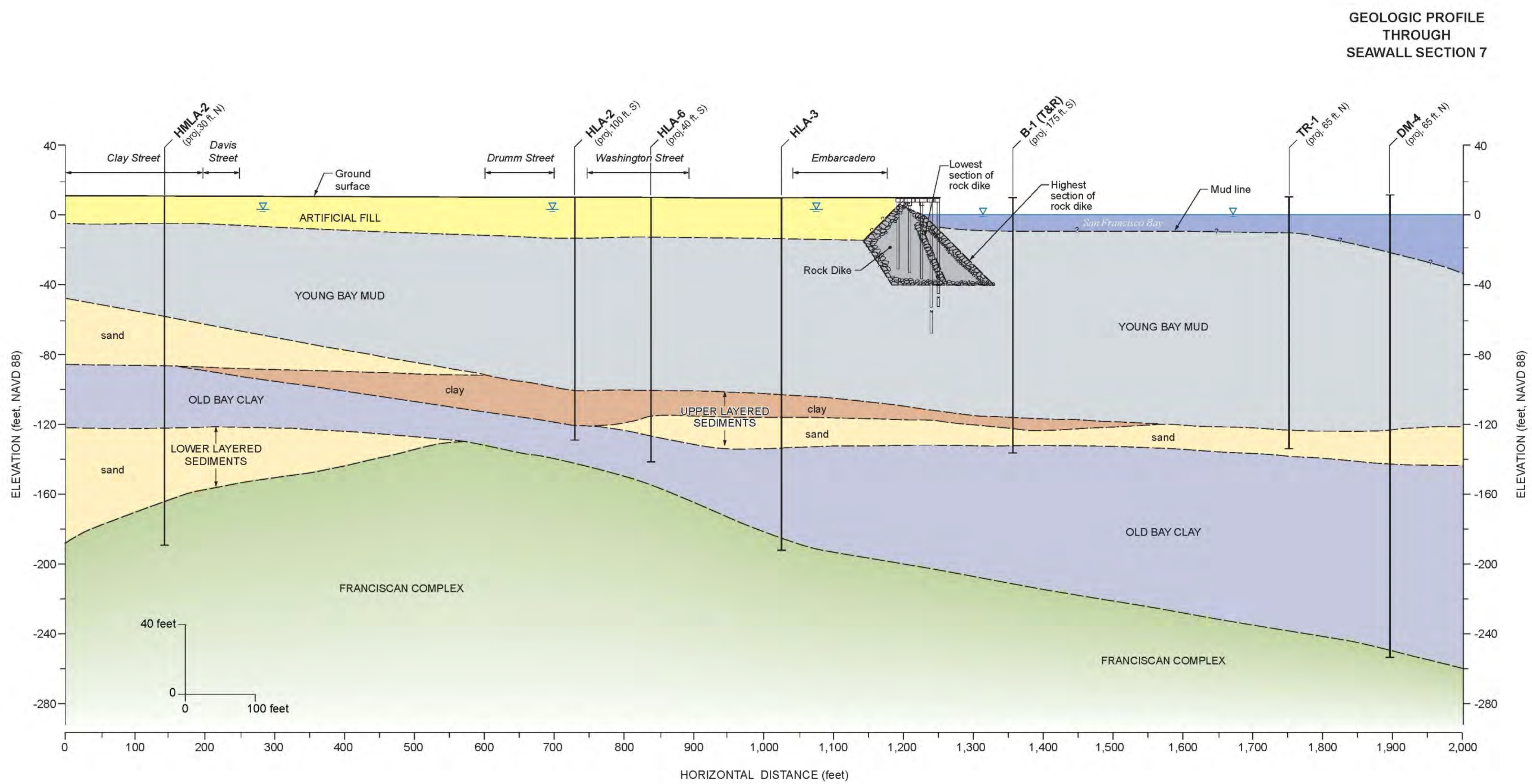


Figure 3-4: Geologic Profile Through Seawall Section 7

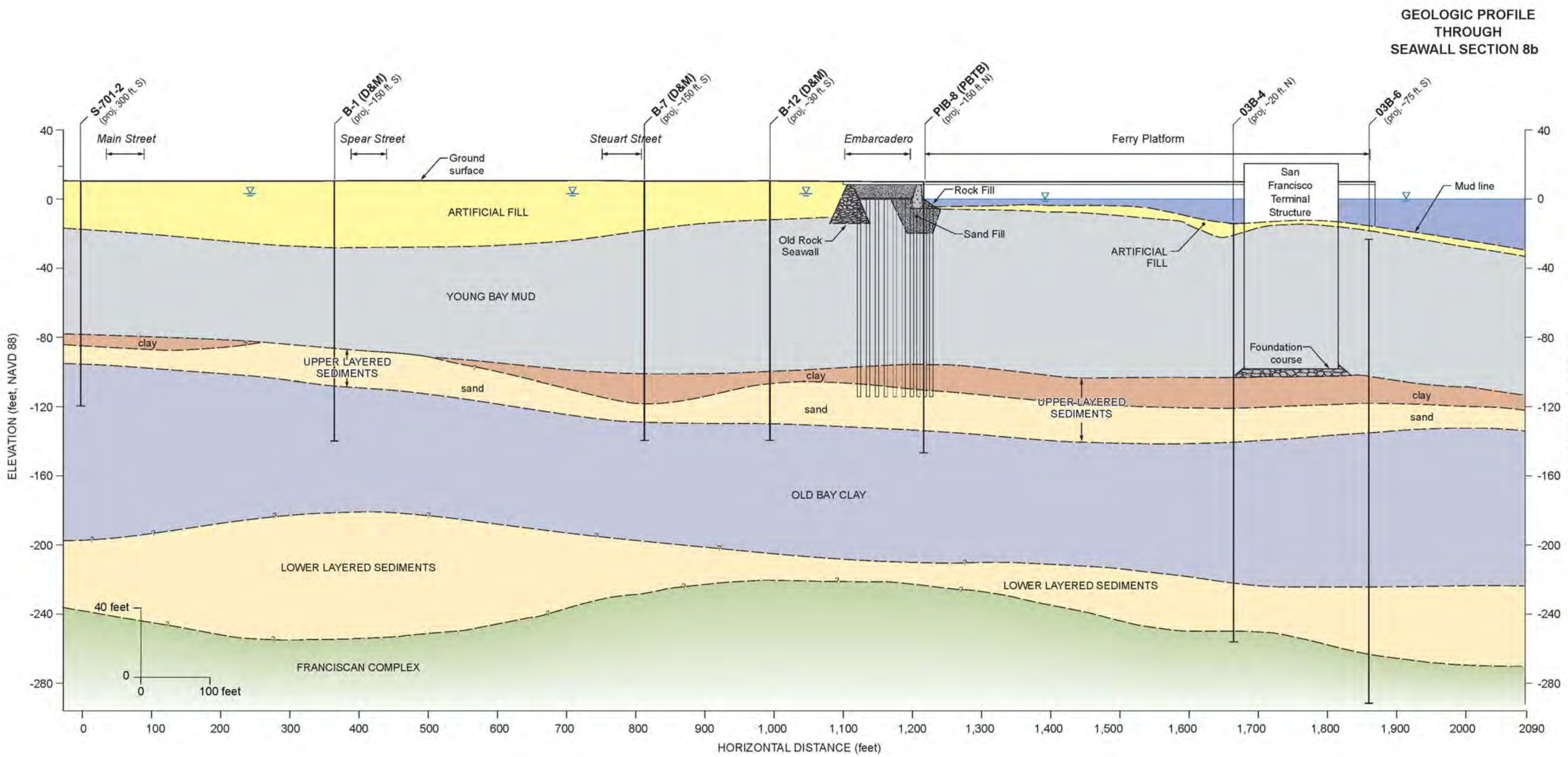


Figure 3-5: Geologic Profile Through Seawall Section 8b

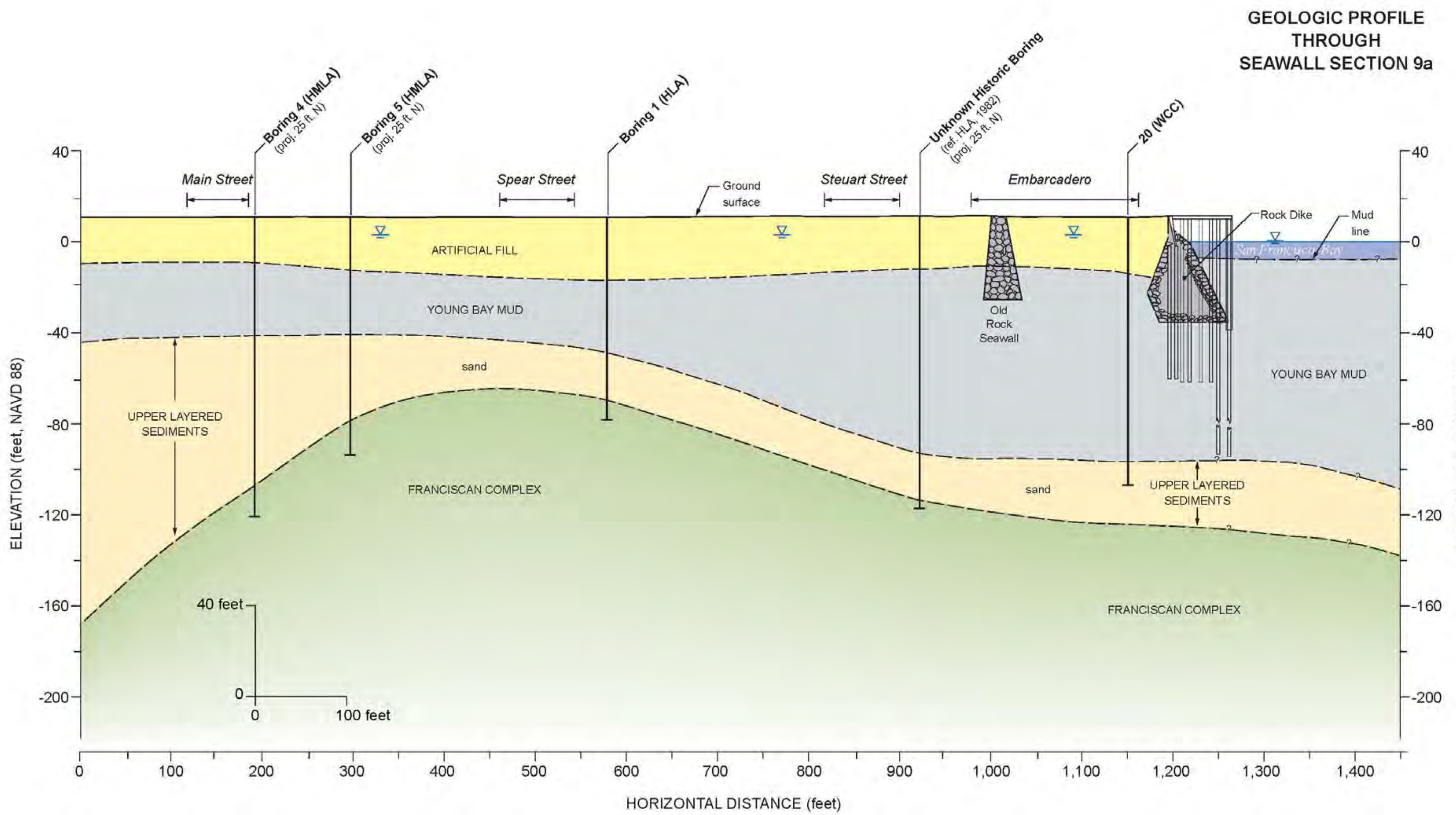


Figure 3-6: Geologic Profile Through Seawall Section 9a

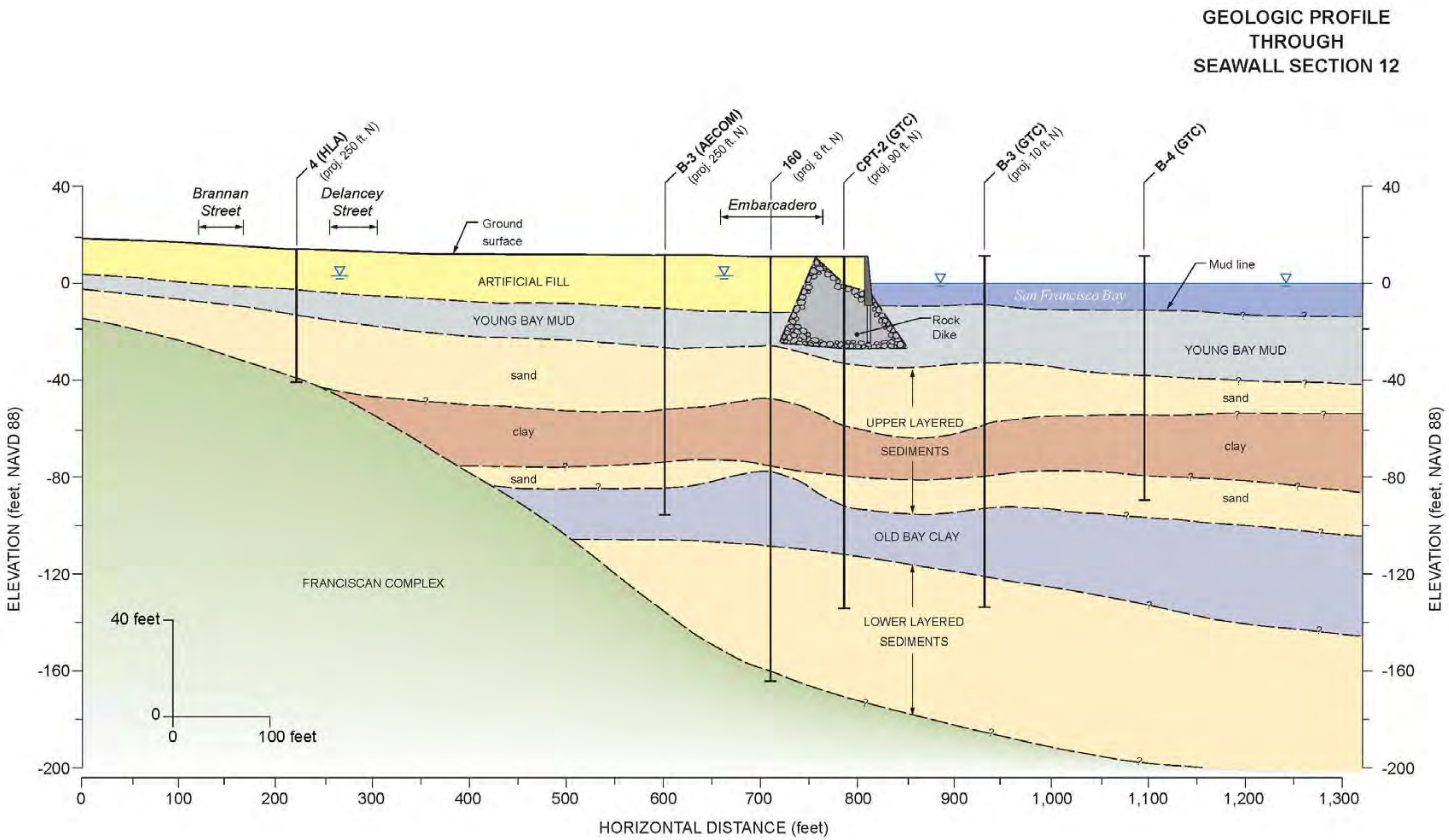


Figure 3-7: Geologic Profile Through Seawall Section 12

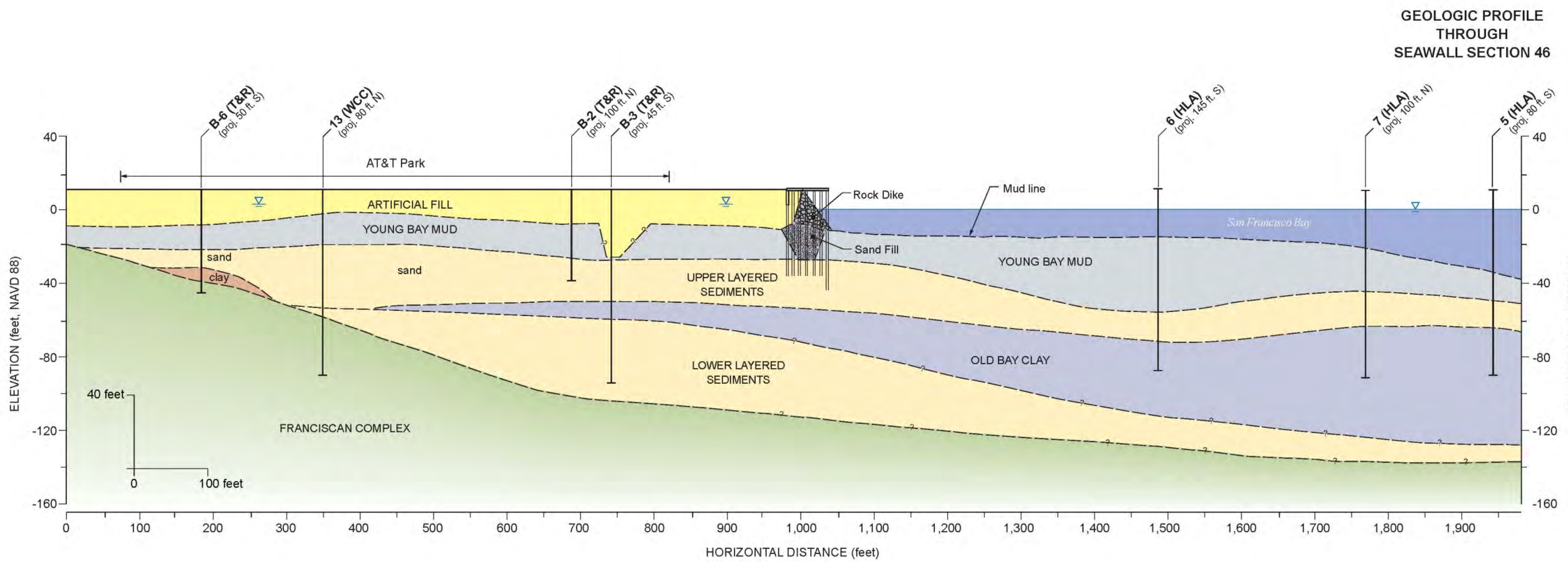


Figure 3-8: Geologic Profile Through Seawall Section 46

3.3. Seismic Ground Motions

Characterization of seismic ground motions serves as the basis for evaluating the performance of soils, foundations, and structures. Seismic strong ground motions contribute to two types of loading, or seismic demand, on structures: i) inertial loading as the result of near-surface ground shaking, and ii) secondary loading associated with permanent soil displacement against foundations or other portions of a structure resulting in a displacement demand on the structure, which is termed kinematic loading. The seismic performance of the seawall, embedded structures, pavements, underground utilities, and ancillary infrastructure is influenced by both inertial and kinematic loading. To evaluate the impact of inertial loading on a structure, the seismic ground motions can be characterized in the form of a ground surface acceleration response spectrum. The evaluation of kinematic loading, however, requires several additional steps of geotechnical analysis: i) evaluation of the potential for and possible extent of soil strength loss due to liquefaction and/or cyclic degradation; and ii) estimation of earthquake-induced soil displacement. Kinematic impacts to the seawall, adjacent piers, pile foundations, and structures are then estimated by the structural engineer using the anticipated soil displacements or forces.

The GHD/GTC team evaluated acceleration response spectra and estimated permanent soil displacements (“Index” Peak Ground Displacements, PGD’s) to evaluate inertial and kinematic loading, respectively. The analyses performed have been conducted using standard-of-practice procedures. Requisite input for these analytical procedures has been obtained from available geologic, geotechnical, and geophysical data from project files at GTC, NA, and the Port of San Francisco, and from publicly available technical literature. Geotechnical site investigation, such as in situ testing, drilling and sampling, or laboratory testing, was not performed as a part of this consultation. The extent of the geotechnical data available at each of the selected seawall sections varies substantially, and the conclusions and recommendations in this report reflect the assumptions and approximations necessary to provide the GHD/GTC team with input for structural analyses. The *estimations of seismic ground motions and “Index” permanent ground deformations are considered to provide reasonable ranges of anticipated seismic performance for a broad seawall seismic vulnerability investigation, the goal of which is to highlight primary vulnerabilities and identify key seawall sections and structures that warrant additional, more refined site-specific geotechnical and structural investigations*. The results of the seismic and geotechnical evaluations are intended to represent reasonable ranges of anticipated seismic behavior and performance at prescribed seismic hazard levels, and *should be interpreted as consistent with an “advanced screening” level of analysis* possibly leading to subsequent, more refined analyses at selected locations.

The impact of inertial loading on the priority asset structures was evaluated by structural engineering specialists on the project team using estimated ground surface response spectra prepared by GTC and NA for the various seismic hazard levels of interest, defined in terms of the Average Return Periods (ARPs). It should be noted that the *ground surface response spectra developed in this investigation are intended for use in seismic performance analyses of existing seawall structures only* and are not intended for design purposes. The estimated ground surface spectra represent “best estimates” of the anticipated ground motions at the seawall sections using trends from computed one-dimensional dynamic site response analyses from sites at and near the Port facilities. In this regard, variation should be anticipated between the recommended spectra for seismic performance assessment and any code-based spectra for use in structural design which are developed with a “squared” or “plateau”-type spectrum, and in conformance with additional code provisions. As previously mentioned, extensive geotechnical data were not available for all of the priority asset sites, and the anticipated ground motions were approximated, where necessary, using data from local sites and judgment-based estimation based on extensive work by GTC, NA, and others at the Port of San Francisco.

The impact of kinematic loading associated with permanent ground deformation (PGD) has also been evaluated for the selected seawall sections by the project team using the trends of “index” PGD versus ARP developed by GTC and NA. The vertical and lateral PGDs are considered “index” values in that they have been estimated using standard-of-practice engineering procedures in conjunction with currently available geotechnical data for each site. PGD trends were developed for free-field conditions. Site-specific adjustments to the free-field Index PGDs for aspects of soil-structure interaction, such as possible pile pinning or influence of buried structures and utilities have not been made in this phase of the project. The potential influence of pile pinning on computed lateral ground deformation was considered minor due to the combination of the following; (i) types of piles used (timber and relatively small, lightly reinforced concrete piles), (ii) spacing of the piles and size of pile groups, and (iii) age and condition of the piles. If deemed necessary, the Index PGDs can be modified using simple scaling relationships to account for soil-structure interaction for some of the pile-supported structures. The estimated PGDs are considered applicable and reasonable for the current, general seismic vulnerability assessment; however, they should not be used as the basis for subsequent site-specific design of mitigation schemes. On the basis of the Northern Waterfront seismic vulnerability assessment, we anticipate that additional geotechnical investigations may be completed for specific seawall sections and adjacent structures with more-refined analysis of dynamic soil-foundation-structure interaction performed during subsequent phases of the Port’s seismic vulnerability and resiliency planning.

Approach and Analyses

General

The requisite first step in the seismic performance assessment for the seawall sections is the characterization of the ground motions to be used by the GHD/GTC team members. The development of near-surface motions for use in geotechnical and structural analyses involves the following three steps; (i) estimation of bedrock motions underlying the site of interest, (ii) estimation of dynamic site effects and development of ground motion amplification factors (i.e., surface motion/bedrock motion) as functions of the strength of bedrock shaking, soil profile, and the period of structural response, and (iii) simple conversion of the bedrock motion to ground surface motion as the product of the bedrock spectral ordinate and the amplification ratio for the corresponding structural period (T) (e.g., 0 sec, 0.2 sec, 1.0 sec, and 2.0 sec). The remainder of the acceleration response spectrum for near-surface motions is developed by interpolation (T < 2.0 sec) or extrapolation (T > 2.0 sec), and smoothed to provide a representative spectral shape. The resulting spectra are not adjusted to have the truncated, plateau that characterizes code-based spectra.

The bedrock ground motions have been estimated as functions of the average return period (ARP), or inversely the Annual Frequency of Occurrence, and provided in the form of acceleration response spectral ordinates for four selected oscillator periods (0, 0.2, 1.0, and 2.0 seconds). The complete acceleration response spectrum for bedrock motions (0 sec < T < 4.0 sec) at a specific seawall section and ARP can be developed from the four spectral ordinates based on the shape of the smoothed spectra generated using contemporary ground motion prediction equations (GMPEs). This is accomplished by matching the shapes of the normalized spectra (SA/PGA) for the specific site with the results of the GMPEs for the predominant seismic sources (e.g., M 8.0 at 13 km). To develop trends in spectral accelerations for the four structural periods noted as a function of ARP, site response, liquefaction, and earthquake-induced ground deformation were evaluated for the following six hazard levels, i.e., ARP:

72 years, 225 years, 475 years, 750 years, 975 years, and 2,475 years. These particular ARPs were selected for the sole purpose of defining the site response and ground deformation over a range of earthquake hazard levels from a relatively frequent earthquake (ARP of 72 years) to a rare earthquake (ARP of 2,475 years). The results could then be used by the GHD/GTC team to evaluate the site response and ground deformation behavior at any intermediate ARP without further geotechnical and seismic analysis. This information was used by the GHD/GTC team to evaluate the influence of inertial and kinematic loading on structural performance. The following sections of this report provide a brief summary of our approach to the different analyses.

Seismic Hazard Evaluation and Characterization of Seismic Motions on Bedrock

To evaluate the regional seismic hazard and characterize bedrock ground motions at the locations of the selected seawall sections, we reviewed the results of the 2008 and 2014 U.S. Geological Survey (USGS) Probabilistic Seismic Hazard Analyses (PSHA). For a given site location and ARP, the PSHA provides estimates of the ground motions on bedrock (Site Class B/C boundary) in terms of response spectral ordinates based on a probabilistic evaluation of the spatial and temporal occurrence of earthquakes throughout the San Francisco Bay region. The bedrock motions (geometric mean) estimated by the PSHA are presented in the form of the Uniform Hazard Spectrum (UHS), which serves as the basis for many design standards. The results of the PSHA also provide detailed information regarding the predominant seismic sources that contribute to the ground motions at the selected ARPs. This process, referred to as seismic hazard "deaggregation," is necessary for subsequent geotechnical analyses, such as liquefaction susceptibility and earthquake-induced permanent ground deformation. The GHD/GTC team has used the results of the 2008 USGS PSHA and deaggregation along with the 2014 NGA-West2 Ground Motion Prediction Equations (GMPEs) as the basis for the ground motion characterization. The deaggregation data for the 2014 edition of the USGS PSHA was not available at the time this work was completed. From a comparison of the seismic source catalog and ground motion estimation procedures (i.e., GMPEs) used in the 2008 and 2014 editions of the USGS regional PSHA investigations, it appears that the differences in the estimated ground motions along the Embarcadero waterfront for the seismic hazard levels selected in this study are small. This observation has been confirmed by Powers and Peters (2015) who state that "*Hazard in San Francisco is largely unchanged from 2008 and 2014*." The spectral acceleration estimates from the 2014 NGA-West2 GMPEs provide the geometric mean of two orthogonal directions of ground motion, the results of which were adopted for this study. The trends of 0-, 0.2-, 1.0- and 2.0-second spectral ground motions on bedrock/firm base (Site Class B/C boundary) conditions are shown on ***Figure 3-15***. The variability in the bedrock ground motions were found to be very minor along the Northern Waterfront Seawall alignment for all ARPs therefore the trends provided in ***Figure 3-15***, obtained for Pier 1 ½, are considered applicable for all of the seawall sections evaluated in this investigation.

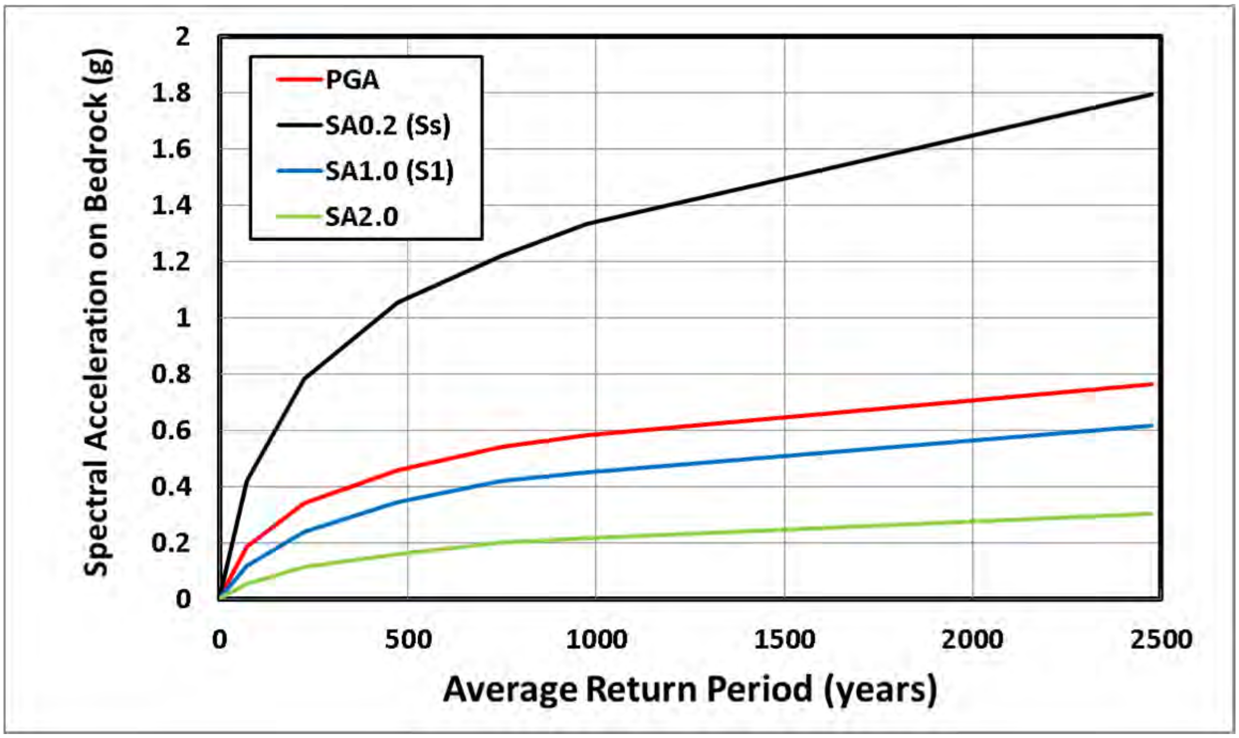


Figure 3-9: Variation of Spectral Acceleration (2014 NGA-West2 GMPEs) with Average Return Period for Bedrock Site Conditions (Site Class B/C Boundary).

Dynamic Soil Response and Site Effects

The seismic performance of the seawalls and adjacent structures is a function of the anticipated seismic motions at or very near the ground surface, as opposed to the seismic motions on bedrock. The anticipated ground surface motions are greatly influenced by soil conditions, requiring dynamic soil response analysis to estimate the site-specific influence of the soil deposits on bedrock ground motions. The site-specific and nonlinear influence of soils on bedrock motions is typically quantified using a Spectral Amplification Ratio (SAR), which is defined as the ratio of the ground surface to bedrock seismic motions at a given spectral period. The SAR is a function of soil stratigraphy (types and thickness), soil stiffness and cyclic behavior, as well as the amplitude of the bedrock motions. It should be noted that the SAR can be greater or less than unity, demonstrating an increase or decrease, respectively, in the amplitude of the bedrock motions at a given ARP.

To streamline the evaluation of dynamic soil response and make full use of numerous previous analyses performed for the Port of San Francisco, GTC and NA estimated the SARs at the ARPs of interest based on three routine methods: i) consideration of the spectral amplification factors provided in current building codes and standards, which are based on soil characteristics in the upper 100 feet of the profile; ii) trends in amplification from site-specific modeling performed by others (mostly using the equivalent linear model SHAKE), and iii) trends in amplification for three generalized waterfront soil profiles from modeling performed by GTC and NA using the nonlinear model DEEPSOIL for total stress analysis to be consistent with the complementary SHAKE analyses and for the development of profiles of Cyclic Stress Ratio with depth for use in simplified liquefaction triggering analyses. The port-specific approaches (methods (ii) and (iii)) are considered to be the most relevant for this project given the soil profiles of deep, soft clay that are present along most of the seawall alignment and the inherent limitations of the code-based procedures to account for soils below a depth of 100 ft (30 m).

It is noted that the alternate method of using current Ground Motion Prediction Equations (GMPEs) to estimate ground surface motions was evaluated for use on the project, but not implemented for the following reasons. Contemporary GMPEs (i.e., PEER NGA-West 2) account for, in a simple manner, the influence of soils in the upper 100 ft (30 m) of the profile on the characteristics of the motions. The ground motion predictive equations include site conditions by way of a time-averaged, low-strain shear stiffness in the upper 30 m of the soil profile [the $(V_s)_{30}$]. The GMPEs are applicable for soil profiles having $(V_s)_{30}$ values within prescribed ranges. The $(V_s)_{30}$ values for most of the seawall sections fall well below the range of application for most of the GMPEs. For this reason, and given the fact that the soil profiles at many of the selected seawall sections have thicknesses that are significantly greater than 30 m, the GMPEs were deemed inappropriate for estimating SARs along the seawall alignment.

The approach to evaluating site response for this project consisted of the following specific steps:

1. Compile shear wave velocity (V_s) data from sites in proximity to the seawall. The limited set of V_s data was supplemented by estimating values using correlation equations developed by NA for local applications along the margins of San Francisco Bay. This data was used to compute the average shear wave velocity in all soils and bedrock.
2. Develop a generalized microzonation scheme for the seawall alignment based on; the V_s profiles for port-specific projects and selected seawall sections, the geotechnical soil profiles, and contour maps for the thickness of selected soil layers and depth to bedrock. The simplified microzonation along the waterfront consisted of the following;

Table 3-1: Seismic Microzonation

SEAWALL SECTION	ZONATION CODE
FW	A1
B	A1
A	A1
1	A1
2	A1
3	B1
4	B1
5	B2
6	C2
7	C2
8a/8b/8	C2
9a	C2
9b/9	B1
10	A2
11a/11	A2
12	A2
13	A2
46	A2

ZONATION CODE

Thickness of Young Bay Mud

A: 10 to 25 ft
B: 25 to 60 ft
C: 60 to 120 ft

Depth to Bedrock

1: 50 to 120 ft
2: 120 to 250 ft

3. Review and compile the results of several site-specific, free-field, one-dimensional site-response analyses completed at, and in the vicinity of, the Port by others. It is noted that most of the dynamic response analyses reviewed for this project were performed using the equivalent linear, total stress model SHAKE.
4. Using the results of both the SHAKE and DEEPSOIL site-response analyses, develop Port-specific trends for soil amplification/de-amplification of bedrock motions as functions of input shaking level (i.e., ARP or Seismic Hazard Level). As previously discussed, soil amplification/de-amplification is most commonly described using a spectral amplification ratio (SAR), which is defined as the ratio of the spectral ordinates for ground surface to bedrock motions at the period of interest. The SAR was specifically evaluated at four discrete oscillator periods of interest; 0.0 seconds (PGA), 0.2 seconds, 1.0 second, and 2.0 seconds. From these four data points the complete acceleration response spectra can be developed.

   The SAR values at each oscillator period were found to define relatively uniform trends with ground motion level (i.e., ARP) for the selected seawall sections and microzonation scheme employed. The trends of SAR with ARP are provided in ***Figures 3-16 through 3-19***. It should be noted that the SAR trends are supplemented with ground motion data recorded during the 1989 Loma Prieta earthquake at ten sites underlain by young bay mud located along the margins of the San Francisco Bay. This data is worthwhile for bracketing the likely range of SAR at lower amplitudes for port waterfront sites.
5. Estimate the site-specific ground surface response spectra at each ARP of interest by multiplying the 2008 USGS Site Class B/C UHS values by the port-specific SAR trends. The resulting trends of ground surface motion for the four oscillator periods at various ARPs are provided in ***Figures 3-20 to 3-23***. The trends have been smoothed to allow for subsequent, possible curve-fitting associated with structural modeling efforts. Our recommended seawall section-specific spectral ordinates for seismic performance analyses were based on the following considerations and judgment; (i) trends developed using the SHAKE modeling results, (ii) trends developed using the DEEPSOIL modeling results, (iii) strong motion data recorded at soft and deep clay soil sites during the 1989 Loma Prieta earthquake, and (iv) the general trends of ground motion with ARP prescribed by ASCE 7-10 code provisions.
6. The generation of acceleration response spectra for ground surface motions at specific ARPs facilitates comparison of the site-specific response spectra with pertinent codes and seismic design provisions, such as the ASCE 7-10 ground surface spectra. This was performed by structural engineering members of the GHD/GTC team as a portion of subsequent analyses.

For the deep young bay mud soil profiles existing along much of the seawall alignment, the recommended ground surface motions for seismic performance and fragility analysis are commonly lower than code-based values. As previously discussed, the recommended motions represent “best-estimate” trends for structural performance assessment and are not intended for design purposes, which require adherence to applicable standards and codes. In this regard, the resulting ground surface motions for vulnerability analysis do not necessarily match the ground motions developed in accordance with ASCE 7-10, ASCE 41-13, or the Port of San Francisco Seismic Design Code.

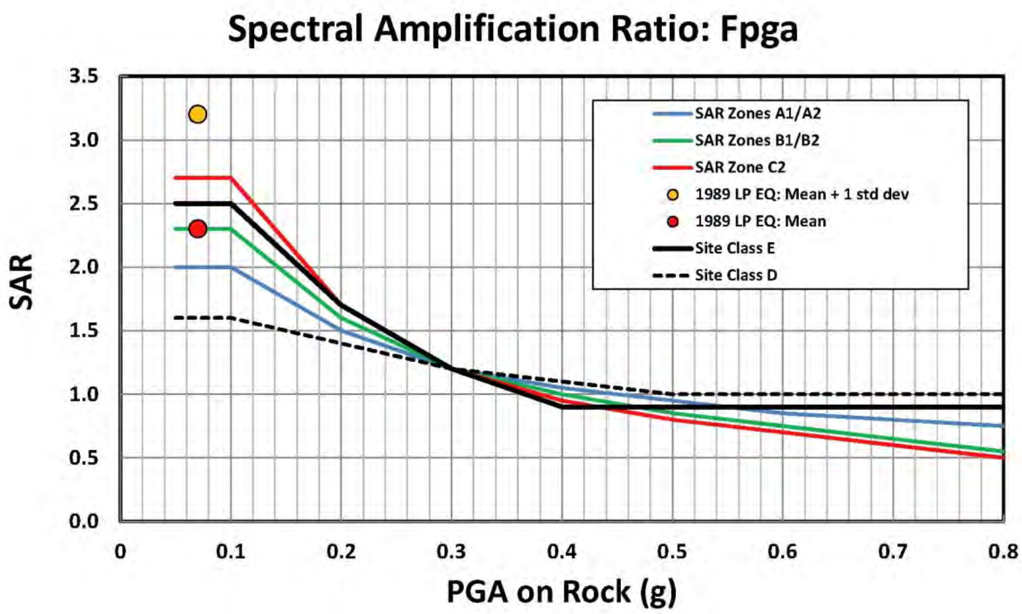


Figure 3-16:Trend of Spectral Amplification Ratio with Amplitude of the Bedrock Motions for the Zero Period (PGA).

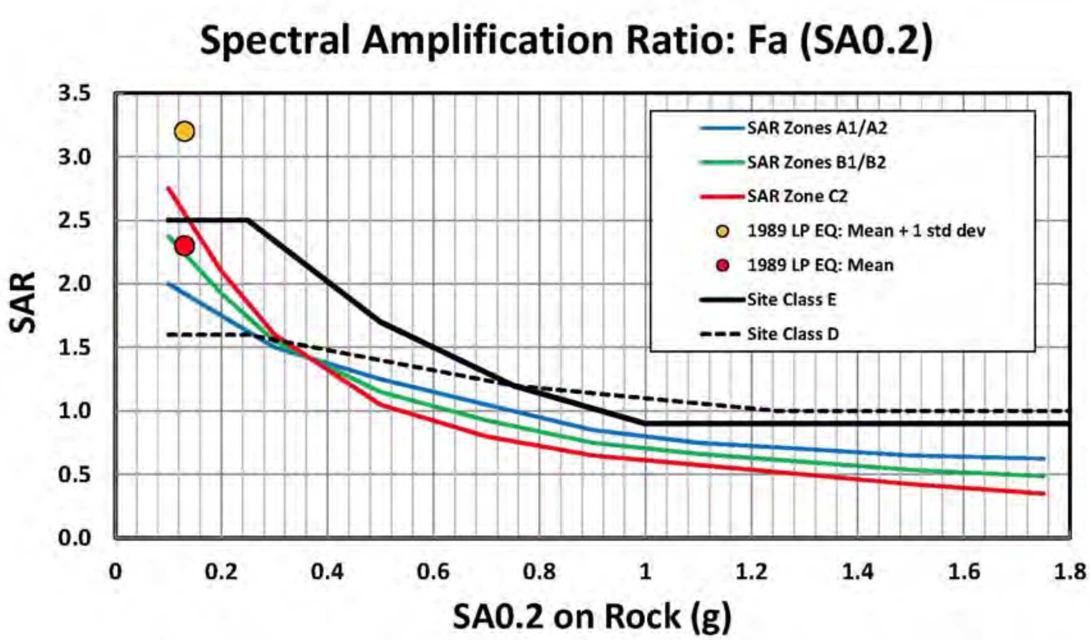


Figure 3-17:Trend of Spectral Amplification Ratio with Amplitude of the Bedrock Motions for the 0.2-Second Oscillator Period ($SA_{0.2}$).

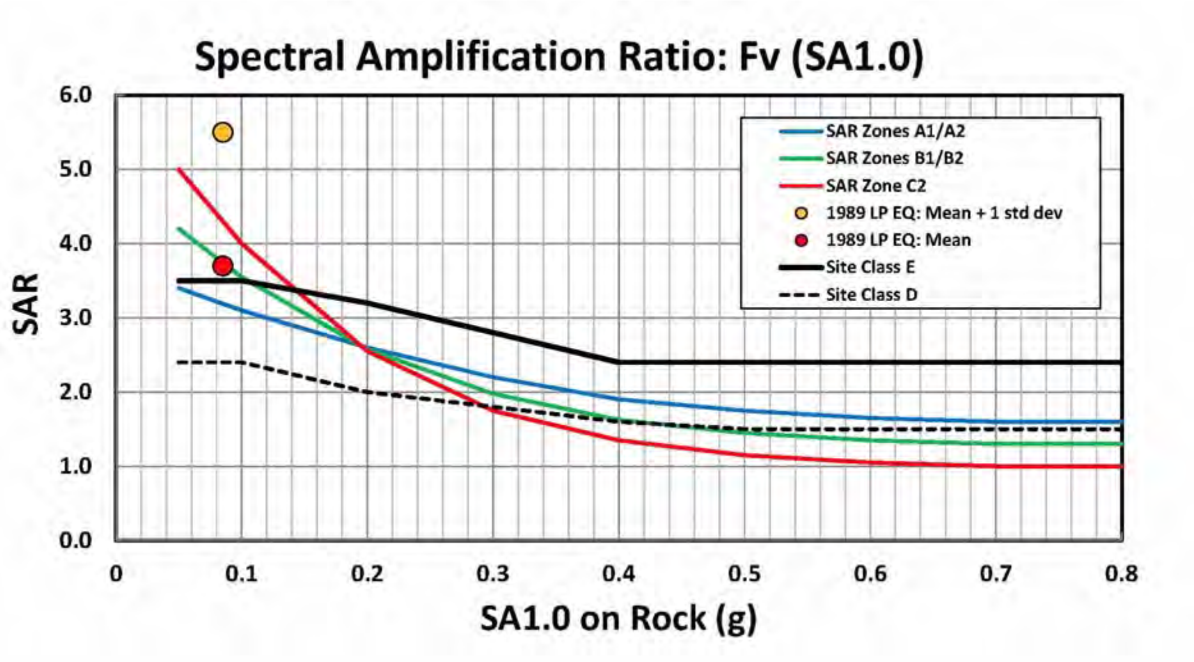


Figure 3-108: Trend of Spectral Amplification Ratio with Amplitude of the Bedrock Motions for the 1.0-Second Oscillator Period ($SA_{1.0}$).

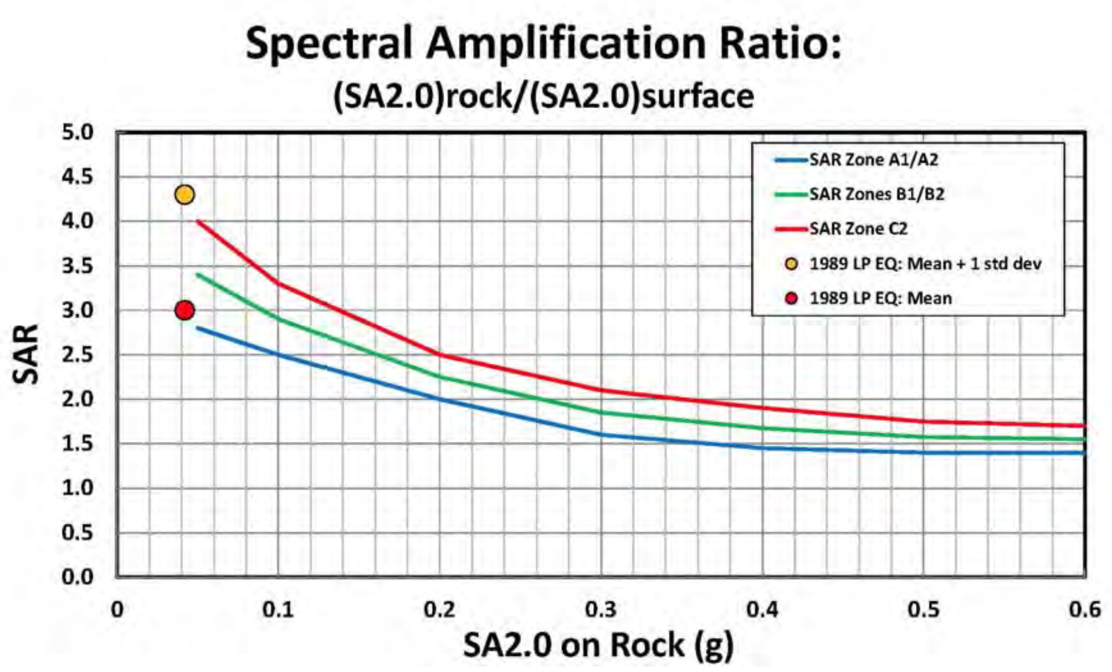


Figure 3-19: Trend of Spectral Amplification Ratio with Amplitude of the Bedrock Motions for the 2.0-Second Oscillator Period ($SA_{2.0}$).

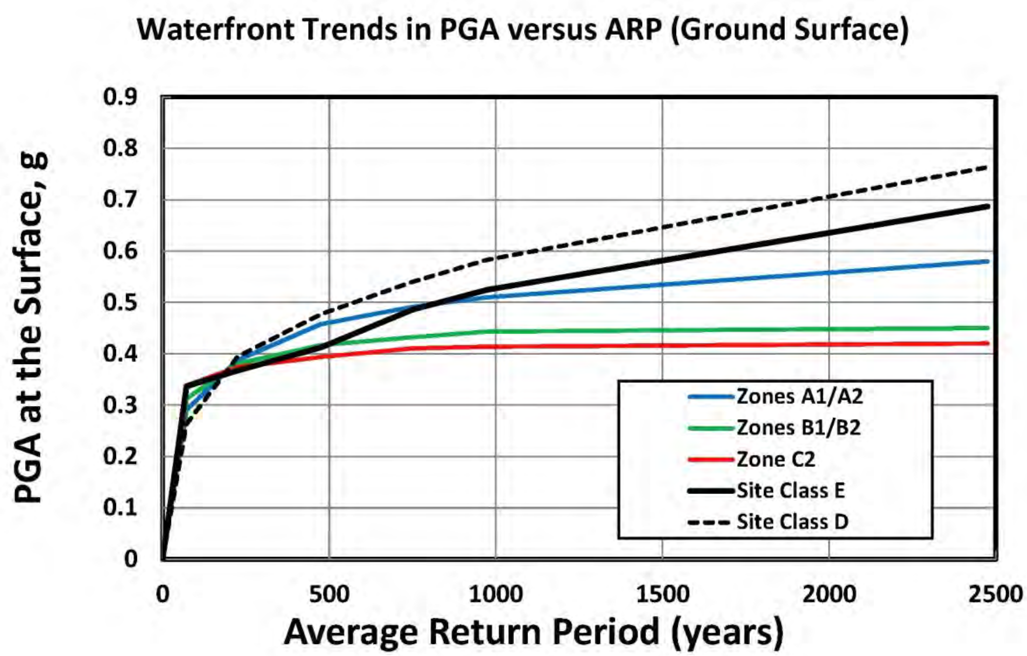


Figure 3-20:Trend of Ground Surface PGA with Average Return Period.

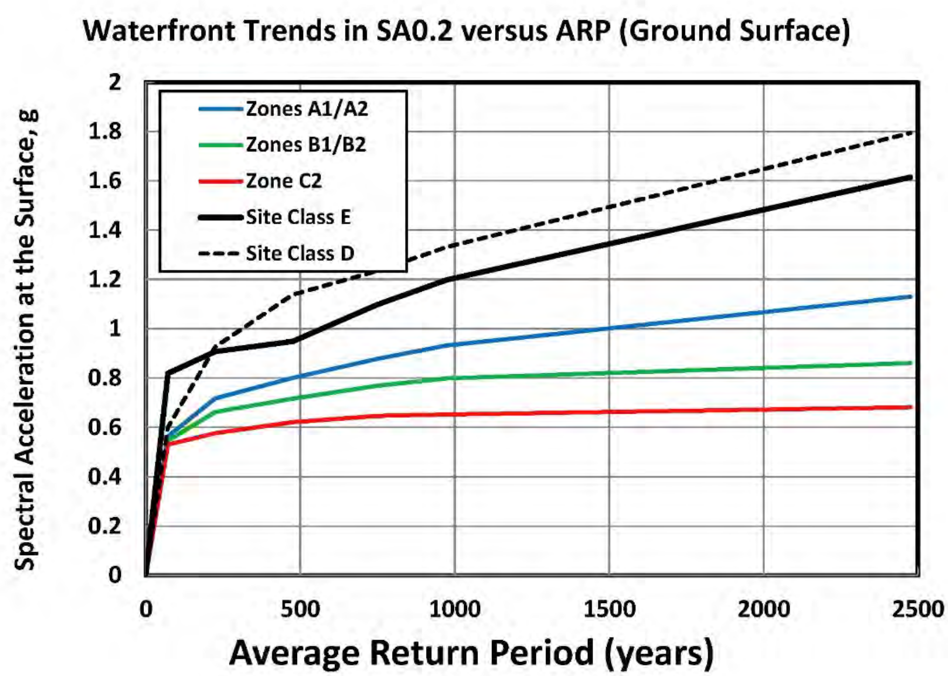


Figure 3-2111: Trend of Ground Surface Spectral Acceleration at 0.2-second Period with Average Return Period.

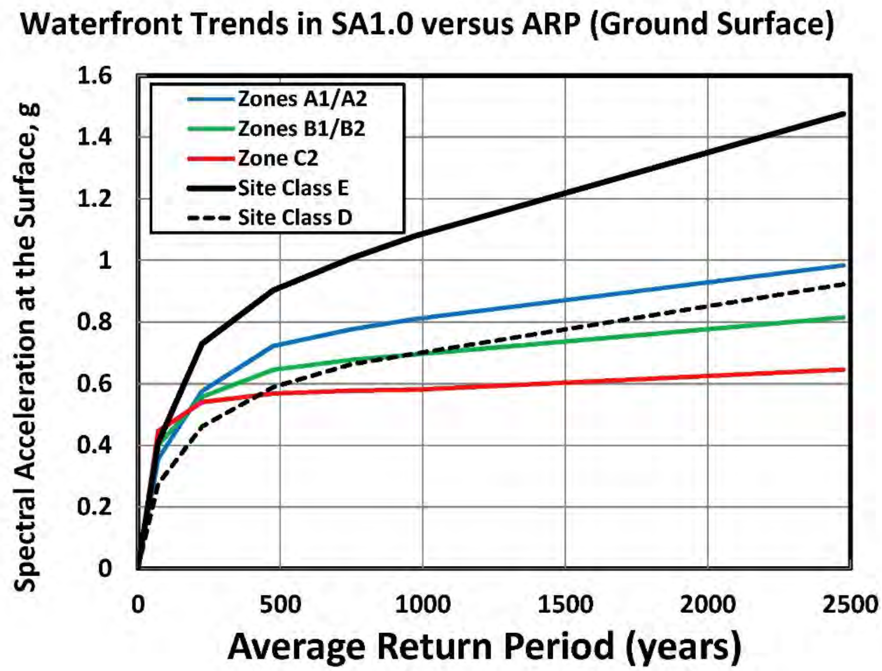


Figure 3-22:Trend of Ground Surface Spectral Acceleration at 1.0-second Period with Average Return Period.

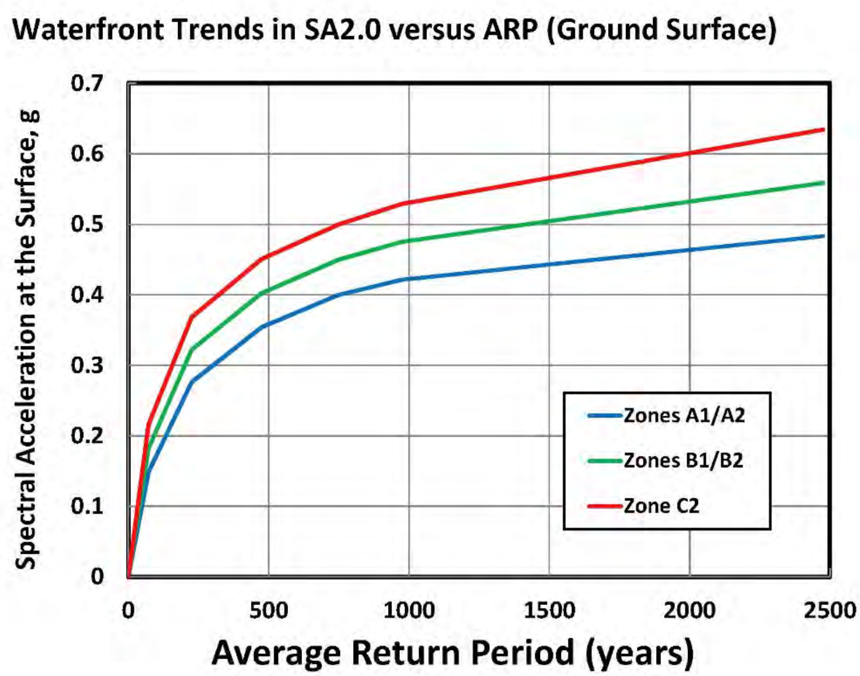


Figure 3-23Trend of Ground Surface Spectral Acceleration at 2.0-second Period with Average Return Period.

3.4. Slope Stability

General

The GHD/GTC team evaluated the slope stability and lateral spread potential of representative cross sections along the Northern Waterfront Seawall. The slope stability analyses results are shown on ***Figures 3-25 through 3-54***. The sections were selected based on several factors including: i) representative of a specific portion of the Northern Waterfront Seawall, ii) a larger amount of available subsurface information, iii) areas where the seawall was unique and may present increased or decreased risk from the more typical seawall section, and iv) areas where a previous lateral spread analysis was performed. In our analyses of slope stability and lateral spread potential, we generally followed the procedures outlined in the Guidelines for Analyzing and Mitigating Landslide Hazards in California (SCEC, 2002). This involved evaluating static and pseudo-static (seismic) slope stability using General Limit Equilibrium (GLE) procedures. For the seismic slope stability, a yield acceleration, k_y, was determined at each representative seawall section for subsequent analysis using Newmark-type sliding block analyses. The results of the limit equilibrium analyses are provided below, and the results of the lateral spread analysis are provided in *Section 3.7*.

Generalized Soil Properties

Soil strengths were characterized as follows: undrained shear strengths were used for clayey soils, and drained strength parameters were used for sandy soils in a non-liquefied condition. Our seismic analysis considered both peak soil strengths at the initiation of slope movement and residual soil strengths at large strains. Soil parameters used in the slope stability analyses are presented below in ***Table 3-2 – Geotechnical Parameters for Slope Stability Analysis***.

Table 3-2: Geotechnical Parameters for Slope Stability Analysis

Soil Layer	Total Unit Weight, γ (pcf)	Friction Angle, ϕ (°)	Cohesion, c (psf)	Peak Min. Shear Strength[1] (psf)	Peak Shear Stress Ratio [1]	Residual Shear Strength (psf)	Residual Minimum Shear Strength (psf)	Residual Shear Stress Ratio
Artificial Fill [2]	115-132	32				250		
Rock Dike Fill	120	36	200					
Rock Dike Sand Fill [2]	130	30				400		
Young Bay Mud (Bayside)	90-113			100	0.28		0	0.22
Young Bay Mud (Shoreside)	90-113			100	0.28		0	0.22
Marine Deposits Sands [2]	115-132	28-30	0-200			400		
Marine Deposits Clays	105		500-650					
Upper Layered Sediment Sands	132-139	36-40	0-200					
Upper Layered Sediment Clays	126-138		1500-2600					
Old Bay Clay	108-126		1500-2600					
Lower Layered Sediments	120-138	35-42						

Notes:

1. Shear stress ratio is the increase in shear strength per unit increase of vertical effective overburden stress. The peak minimum shear strength of 100 psf and shear stress ratio of 0.28 for the Young Bay Mud under static conditions was increased by 30 percent to account for strain rate effects in pseudo-static analyses.
2. Liquefied sands were modeled as cohesive soils with residual shear strengths as shown.

Static Stability Analysis

We used the computer program Slope/W to perform limit equilibrium analysis of eight slope stability sections. Morgenstern and Price's GLE method of analysis was utilized to compute static factors of safety for both circular and sliding block failure surfaces. The most critical failure surface was then selected for each section. The factors of safety for the critical circular and block failure surfaces were typically very close. Generally, a static factor of safety (FS) of 1.5 is desirable to limit the risk of slope instability due to uncertainties in the soil strength parameters. Our analyses indicated static factors of safety ranging from 1.6 to 2.4.

Calculated factors of safety for each section analyzed are tabulated in ***Table 3-3 – Summary of Limit Equilibrium Analysis Results***. Output graphics showing the critical static failure surface and factor of safety for each section are provided in the figures following this section of the report.

Seismic Stability Analysis

A pseudo-static representation of seismic loading using General Limit Equilibrium (GLE) methods was used through Slope/W to evaluate the seismic stability of the eight slope stability sections. The yield acceleration, k_y, which is the horizontal seismic coefficient corresponding to a slope stability factor of safety of 1.0, was evaluated for both peak strength conditions and residual strength conditions. Peak strength conditions were considered to be representative of the soil behavior during the initial stages of earthquake shaking. Weak clays, such as young bay mud, exhibit an increased strength for dynamic loading conditions, and therefore the peak dynamic shear strengths were increased by 30 percent compared to the strengths assuming long-term static loading. With strong ground shaking, certain soils such as young bay mud and liquefiable sands have a degradation in strength, and at large strains reach a residual strength condition. For one section, Seawall Section 46, the calculated factor of safety for post-cyclic residual strength conditions was 0.86 indicating the possibility of a flow failure and more significant lateral spread. This is due to a potentially liquefiable stratum of sandy fill placed within the rock dike section of the seawall. Other seawall sections were controlled more by the properties and behavior of the young bay mud and other soft marine deposits underlying the rock dike sections than by the properties of the rock dike fill.

Calculated yield accelerations are tabulated in ***Table 3-3 – Summary of Limit Equilibrium Analysis Results***. Output graphics showing critical failure surfaces for peak strength and residual strength conditions for each section are provided in the figures following this section of the report.

Table 3-3: Summary of Limit Equilibrium Analysis Results

Section	Critical Failure Surface Circular / Block	Static Conditions FS	Pseudo-static Peak Strength Conditions k_y	Pseudo-static Residual Strength Conditions k_y
B	Circular	2.4	0.18	0.07
1	Block	2.3	0.15	0.09
3	Circular	1.6	0.12	0.01
7 “High Wall”	Circular	2.0	0.14	0.05
7 “Low Wall”	Circular	1.7	0.13	0.03
8b-b	Circular	2.8	0.14	0.06
8b-c	Circular	1.9	0.14	0.05
9a	Circular	1.7	0.13	0.03
12	Block	2.2	0.15	0.05
46	Circular	1.8	0.12	0.00 [2]

Notes:

2 FS = factor of safety. k_y = yield acceleration.

3. Factor of safety less than 1.0 during post-cyclic conditions due to liquefaction of sandy fill in the rock dike profile.

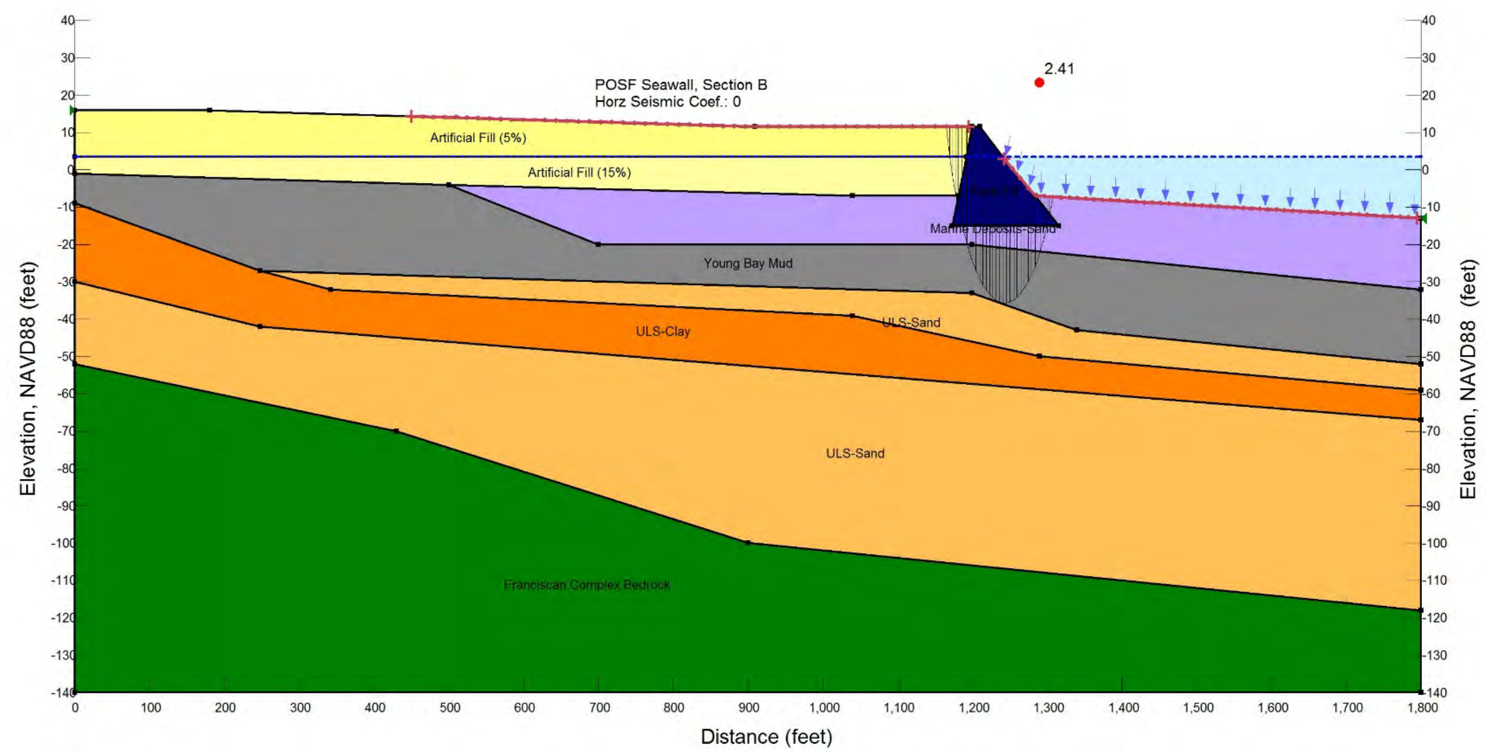


Figure 3-24: Static Slope Stability at Seawall Section B

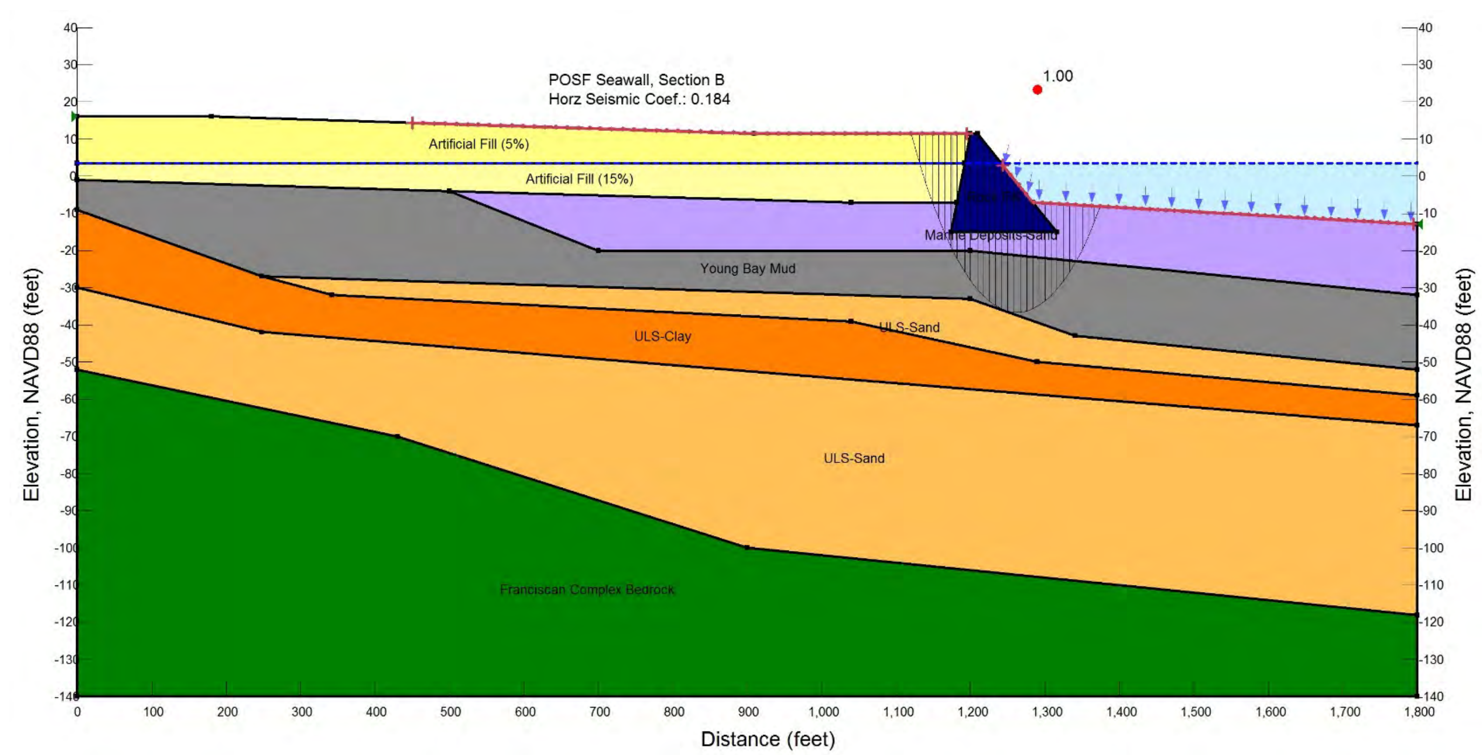


Figure 3-25: Seismic Slope Stability at Seawall Section B – Peak Strength Conditions

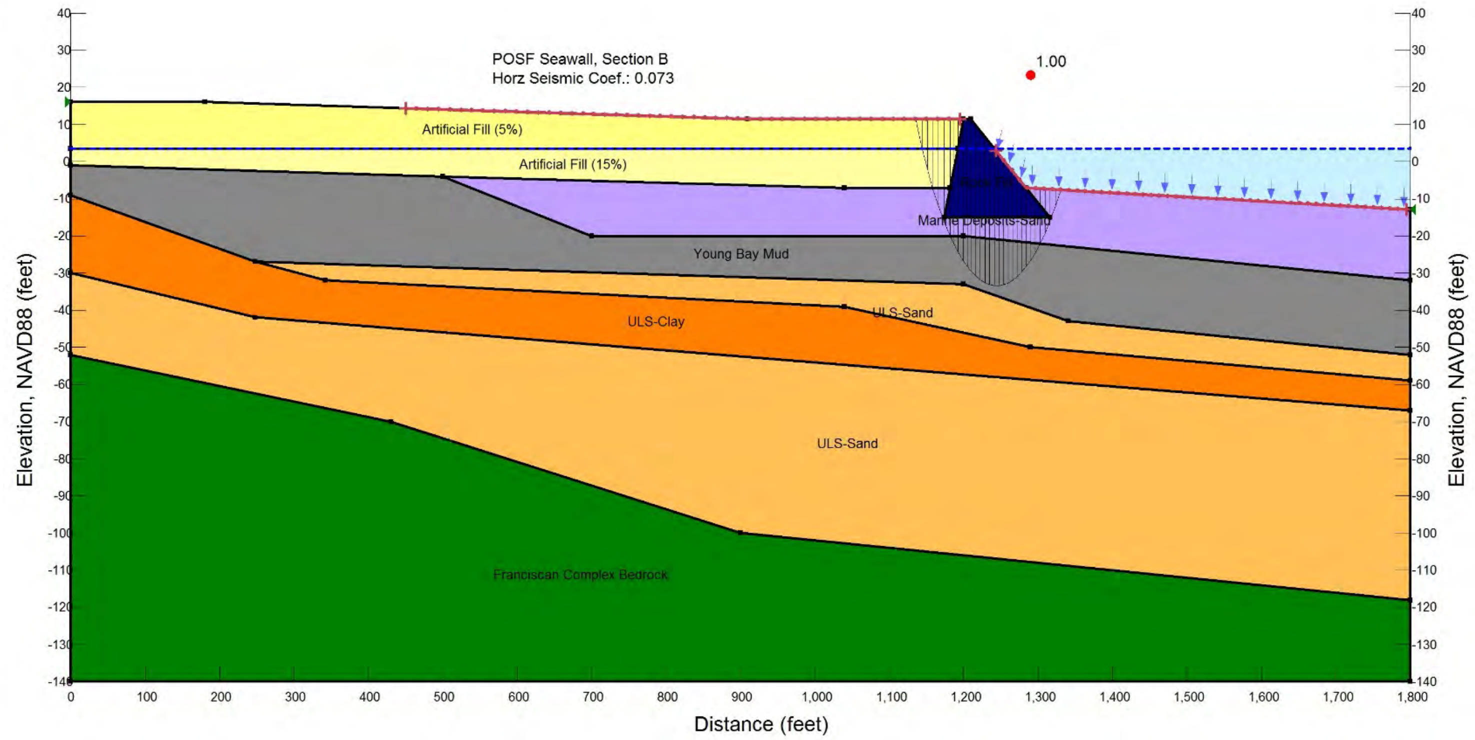


Figure 3-26: Seismic Slope Stability at Seawall Section B – Residual Strength Conditions

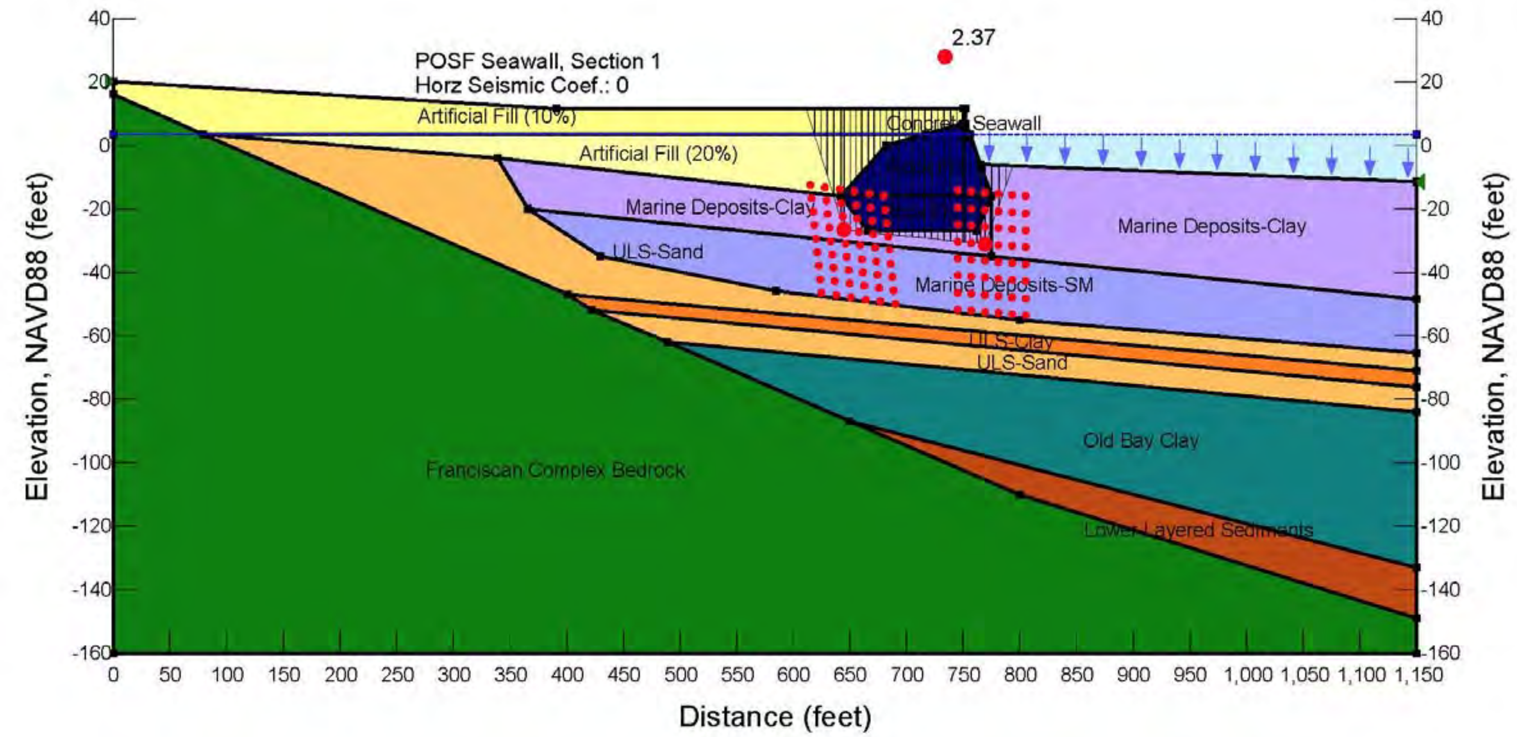


Figure 3-27: Static Slope Stability at Seawall Section 1

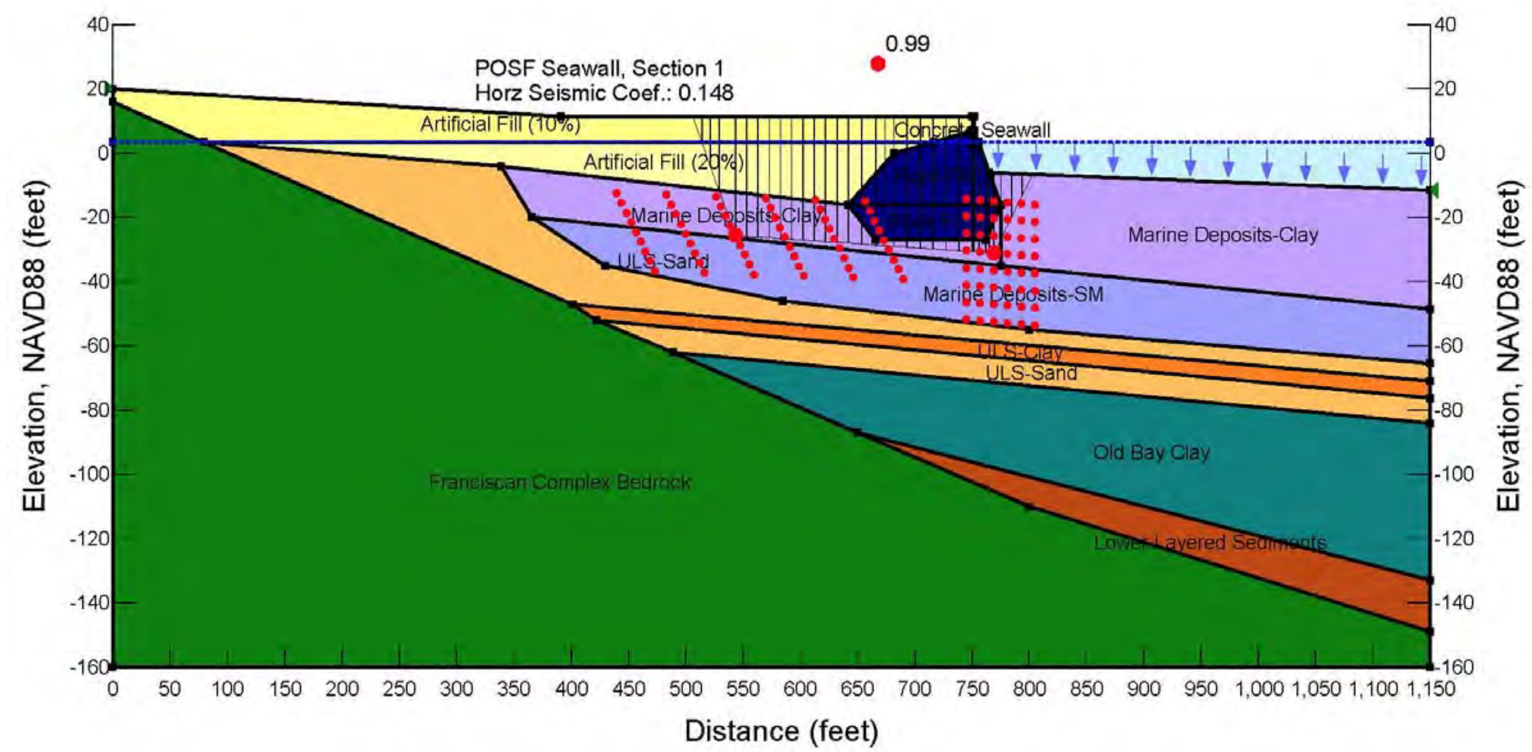


Figure 3-2812: Seismic Slope Stability at Seawall Section 1 – Peak Strength Conditions

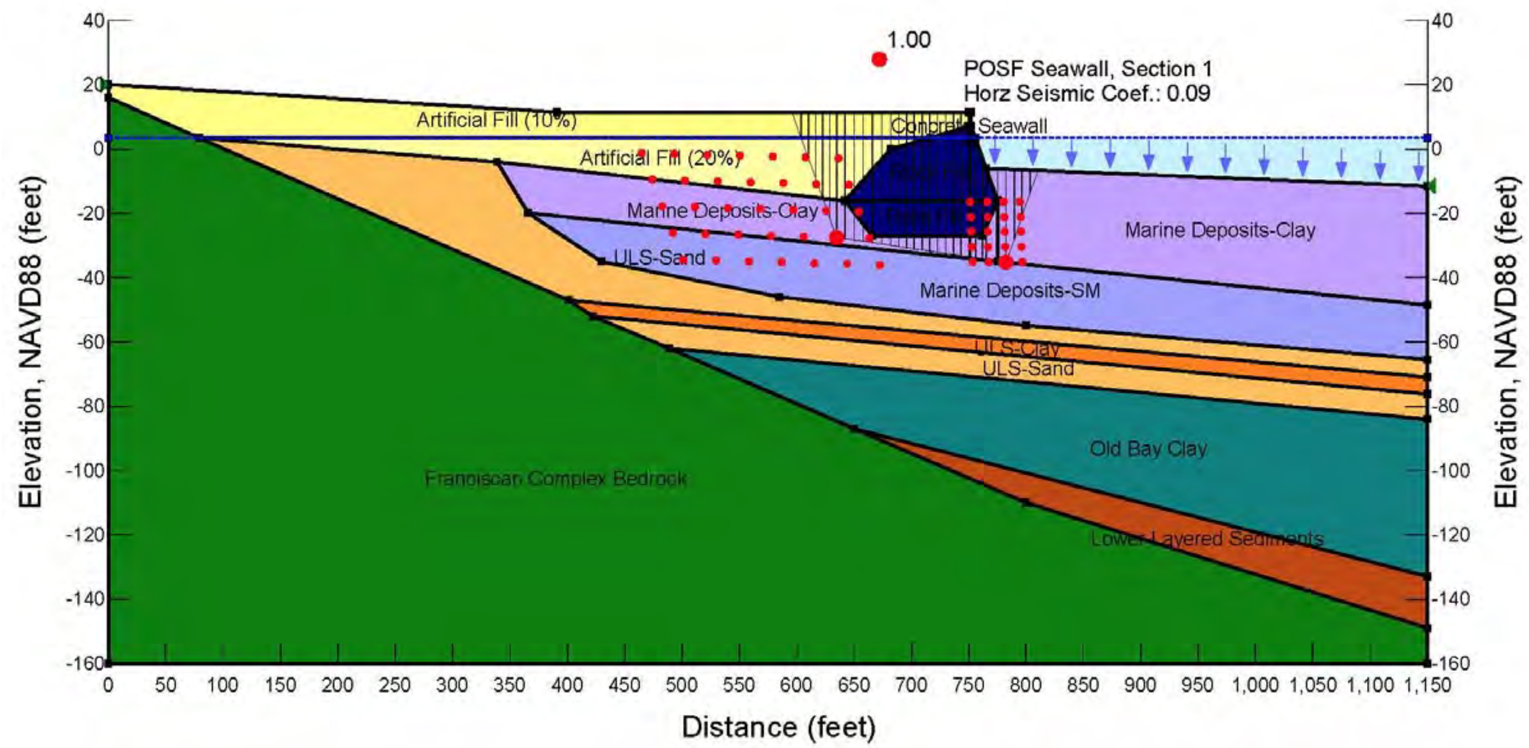


Figure 3-29 Seismic Slope Stability at Seawall Section 1 – Residual Strength Conditions

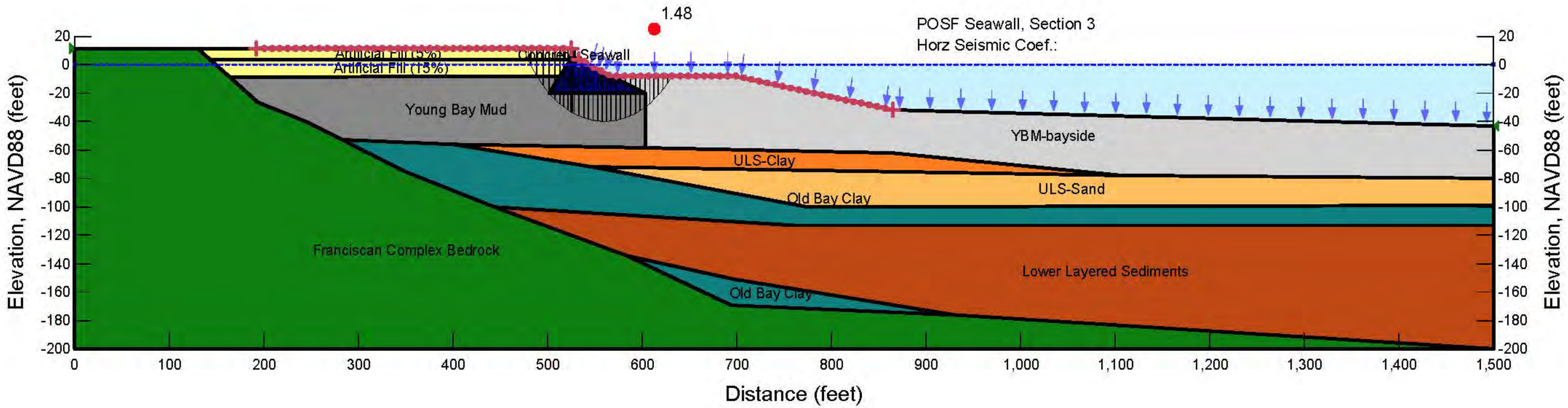


Figure 3-30: Static Slope Stability at Seawall Section 3

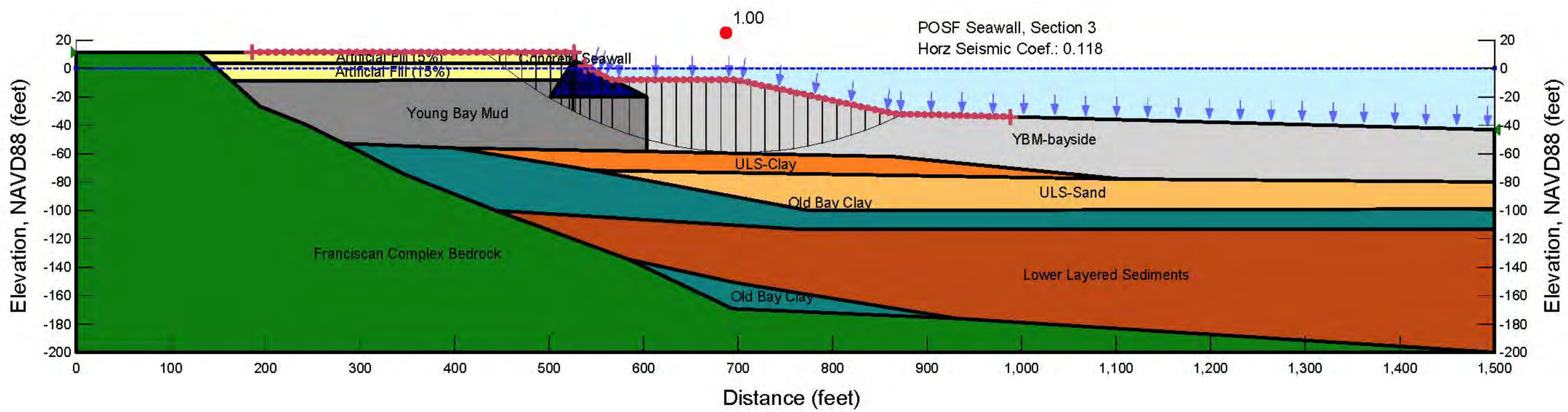


Figure 3-31: Seismic Slope Stability at Seawall Section 3 – Peak Strength Conditions

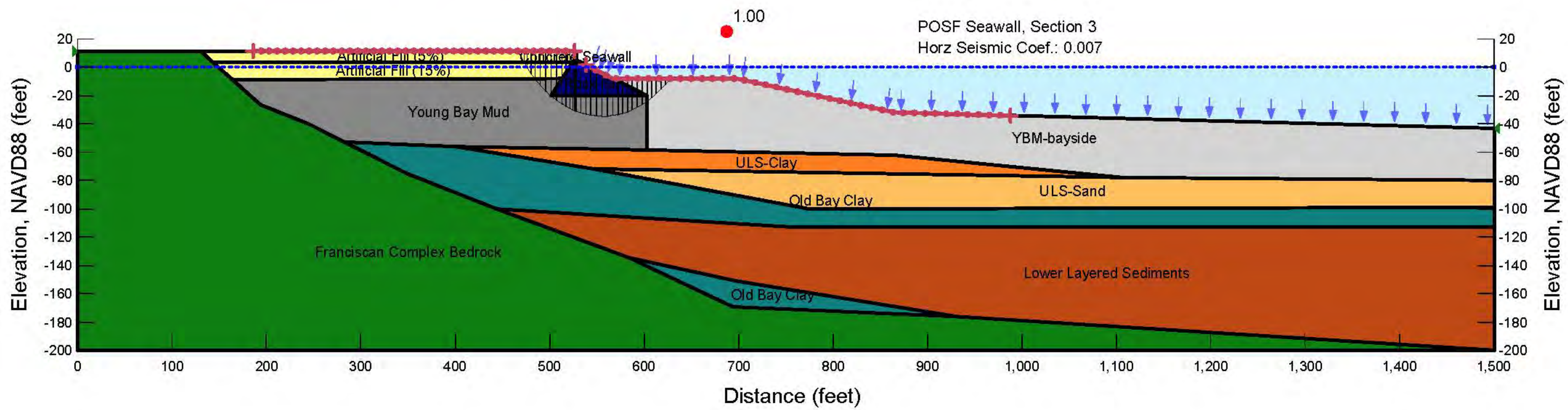


Figure 3-32: Seismic Slope Stability at Seawall Section 3 – Residual Strength Conditions

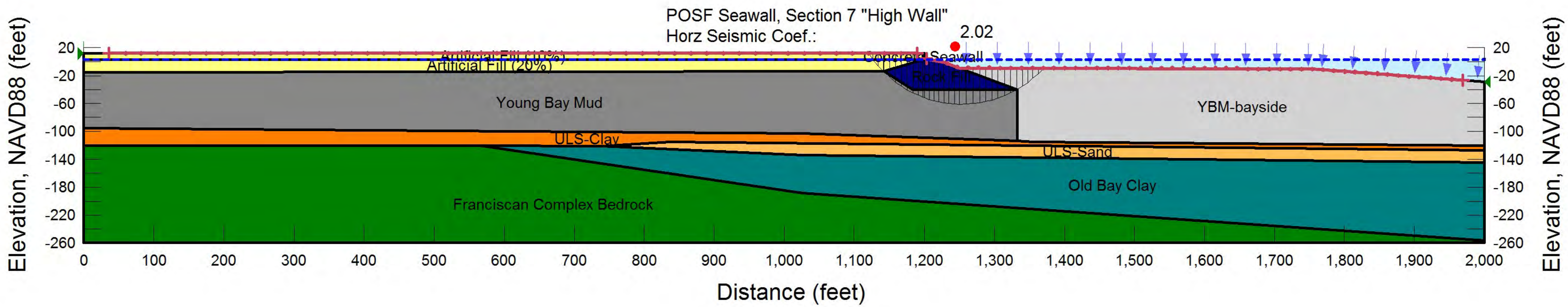


Figure 3-33: Static Slope Stability at Seawall Section 7 - High Wall

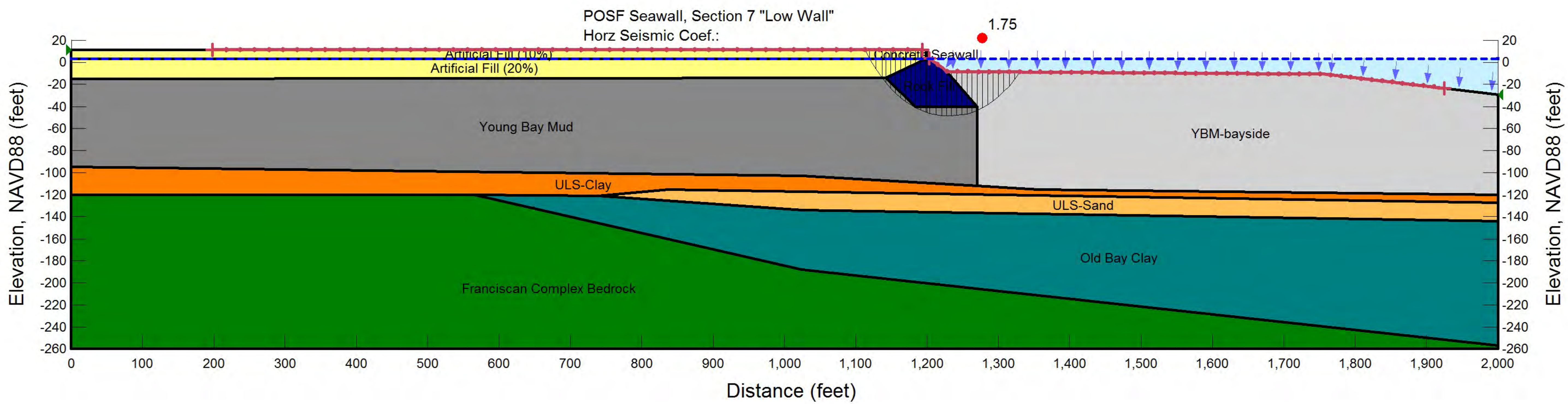


Figure 3-34: Static Slope Stability at Seawall Section 7 - Low Wall

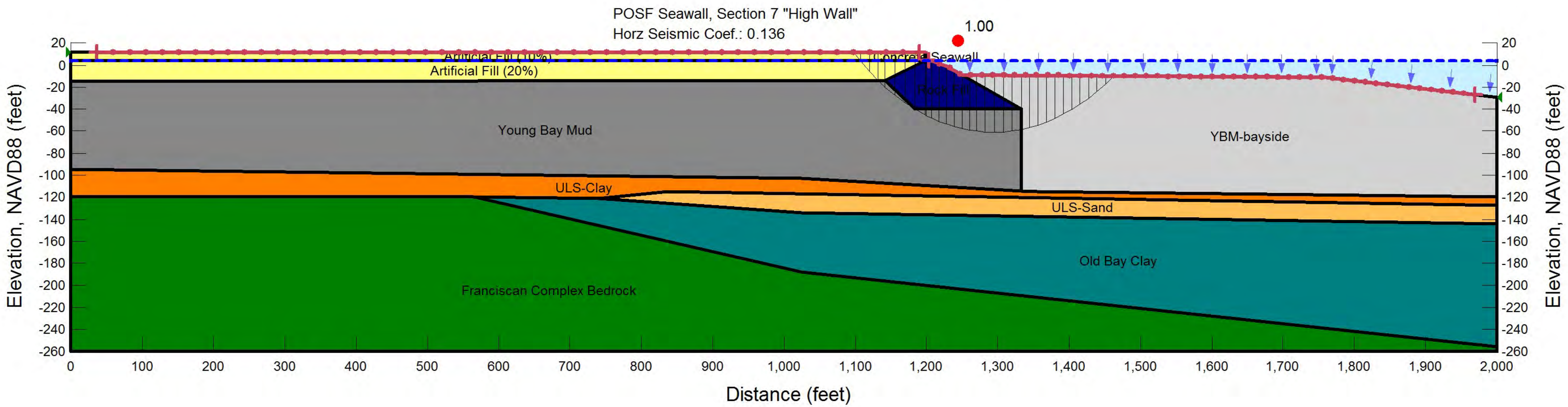


Figure 3-35: Seismic Slope Stability at Seawall Section 7 – High Wall – Peak Strength Conditions

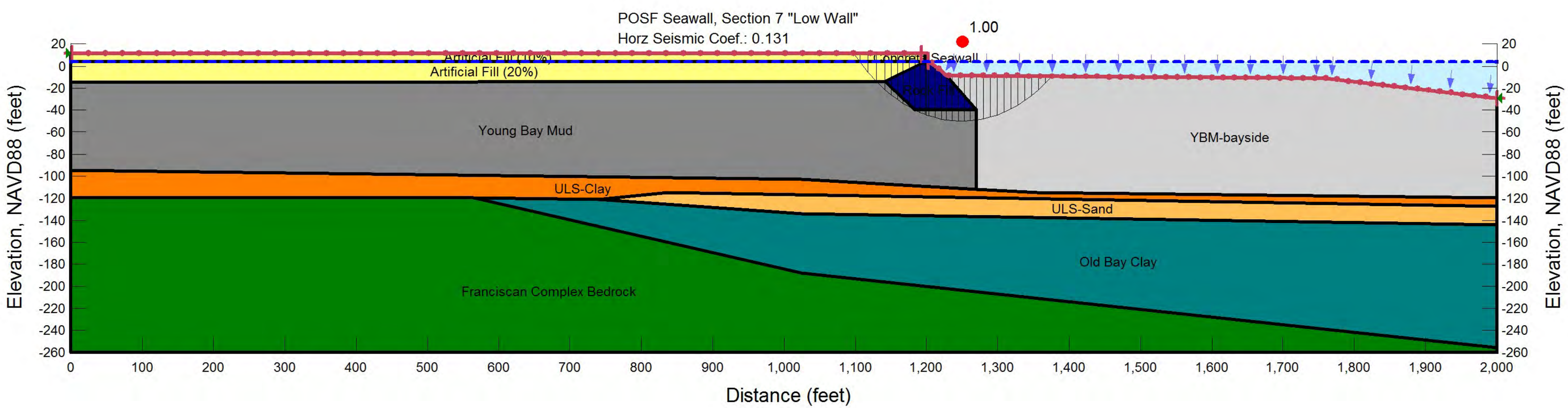


Figure 3-36: Seismic Slope Stability at Seawall Section 7 – Low Wall – Peak Strength Conditions

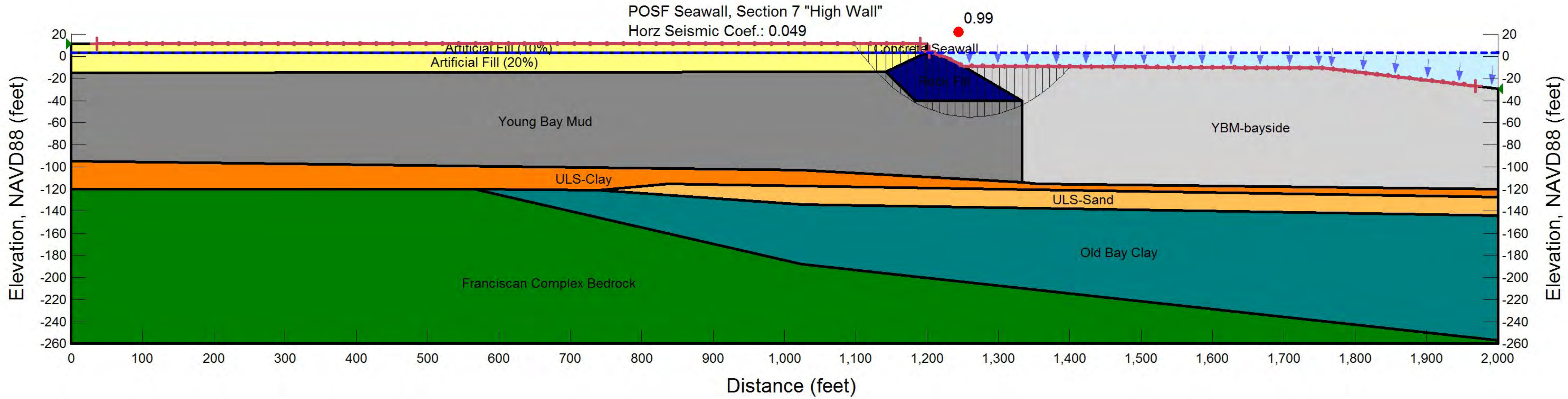


Figure 3-37: Seismic Slope Stability at Seawall Section 7 – High Wall – Residual Strength Conditions

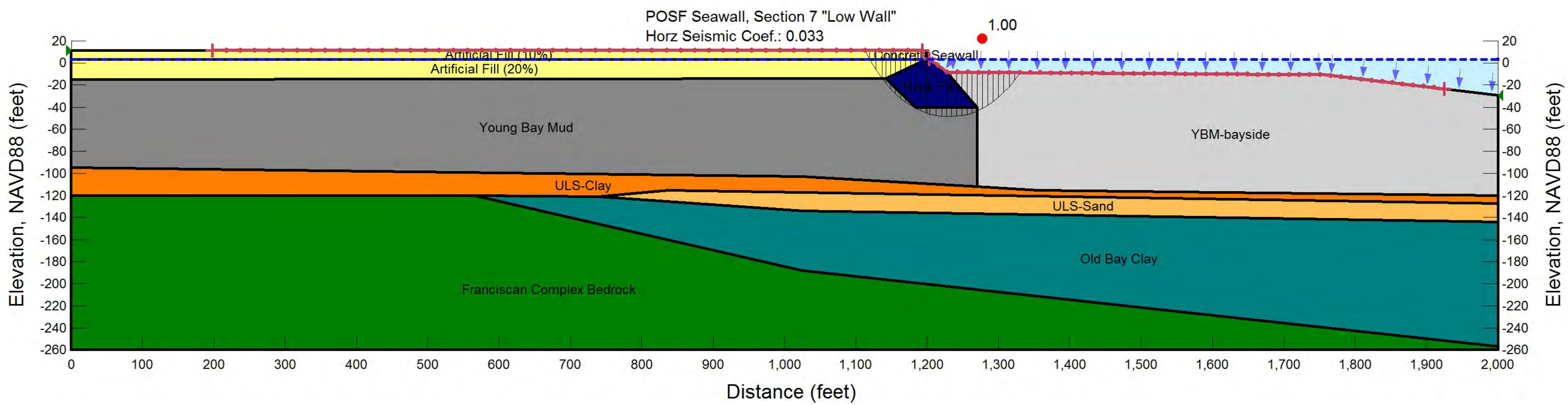


Figure 3-38: Seismic Slope Stability at Seawall Section 7 – Low Wall – Residual Strength Conditions

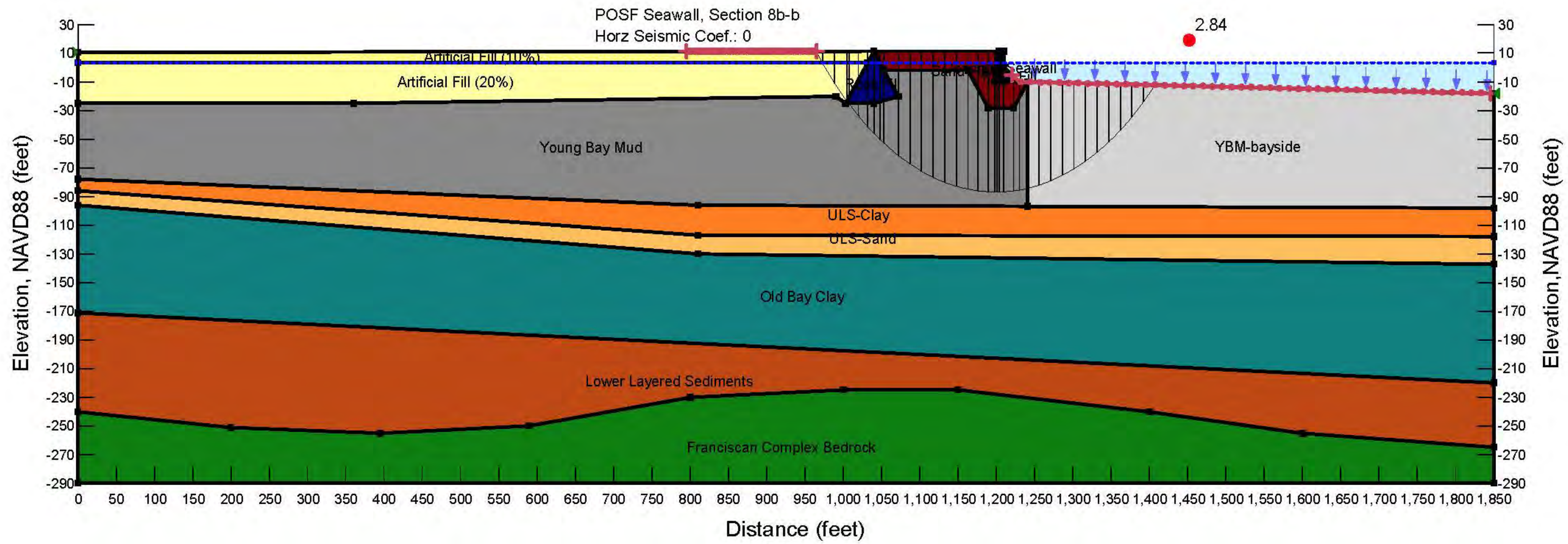


Figure 3-39: Static Slope Stability at Seawall Section 8b – Deep Failure Surface

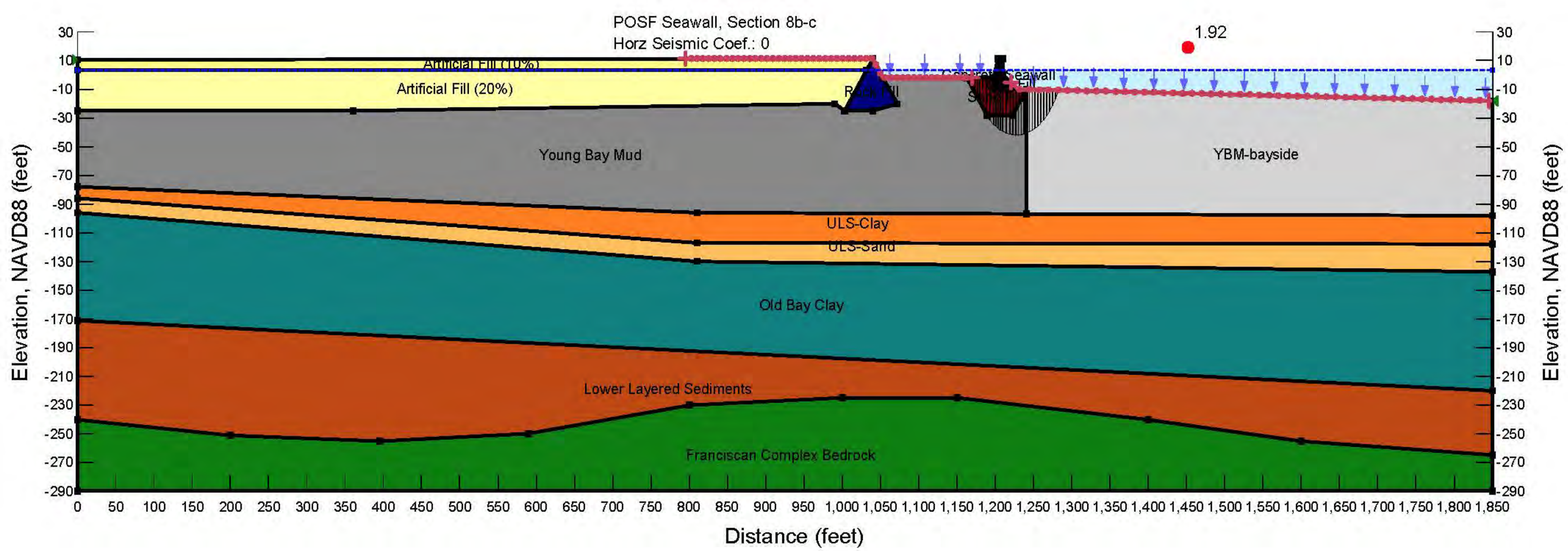


Figure 3-40: Static Slope Stability at Seawall Section 8b – Shallow Failure Surface

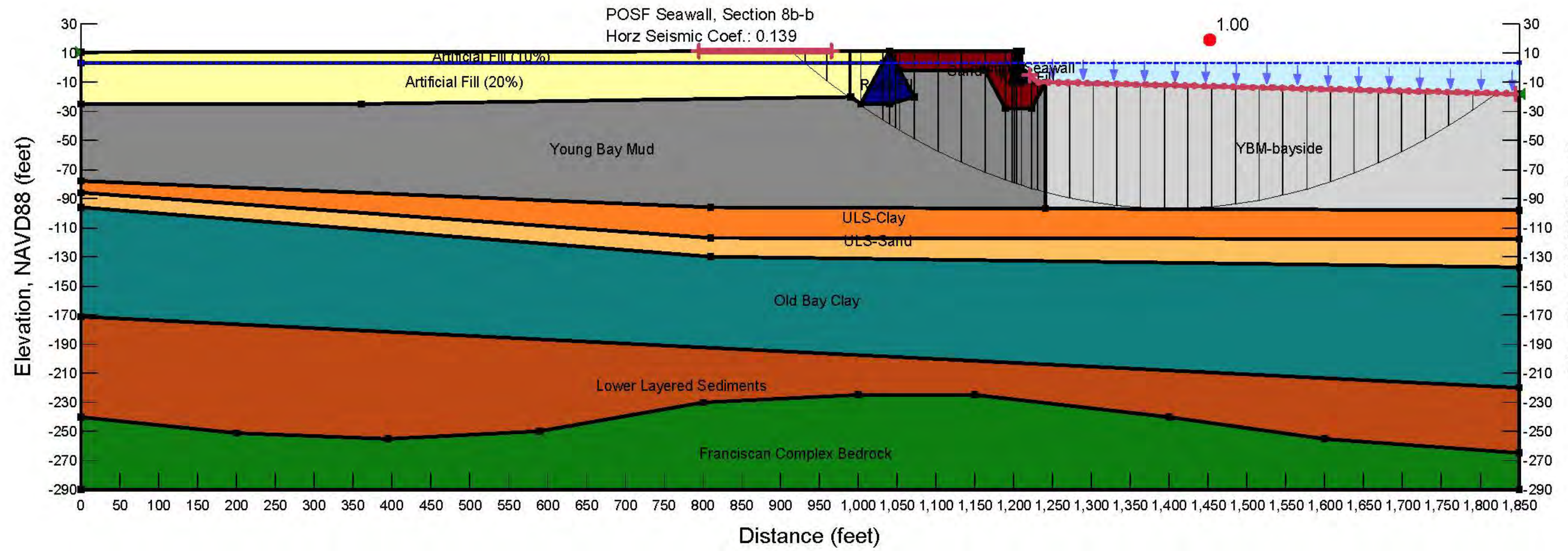


Figure 3-41: Seismic Slope Stability at Seawall Section 8b – Deep Failure Surface – Peak Strength Conditions

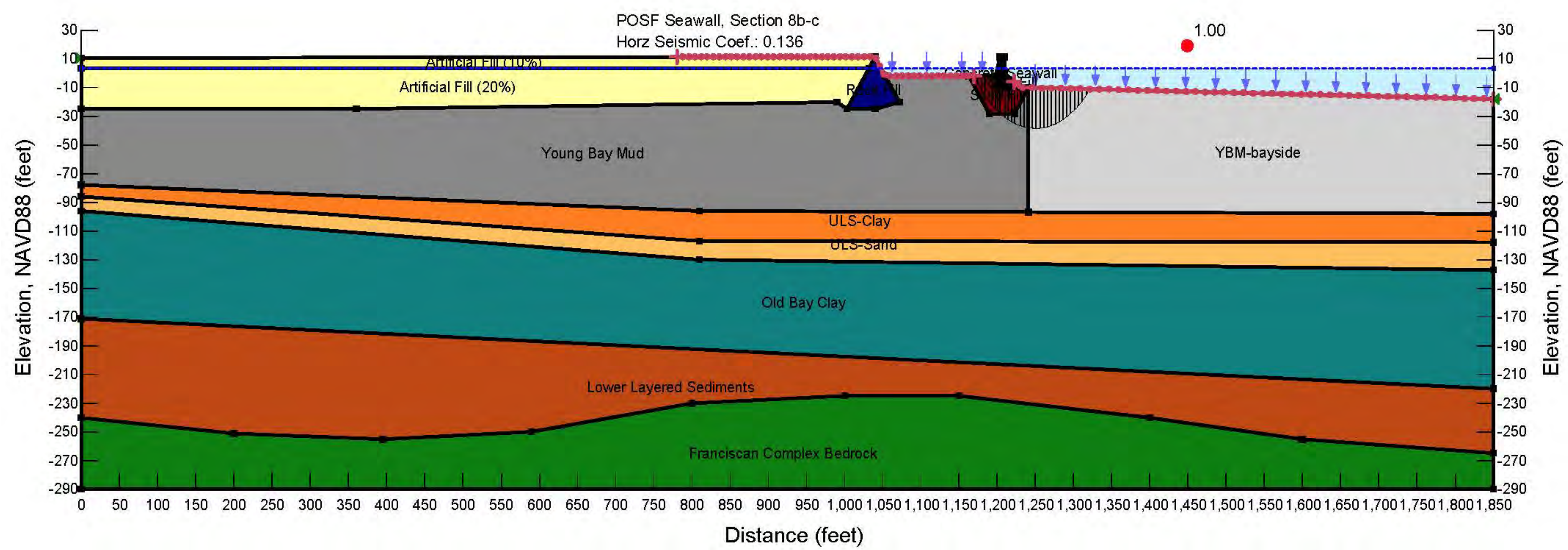


Figure 3-42: Seismic Slope Stability at Seawall Section 8b – Shallow Failure Surface – Peak Strength Conditions

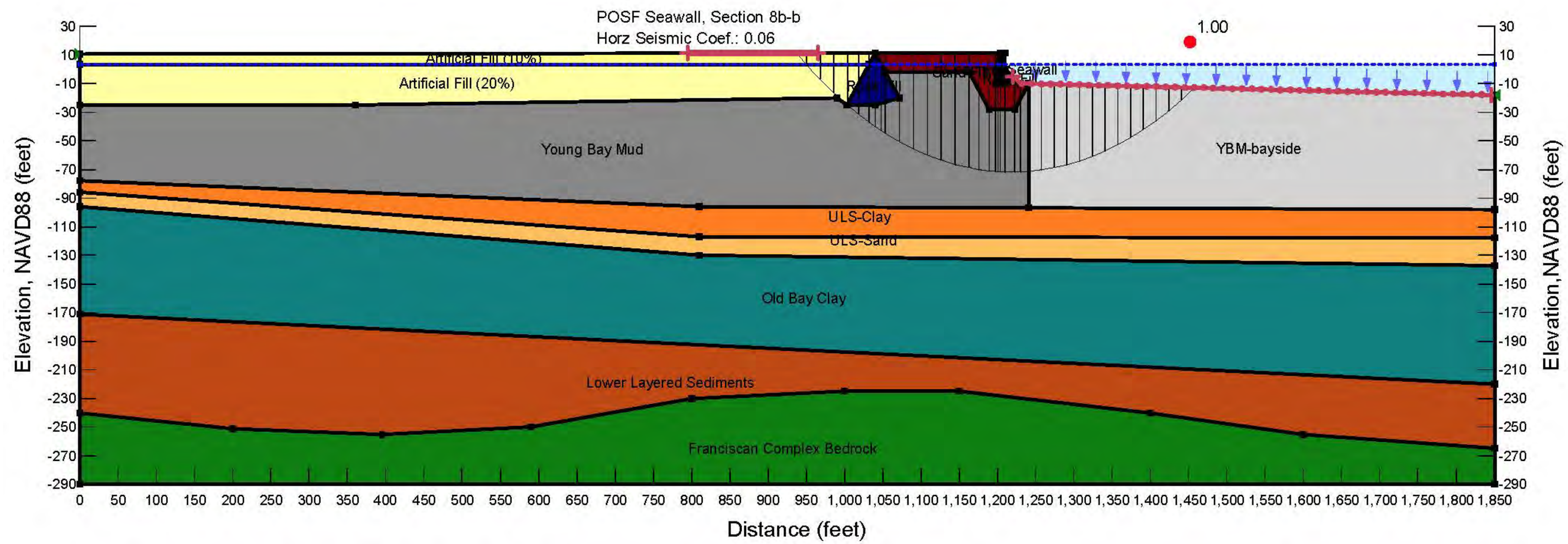


Figure 3-43: Seismic Slope Stability at Seawall Section 8b – Deep Failure Surface – Residual Strength Conditions

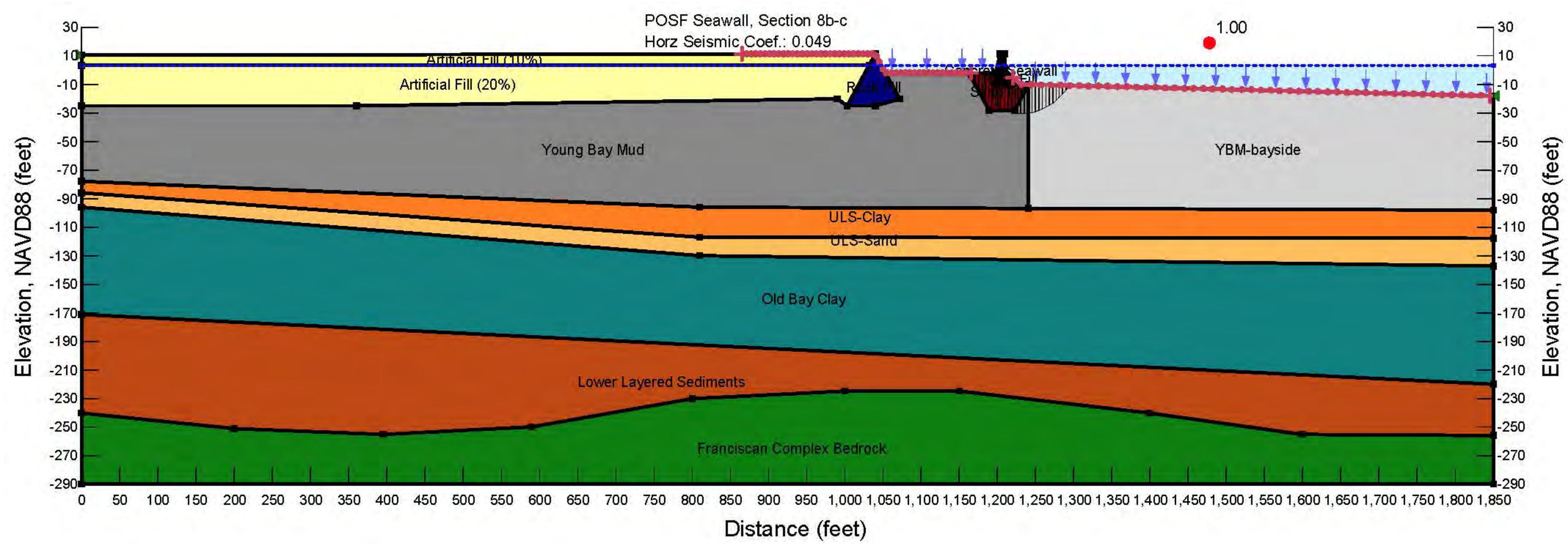


Figure 3-44: Seismic Slope Stability at Seawall Section 8b – Shallow Failure Surface – Residual Strength Conditions

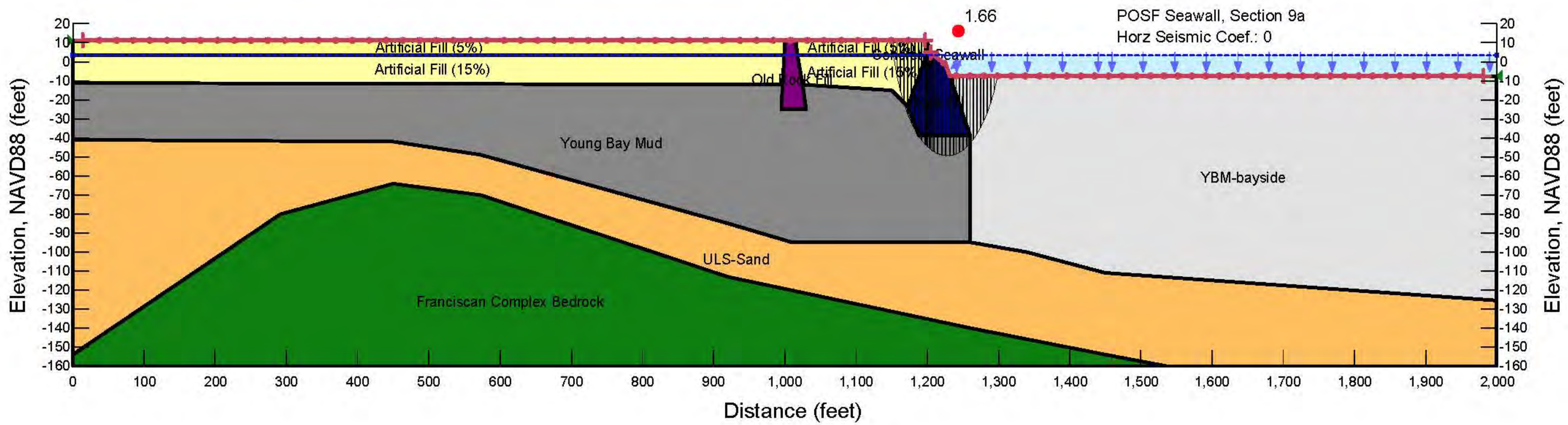


Figure 3-45: Static Slope Stability at Seawall Section 9a

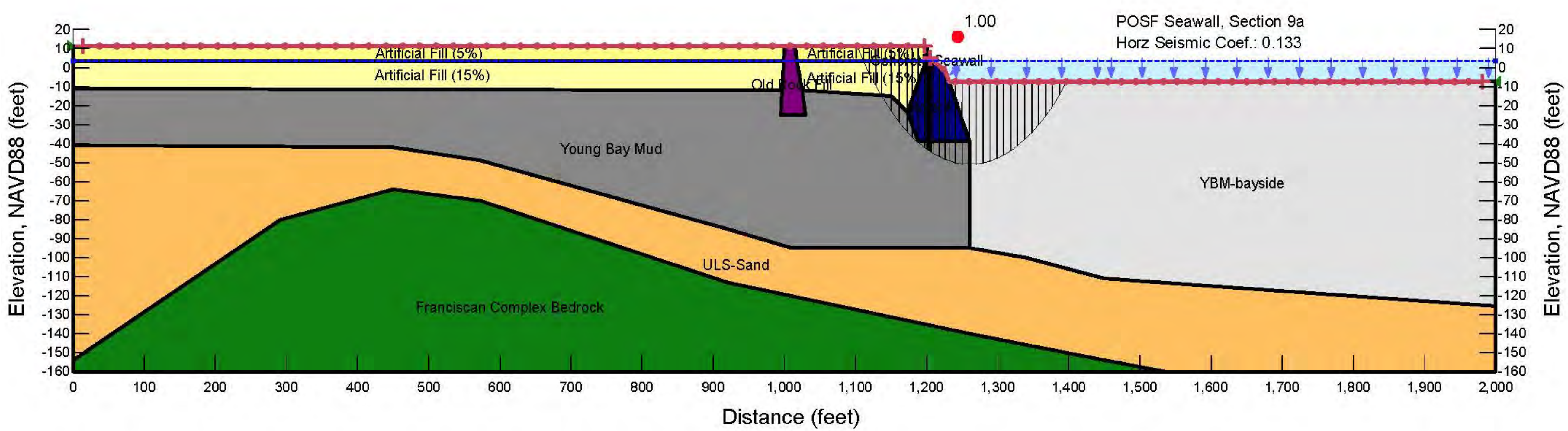


Figure 3-46: Seismic Slope Stability at Seawall Section 9a – Peak Strength Conditions

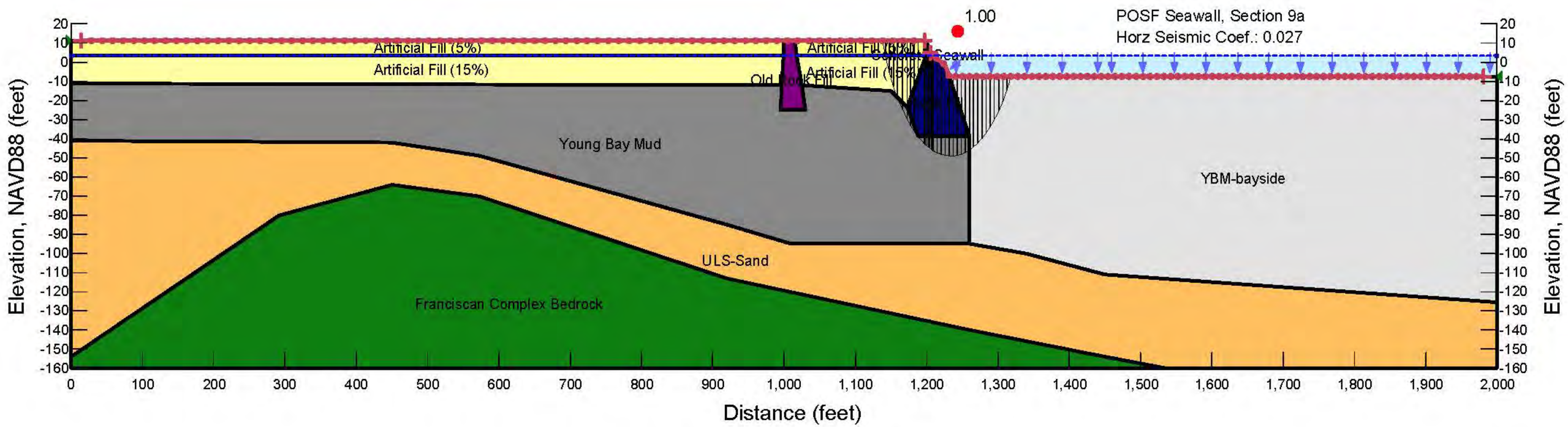


Figure 3-47: Seismic Slope Stability at Seawall Section 9a – Residual Strength Conditions

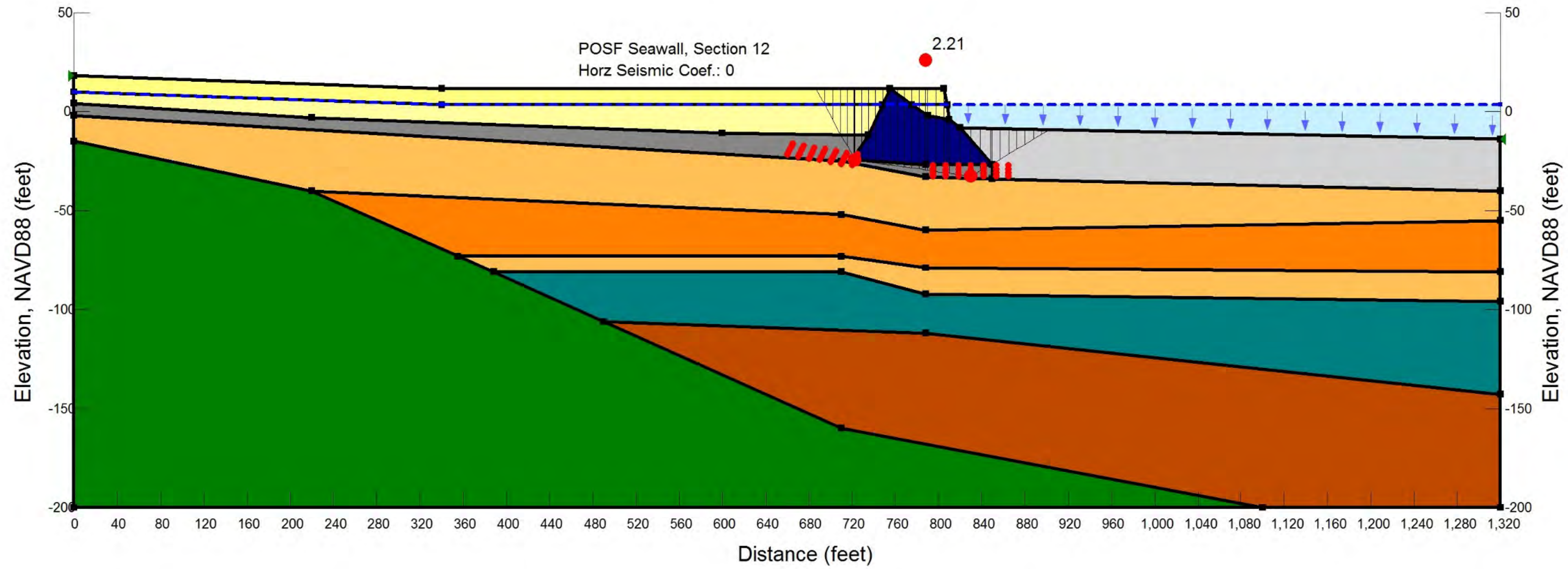


Figure 3-4813: Static Slope Stability at Seawall Section 12

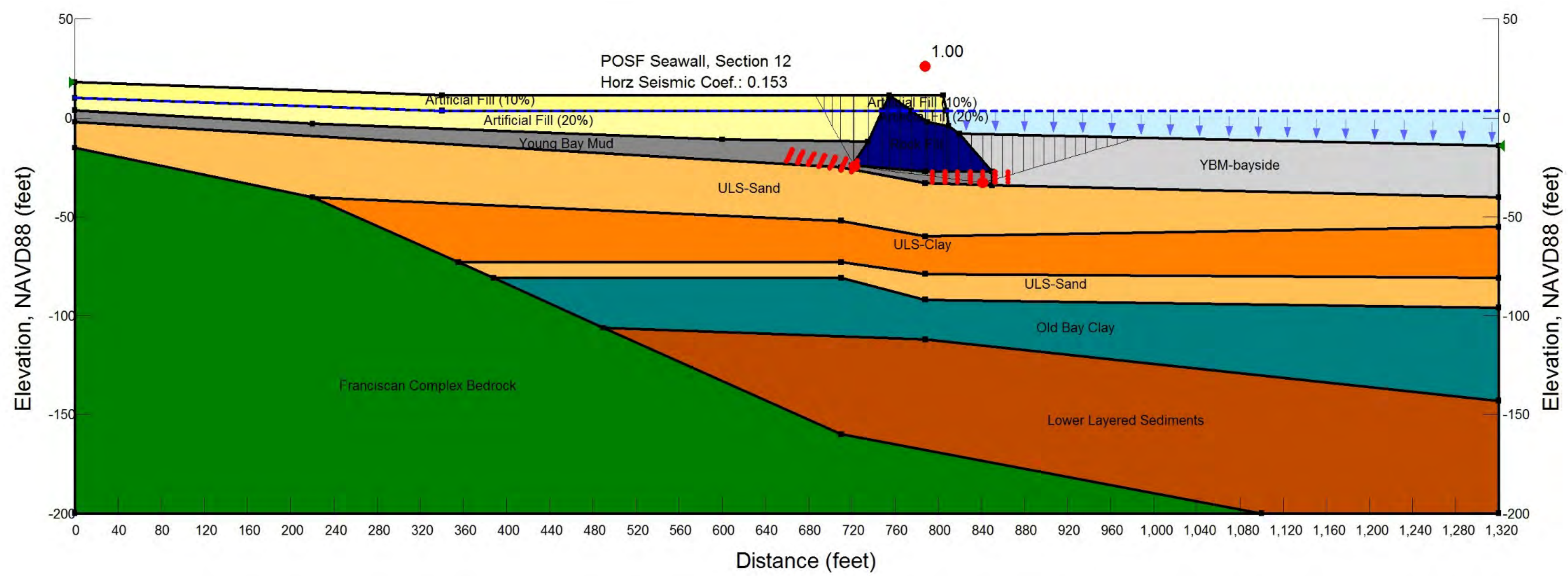


Figure 3-49: Seismic Slope Stability at Seawall Section 12 – Peak Strength Conditions

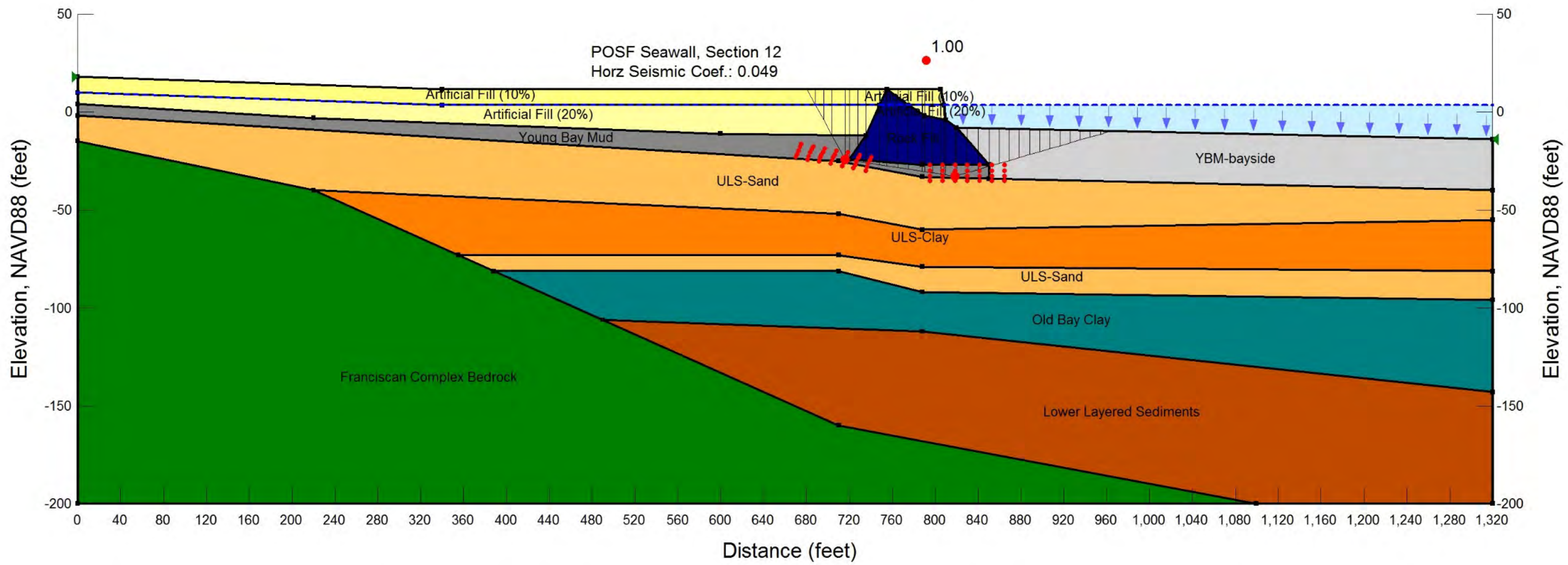


Figure 3-50: Seismic Slope Stability at Seawall Section 12 – Residual Strength Conditions

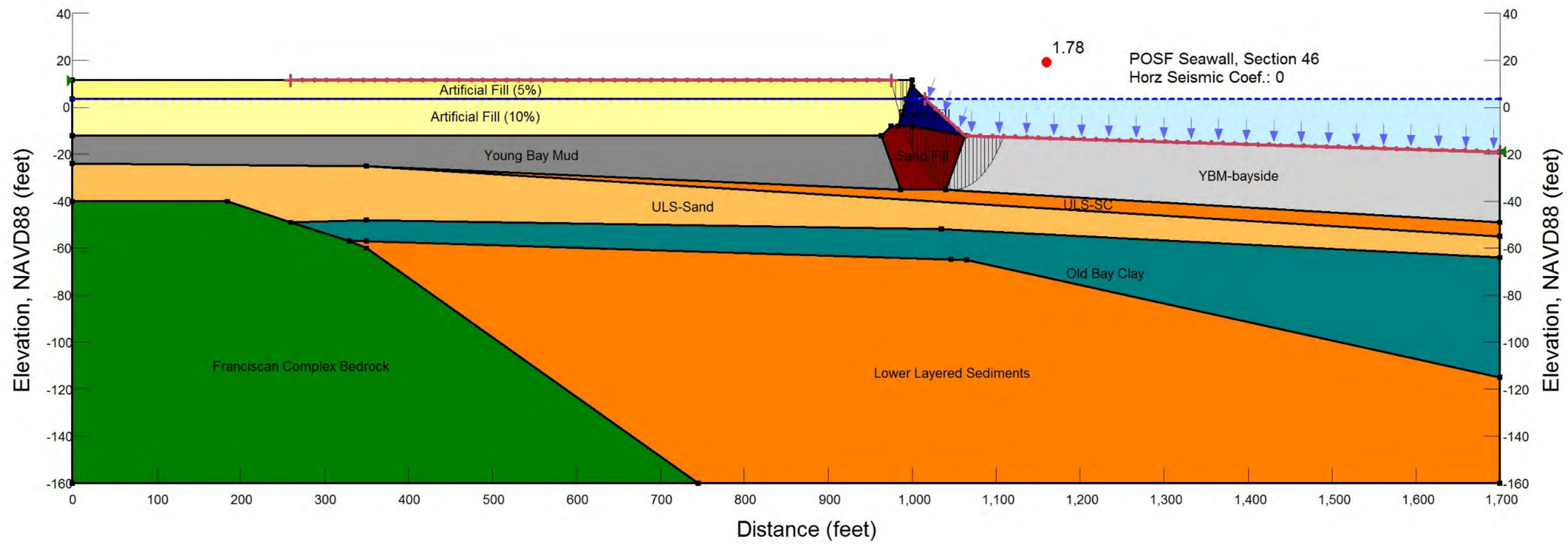


Figure 3-51: Static Slope Stability at Seawall Section 46

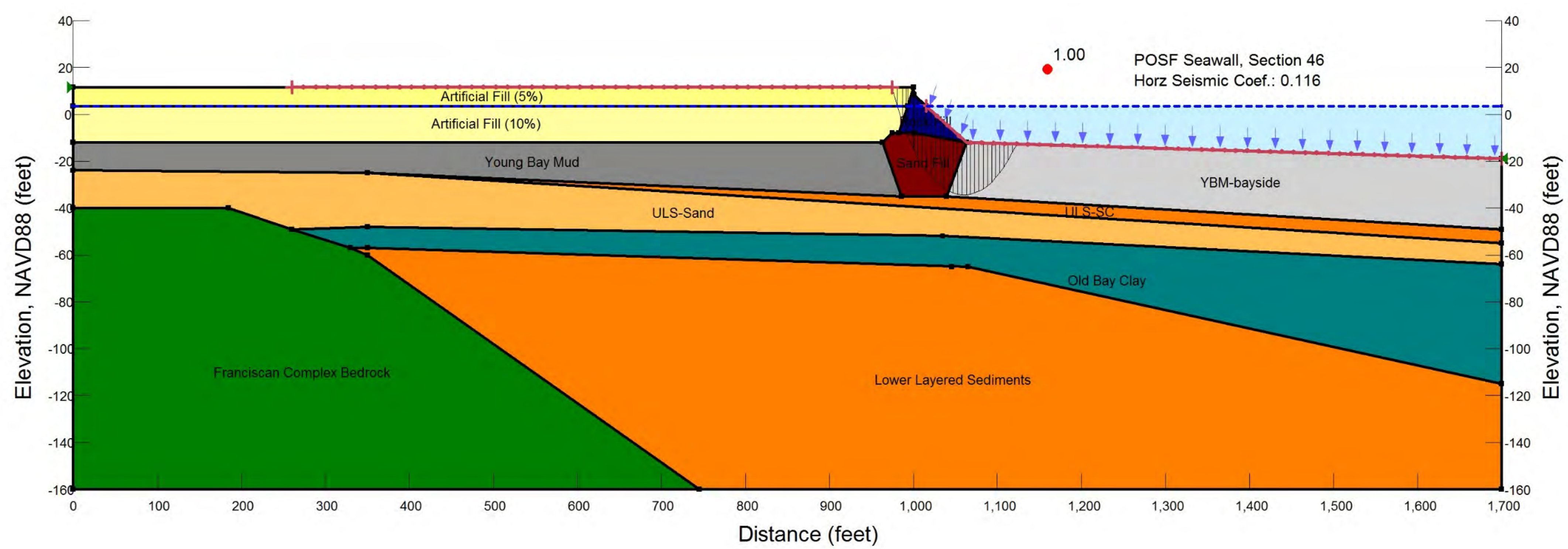


Figure 3-5214: Seismic Slope Stability at Seawall Section 46 – Peak Strength Conditions

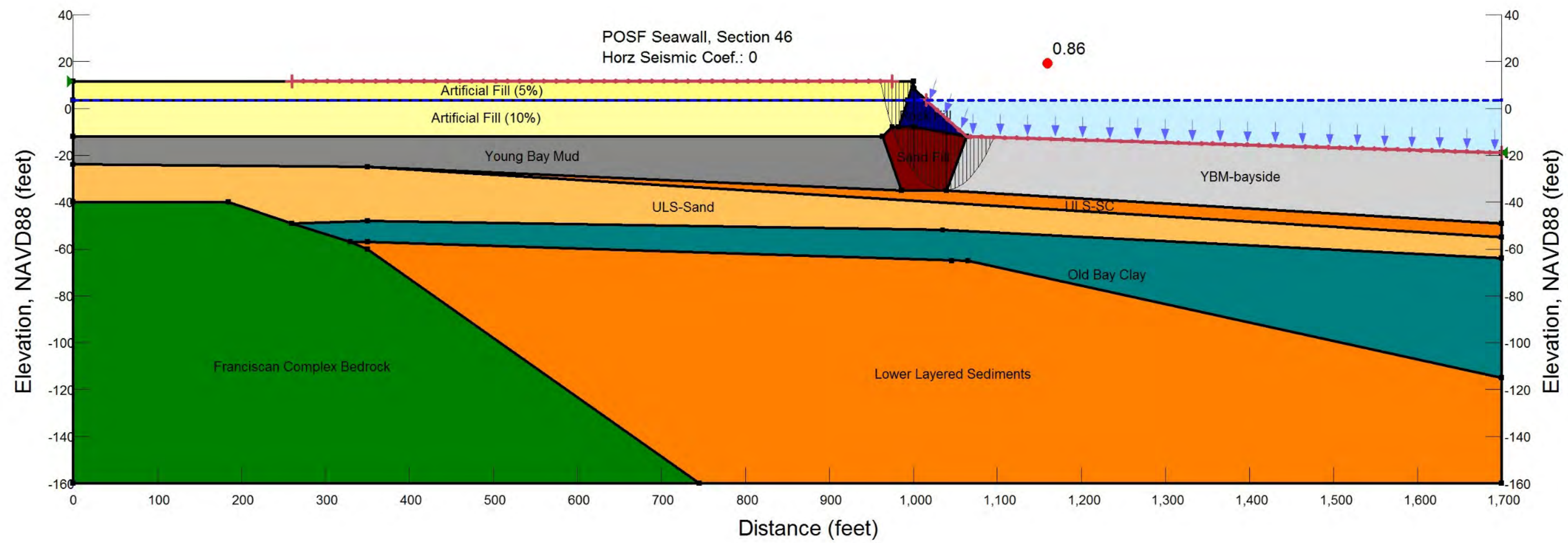


Figure 3-5315: Seismic Slope Stability at Seawall Section 46 – Residual Strength Conditions

3.5. Seismically-Induced Ground Deformation

General

Seismically-induced, free-field permanent ground deformations (PGD) were evaluated for both horizontal (PGD_h) and vertical (PGD_v) displacements. Our approach to estimating seismically-induced ground deformations are described in the following sections.

Horizontal Ground Deformation (Index PGD_h) at the Seawall

1. The results of liquefaction triggering evaluations for deposits of the loose to medium dense sandy fill behind the seawall were used to identify zones within the soil profile at each seawall section that are susceptible to liquefaction triggering and which demonstrate a potential for shear strain mobilization contributing to lateral spreading displacements during and immediately following seismic loading. In light of the low cyclic resistance of much of the sandy waterfront fill (as demonstrated by pervasive liquefaction during the low- to moderate-amplitude, short-duration, near-surface motions experienced during the 1989 Loma Prieta earthquake), *it was assumed for initial screening purposes that the fill is liquefiable when subjected to strong ground motions having an Average Return Period (ARP) of 75 years or greater*.
2. Slope stability analyses were performed for each seawall section using General Limit Equilibrium (GLE) procedures implemented in the software Slope-W, as addressed in *Section 3.6* of this report. Static, cyclic, and post-cyclic (residual) soil strengths were assigned to each of the soil layers in the GLE model and factors of safety against sliding computed for all three loading conditions. The soil strengths representative of cyclic loading conditions were used to compute the yield acceleration (k_y), at which the factor of safety against slope failure is 1.0. The yield acceleration was used in subsequent deformation analyses outlined as follows. The lateral force coefficients (k_h) used in pseudo-static GLE analyses for each seawall section were obtained from the trends of near-surface PGA versus Average Return Period (ARP) developed for the seawall vulnerability project (addressed in *Section 3.5*). The possible influence of pile foundations on sliding resistance was not included in the slope stability analyses, thus the computed values of k_y and subsequent ground displacements are for free-field conditions.
3. The soil deposits that most influence the stability of the seawall, as modeled by GLE, are the sandy fill and underlying young bay mud. Both soils are prone to degradation and strength loss during cyclic loading. The static and residual shear strengths of both soils were evaluated on the basis of standard-of-practice procedures for estimating the post-liquefaction, residual strength of sands, and the results of laboratory testing of young bay mud (cyclic triaxial, direct simple shear, and large-scale centrifuge testing). *As the soil strengths progressively decrease due to the effects of cyclic loading there is a reduced margin of safety against slope failure and a reduction in k_y. In light of the anticipated change in k_y during seismic loading a displacement-dependent trend in k_y was applied in the Newmark analyses. The yield acceleration was varied from the lower-strain, static k_y to the large-strain, residual value. The yield acceleration was reduced from the static value to the residual value in four steps over a lateral displacement of 18 inches.*
4. Seismically-induced horizontal deformations at the seawall using practice-oriented, Newmark-type sliding block analyses were estimated. The primary method of analysis involved the sliding block analyses implemented in the software SLAMMER (Jibson et al., 2013) for coupled, dynamic response and sliding along a well-defined slip plane. SLAMMER allows for the use of displacement-dependent k_y trends. The coupled analysis was applied to all of the seawall sections and the median trend in displacement used for application.

The displacement results computed using SLAMMER with constant values of k_y were compared against three practice-oriented procedures; (i) NCHRP 611 (Anderson et al., 2008), (ii) Bray and Travasarou (2007), and (iii) Rathje and Saygili (2009). The results of the four procedures provided complementary trends. After confirming these trends, the coupled analysis in SLAMMER was used with displacement-dependent k_y to compute the recommended trends of Index PGD_h versus ARP.

5. Using the results of the deformation analyses, we developed trends for Index PGD_h versus ARP for the selected seawall sections. These trends are provided in ***Figures 3-55 to 3-57***.

The slope displacement trends provided in ***Figures 3-55 to 3-57*** reflect both the cyclic demand (i.e. near-surface ground motions) and cyclic resistance as reflected in k_y. The computed displacements at the ARPs of primary interest (i.e., ≥ 475 years) are sensitive to the k_y computed using residual soil strengths. This is evident for Section 46 where the post-cyclic stability is very low and the possibility of flow failure has been indicated in this screening-level analysis.

With respect to the cyclic loading, it is important to note the relative trends in ground motion intensity with ARP for each of the three microzones defined for the investigation. As provided in *Section 3.5* (***Figure 3-19***), it is apparent that PGA reaches a plateau, or "saturates", at ARP greater than roughly 750 years. This is due to the cyclic behavior of the deep deposits of young bay mud and maximum amplitude of higher frequency motions that can be transmitted through the thick deposits of soft clay. This aspect of dynamic soil response has a significant influence on the trends of slope displacement computed using simple models based on PGA, such as the coupled sliding block model in SLAMMER and the three other methods previously listed.

The results of PSHA deaggregation indicate that the predominant seismic source contributing to PGA along the entire seawall alignment considered in this investigation is an approximately M 8.0 San Andreas fault event having a source-to-site distance of roughly 13 km. This remains constant for all ARPs greater than 750 years – the portion of the PGA hazard curve that saturates for Zone C2. For this reason, the magnitude and ground motion intensity (PGA) for all ARPs greater than 750 years remain roughly the same, and it follows that the computed slope displacement, which is directly linked to PGA, also remains essentially constant. This trend in PGD_h with ARP is not considered representative at longer ARPs due to the likely influence of strong motion duration, which may increase to well above mean values at the longer ARPs. For this reason, subjective scaling factors were applied to the PGD_h values for seawall sections in Zone C2 for ARPs of 975 years and 2475 years. Scale factors of 1.07 and 1.5 were applied for the ARPs of 975 years and 2475 years, respectively, based in part on consideration of standard deviation in the trend of significant duration with magnitude, and standard deviation in trends of slope displacement using simplified Newmark-based procedures. This adjustment for saturation and "duration effects" was applied to the PGD_h trends in ***Figure-3-57***.

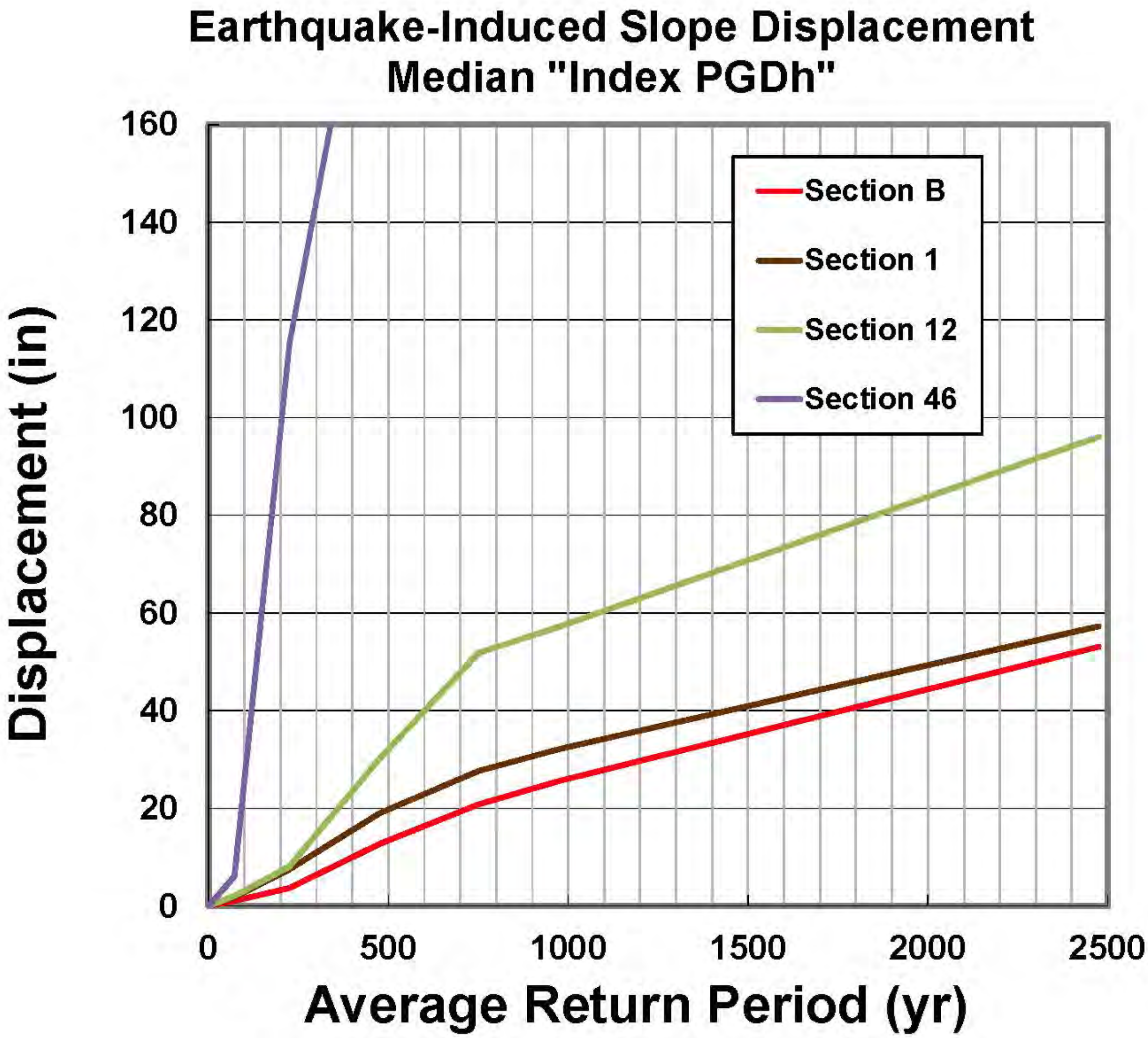


Figure 3-54: Trend of Index Lateral Ground Deformation with Average Return Period at Four Seawall Sections in Microzone A1/A2.

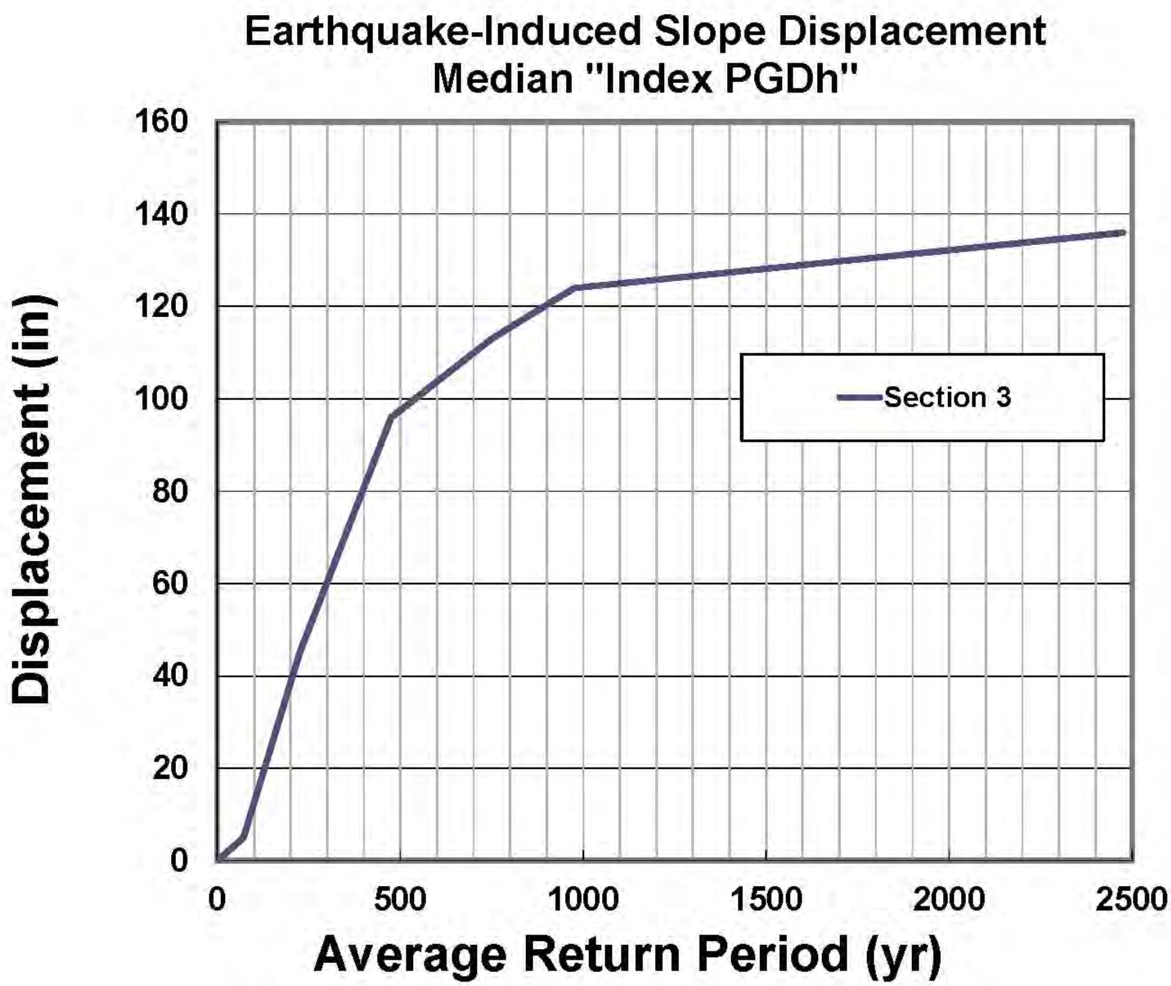


Figure 3-55: Trend of Index Lateral Ground Deformation with Average Return Period at One Seawall Section in Microzone B1/B2.

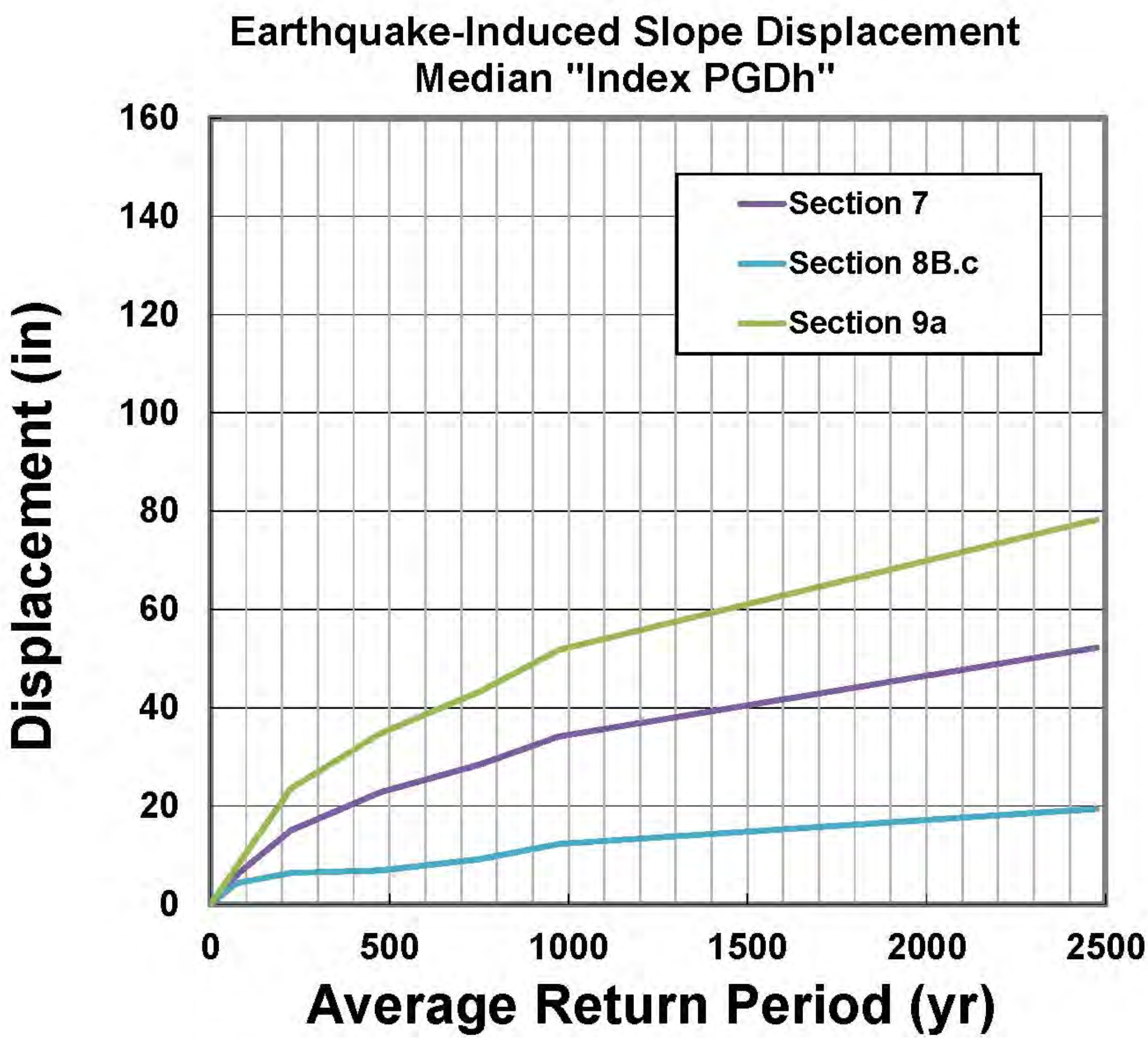


Figure 3-56: Trend of Index Lateral Ground Deformation with Average Return Period at Three Seawall Sections in Microzone C2.

Variation of Horizontal Ground Deformation with Distance behind the Seawall

The GHD/GTC team was tasked with providing estimates for the variation of free-field lateral ground deformation behind the seawall in response to waterfront slope deformation. This was pursued with the goals of facilitating estimates of lateral ground deformation at locations behind the seawall with surface and buried structures, utilities, and surface transportation routes, as well as possibly refining the estimated "Zone of Influence" associated with earthquake-induced deformation of the seawall. Estimating the pattern of ground deformation behind the seawall is complicated by the presence of soils that are prone to liquefaction and cyclic degradation. Potential methods for estimating the gradual decrease in lateral deformation with distance behind the seawall include; (i) empirical procedures developed from case histories for lateral spreading (e.g., Youd et al., 2002; Zhang et al., 2004), (ii) field observations and empirical trends from ports and harbors around the world, and where thorough geotechnical site characterization permits; (iii) two-dimensional numerical simulation of seismic performance using well-calibrated models. The trends in PGD_h with distance from the waterfront were estimated in this investigation using methods (i) and (ii).

Field case studies, predominantly from ports in Japan, provide useful trends for the variation of PGD_h with distance from waterfront gravity retaining walls (concrete caissons). These cases provide ground deformations for a relatively simple boundary condition of a rigid, yielding wall. While this is not, strictly speaking, the mode of deformation over most of the Northern Waterfront Seawall alignment, the general trends provide guidance on ground deformations adjacent to the seawall and they have been used to help bracket the likely range of lateral displacements. The trends from Japan have been supplemented with data for three pertinent case studies investigated by members of the GHD/GTC team, two of which involve the performance of local bay front sites (St. Francis Yacht Club and Treasure Island) during the 1989 Loma Prieta earthquake. The trend of ground deformation for liquefaction sites is provided in ***Figure 3-58***. The plot provides the ratio of lateral ground displacement at a distance L ($PGD_{h(L)}$) from the seawall to the maximum lateral displacement ($PGD_{h\text{-}max}$), which occurs at the seawall, as a function of the normalized distance (L/H) from the seawall. The distance measure, L, is the lateral distance and H is the height of the rock dike at the location of interest. The lateral distance is measured from a point at the ground surface above the landward crest of the rock dike, this yields an L/H ratio of roughly 0.6 at the mid-height of the rock dike slope and is consistent with the solid black trend line labeled "Port Island (average)". Note that the lateral ground displacement remains constant and equal to $PGD_{h\text{-}max}$ from the crest of the rock dike to a distance of 0.6 times H, then it begins to diminish with increasing distance.

The empirical trend developed for field data involving gravity walls has been supplemented with three pertinent data points involving; sloping rockfill with liquefiable sand underlain by young bay mud (Treasure Island, TI), a bayfront site at the St. Francis Yacht Club, and a sheetpile bulkhead with liquefied backfill (1993 Guam Earthquake). The average trend from Japan is supported by the additional three data points. For the sake of comparison, the normalized ground displacement trends provided using the empirical, free-field lateral spread procedures of Youd et al. (2002) and Zhang et al. (2004) are plotted. It is useful to note that the trends from the free-field, lateral spread procedures greatly over-estimate the lateral ground deformations adjacent to waterfront development (i.e., non-free field conditions) at all distances from the slope or free-face.

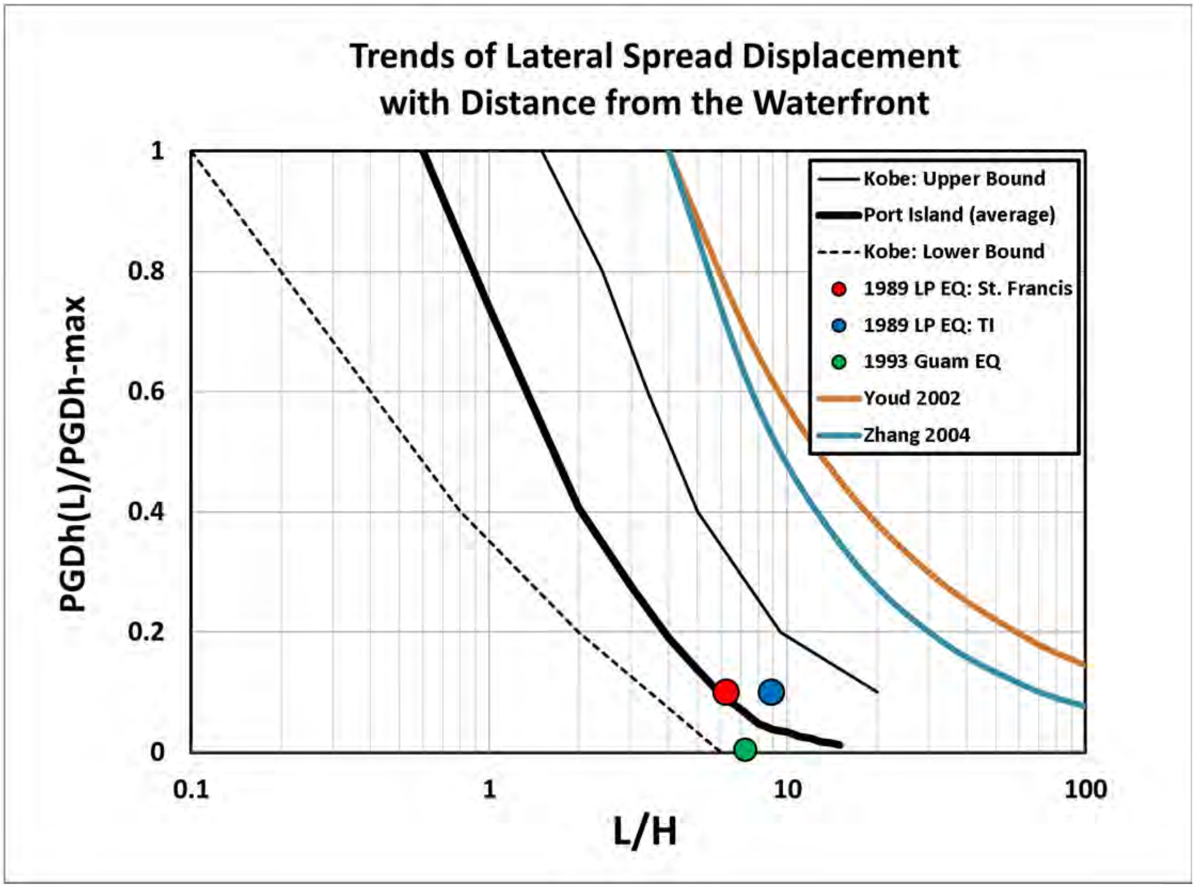


Figure 3-57: Empirical Trends in Lateral Ground Deformation with Distance from the Waterfront.

Using the procedure outlined above and the results from the horizontal ground deformation analysis, we prepared maps showing the zone of influence and amount of permanent horizontal ground deformation for four seismic hazard levels – the median estimate of a M8.0 San Andreas seismic event, 475-year return period, 975-year return period, and Maximum Considered Earthquake. These maps are provided in ***Figures 3-59 through 3-66*** at the end of this section of the report.

Ground Surface Settlement (Index PGD_v) Associated with Deviatoric Strains

Ground surface settlements behind the seawall will develop in response to the following three conditions; (i) deviatoric shear strains in the fill that occur in response to the lateral movement of the seawall and foundation soils, (ii) settlement associated with possible rotational or combined (rotational and translational) modes of failure of the seawall, and (iii) volumetric strain associated with post-liquefaction, re-consolidation of sandy fill. The total ground surface settlement is often approximated as the sum of the three components evaluated independently. This simplified, uncoupled method of analysis provides general, yet adequate, estimates of ground surface settlement for the sake of screening analyses. This subsection primarily addresses the settlements attributed to condition (i), with a lesser contribution of (ii).

As the seawall moves in response to seismic loading, the volume of soil that displaces laterally should be balanced by an equal volume of soil that moves vertically (i.e., settles) if undrained, constant volume conditions are assumed. Alternatively stated, the areas of the vertical and horizontal displacement profiles should be approximately equal. The results of GLE slope stability analyses highlight potential slip surfaces that are formed at the base of the rock dike in contact with underlying young bay mud. The slip plane can be approximated with wedge, or block, geometry, thus translational sliding may be the predominant mode of movement with some smaller amount of rotational movement. For the sake of screening ground deformation hazards in this investigation, the extent and pattern of deviatoric settlements behind the seawall have been directly related to the peak lateral ground displacement (PGD_{h-max}) occurring at the crest of the rock dike.

The shape of the settlement profile has been developed using several independent lines of empirical evidence; (i) the trend of PGD_v/PGD_{h-max} from field case histories in Japan for gravity retaining walls and liquefied backfill, (ii) trends of PGD_v/PGD_{h-max} from field case studies involving static ground deformations behind braced excavations in fine-grained soils (undrained loading conditions), and (iii) considerations based on constant volume deformations of the soils behind the seawall. Several approximations have been made in order to establish the boundary conditions adjacent to the rock dike. These include:

- The primary mode of rock dike movement is translational; however, a minor component of rotation is assumed such that the ratio PGD_v/PGD_{h-max} at the crest of the rock dike is 0.20.
- Soil loading is globally undrained (constant volume) and therefore the volume associated with settlement is approximately equal to the volume associated with lateral movement.
- Settlement adjacent to the crest of the rock dike is a minimum (0.20 x PGD_{h-max}) then initially increases with distance from the dike as the thickness of the sand fill increases.
- The thickness of the sand fill is routinely between 20 ft and 25 ft at the seawall sections evaluated. The deviatoric strains are primarily associated with soil movement in and above the liquefied sand, which has a very low stiffness. Settlement associated with deviatoric strains in the underlying soil deposits (young bay mud and Upper Layered Sediments) is likely to be a small component of the overall settlement for rock dike sections founded on the young bay mud with no embedment.
- The maximum settlement occurs at L/H = 1.0, a distance from the landward crest of the rock dike equal to the height of the rock dike. For L/H greater than 1.0 the settlements begin to decrease, trending to 0 at L/H of 10.

The proposed trend in settlement with distance from the rock dike and seawall is provided in ***Figure 3-58***. The settlement profile is plotted as PGD_v/PGD_{h-max} versus L/H, where “Distance from the Waterfront” is L, and the “Height of the Rock Dike” is H. The distance, L, is scaled from the landward crest of the rock dike, consistent with the procedure for estimating PGD_h. The settlement profile initially increases with distance from the rock dike to an L/H ratio of 1.0, then decreases with a significant zone of influence extending roughly 4.5 to 5.5 times the height of the rock dike. The settlement trends for field data from Japanese case studies (Port Island, 1995 Kobe Earthquake) and for static ground deformations adjacent to braced excavations are provided for comparison in ***Figure 3-58***. The significant settlement indicated at greater distances (L/H ≥ 4.0) for field case studies at the Port of Kobe is attributed to volumetric strain in the deep fill of loose sand. Post-liquefaction volumetric strain at these sites contributed to PDG_v/PGD_{h-max} ratios of roughly 0.1 at greater distances from the quay walls. This influence is evident in ***Figure 3-58***. Ground settlements due to volumetric strain in liquefied fill have been evaluated separately in this investigation, and therefore the proposed settlement profile in ***Figure 3-58*** is significantly below the Port Island data at L/H greater than 3.5 to 4.0, and is much more consistent with field data for construction-induced ground settlement adjacent to braced excavations in weak soils.

The following simple procedure has been developed for screening applications at the seawall sections.

1. The maximum lateral deformation (PGD_{h-max}) at the crest of the seawall is first estimated using the PGD_h trends previously provided in ***Figures 3-55 to 3-57***.

2. The settlement profile behind the seawall is estimated using the trend of $PGD_v/PGD_{h\text{-}max}$ versus L/H provided in ***Figure 3-58***.

Using the procedure outlined above and the results from the horizontal ground deformation analysis in *Section 3.7.1*, we prepared maps showing the zone of influence and amount of vertical deformation as a result of seawall movement for four seismic hazard levels – the median estimate of a M8.0 San Andreas seismic event (similar to a 225-year return period), 475-year return period, 975-year return period, and Maximum Considered Earthquake. These maps are provided in ***Figures 3-67 through 3-74*** at the end of this section of the report.

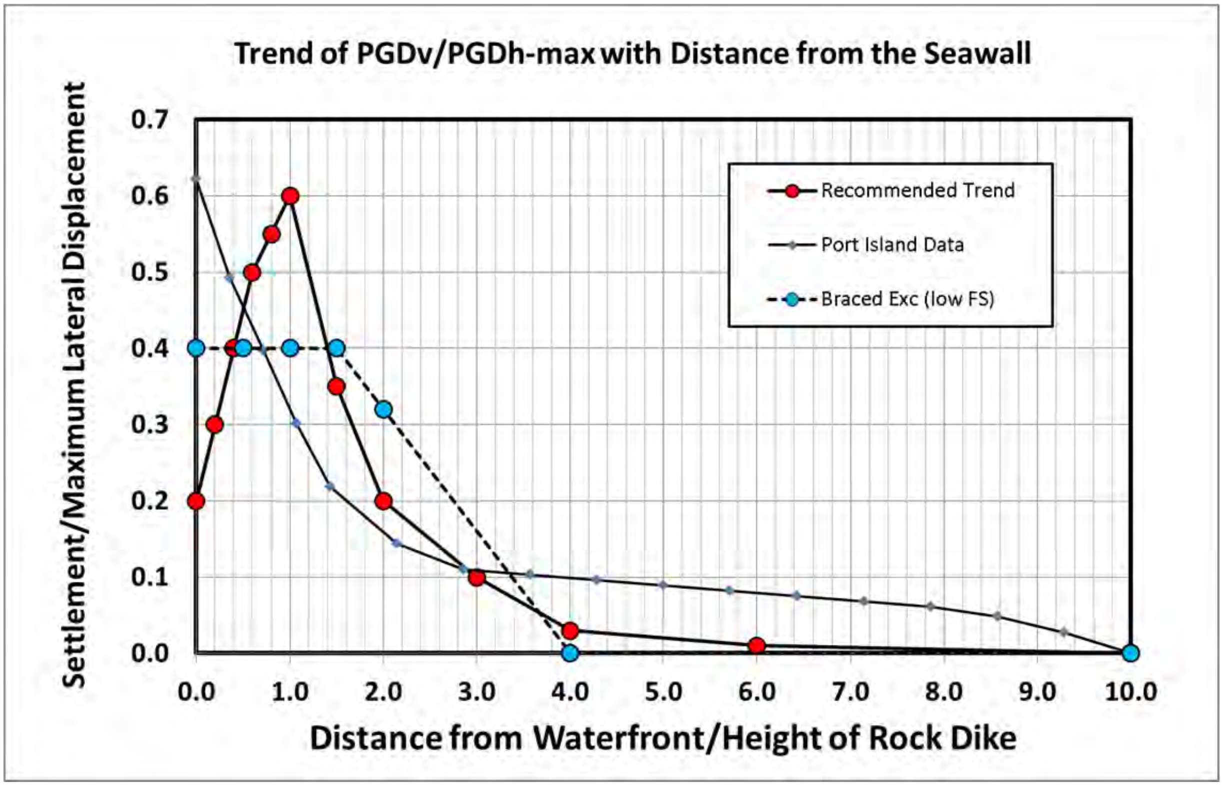


Figure 3-58: Recommended Trend in Vertical Ground Deformation with Distance from the Waterfront.

Ground Surface Settlement (Index PGD_v) Associated with Volumetric Strains

Volumetric strain associated with post-liquefaction, re-consolidation of sandy fill is influenced by the initial density of the sand, and the maximum shear strain and amount of excess pore pressure generated by the earthquake. The initial density of the sand is generally related to the corrected Standard Penetration Test (SPT) blow count, $(N_1)_{60}$, or the corrected cone penetration test (CPT) tip resistance, q_{c1N}. For $(N_1)_{60}$ values equal to or larger than about 30 and q_{c1N} values equal to or larger than about 160, the sandy fill is resistant to liquefaction triggering (Youd et al., 2001). As these values decrease, indicating a looser state of the sand, the potential for liquefaction and the amount of volumetric strain increase. Tokamatsu and Seed (1987) estimated that the volumetric strain for saturated clean sands is approximately 2 percent at an $(N_1)_{60}$ of 15 and 3 percent at an $(N_1)_{60}$ of 7. These $(N_1)_{60}$ values correspond to medium dense sand (relative densities of about 60 percent) and loose sand (relative densities of about 40 percent), respectively. Ishihara and Yoshimine (1992) estimate slightly higher post-liquefaction volumetric strains with about 2.7 percent and 4.5 percent volumetric strain at relative densities of 60 percent and 40 percent, respectively. Very loose sands may experience even greater volumetric strains during post-liquefaction reconsolidation.

Based on a review of boring logs, the depth of artificial fill behind the rock dike sections of the seawall are typically in the 20- to 25-foot range. The artificial fill includes layers of clean sand, but also include gravels, silty sand, clayey sand, silt, and clay with varying amounts of construction debris. Most of the artificial fill, except for some close to the ground surface, was placed in the mid to late 1800's and early 1900's without much compactive effort. Therefore, any fill soils that are comprised of grain sizes prone to liquefaction and volumetric reconsolidation (i.e. primarily sands and silty sands), will likely experience some settlement during moderate to strong seismic ground shaking. This has been observed during both the 1906 San Francisco Earthquake and 1989 Loma Prieta Earthquake in which large portions of the artificial fill in the man-made land behind the seawall showed evidence of liquefaction-related phenomena including settlement and sand boils.

Typically, the sandy fills encountered in borings can be described as very loose to loose to medium dense. Based on the volumetric strains observed in prior case histories and laboratory testing, and as reported by Tokamatsu and Seed (1987) and Ishihara and Yoshimine (1992), a generalized, average volumetric strain of 3 percent may be assumed for the sandy portions of the fill. Only saturated portions of the fill below the groundwater table, which was assumed at elevation +3.5 feet (NAVD88), were included in the volumetric strain calculations. Also, artificial fill soils consisting primarily of gravels, clayey sand, silt and clay were screened out as not experiencing reconsolidation settlements. Based on these simplifying assumptions, ground surface settlements associated with volumetric strains were evaluated to be between zero and 12 inches, with a more typical volumetric strain in the 2 to 6 inch range. The liquefiable soils at AT&T Park were improved using stone columns, so liquefaction potential and settlement were considered to be minor at this location.

The triggering of liquefaction for such loose sandy soils is expected to occur at relatively low levels of earthquake shaking. Sandy soils were liquefied during the relatively short duration Loma Prieta Earthquake in which the peak ground acceleration at the top of bay mud / base of fill was estimated to be 0.2 g. The excess pore pressure generation and volumetric reconsolidation were likely limited due to the characteristics of this earthquake. However, at higher levels of shaking and longer duration, including the 1906 San Francisco Earthquake, probabilistic earthquakes in excess of about 150 year return period, and DE and MCE earthquakes in accordance with ASCE 7-10, the amount of settlement caused by liquefaction-related volumetric strain are considered to be nearly equivalent.

We summed the contributions of vertical settlement from deviatoric strains caused by seawall movement and volumetric strains from liquefaction, and prepared maps showing the zone of influence and amount of total vertical settlement for four seismic hazard levels – the median estimate of a M8.0 San Andreas seismic event, 475-year return period, 975-year return period, and Maximum Considered Earthquake. These maps are provided in ***Figures 3-75 through 3-82*** at the end of this section of the report.

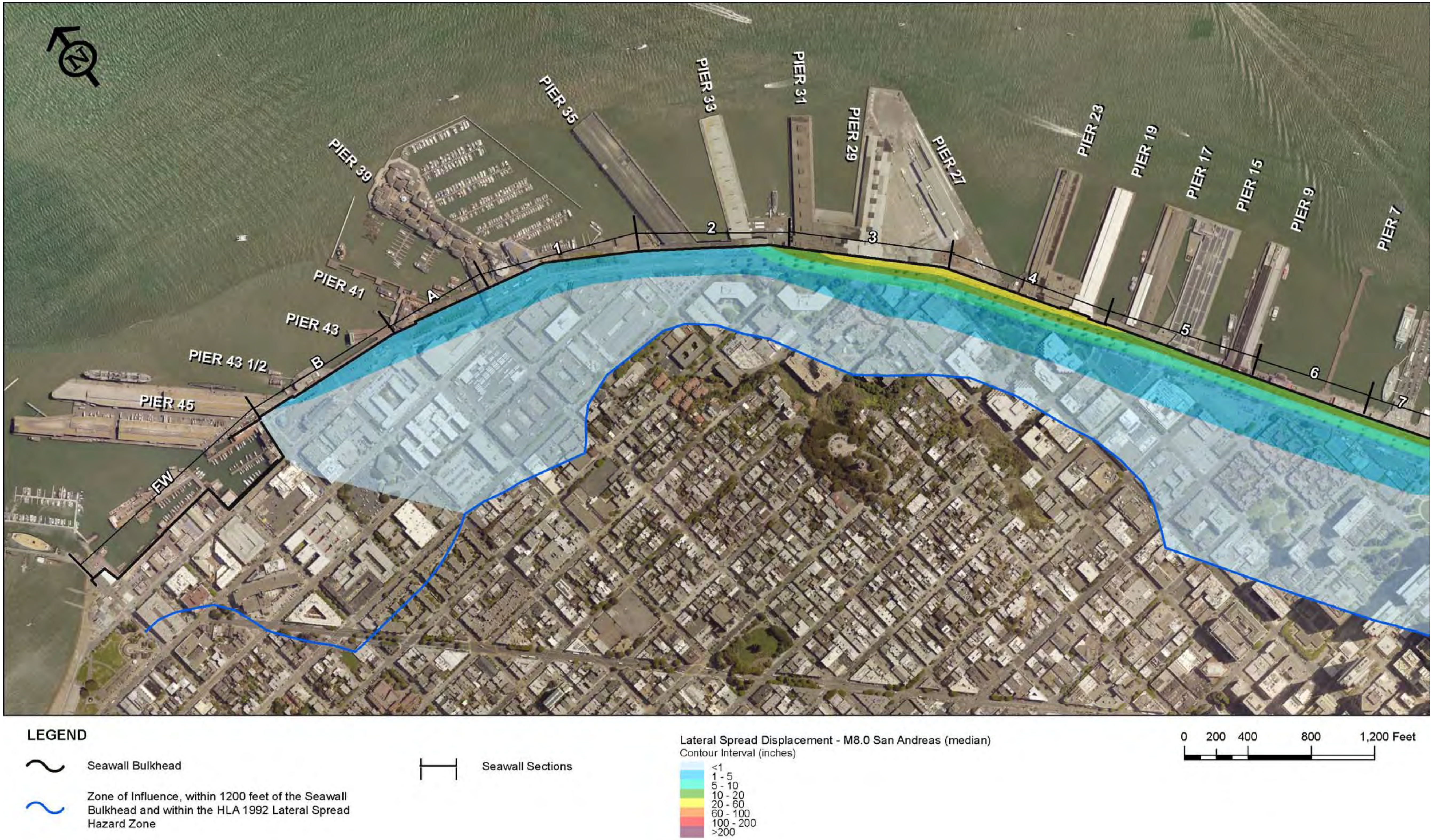


Figure 3-59: Lateral Spread Displacement from M8.0 San Andreas (Median) Seismic Event – Seawall Sections B through 6

LEGEND

Seawall Bulkhead

Zone of Influence, within 1200 feet of the Seawall Bulkhead and within the HLA 1992 Lateral Spread Hazard Zone

Seawall Sections

Lateral Spread Displacement - M8.0 San Andreas (median)
Contour Interval (inches)

- <1
- 1 - 5
- 5 - 10
- 10 - 20
- 20 - 60
- 60 - 100
- 100 - 200
- >200

0 200 400 800 1,200 Feet

Figure 3-60: Lateral Spread Displacement from M8.0 San Andreas (Median) Seismic Event – Seawall Sections 7 through 46

Figure 3-61: Lateral Spread Displacement from 475-Year Return Period Seismic Event – Seawall Sections B through 6

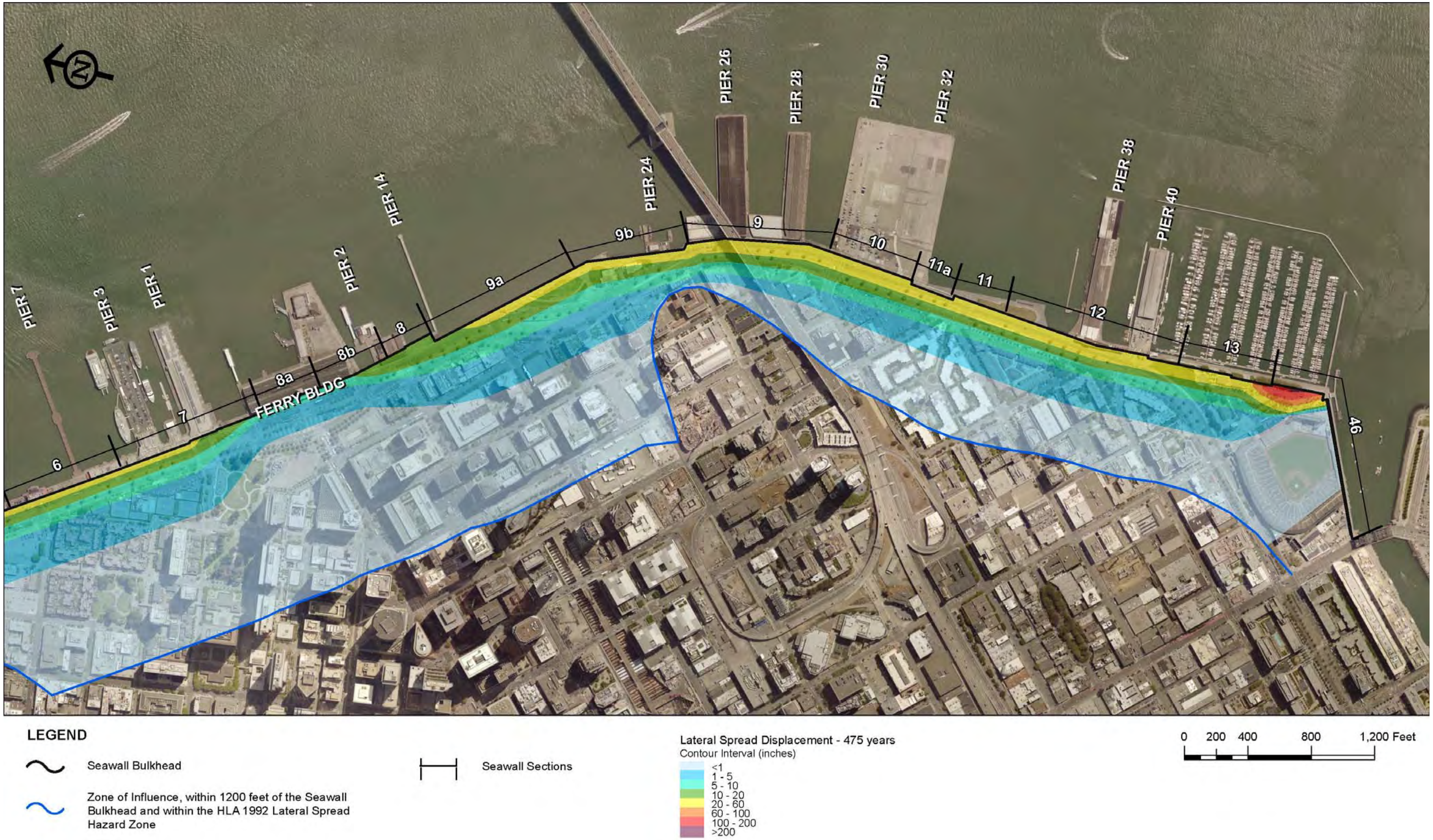


Figure 3-62: Lateral Spread Displacement from 475-Year Return Period Seismic Event – Seawall Sections 7 through 46

Figure 3-63: Lateral Spread Displacement from 975-Year Return Period Seismic Event – Seawall Sections B through 6

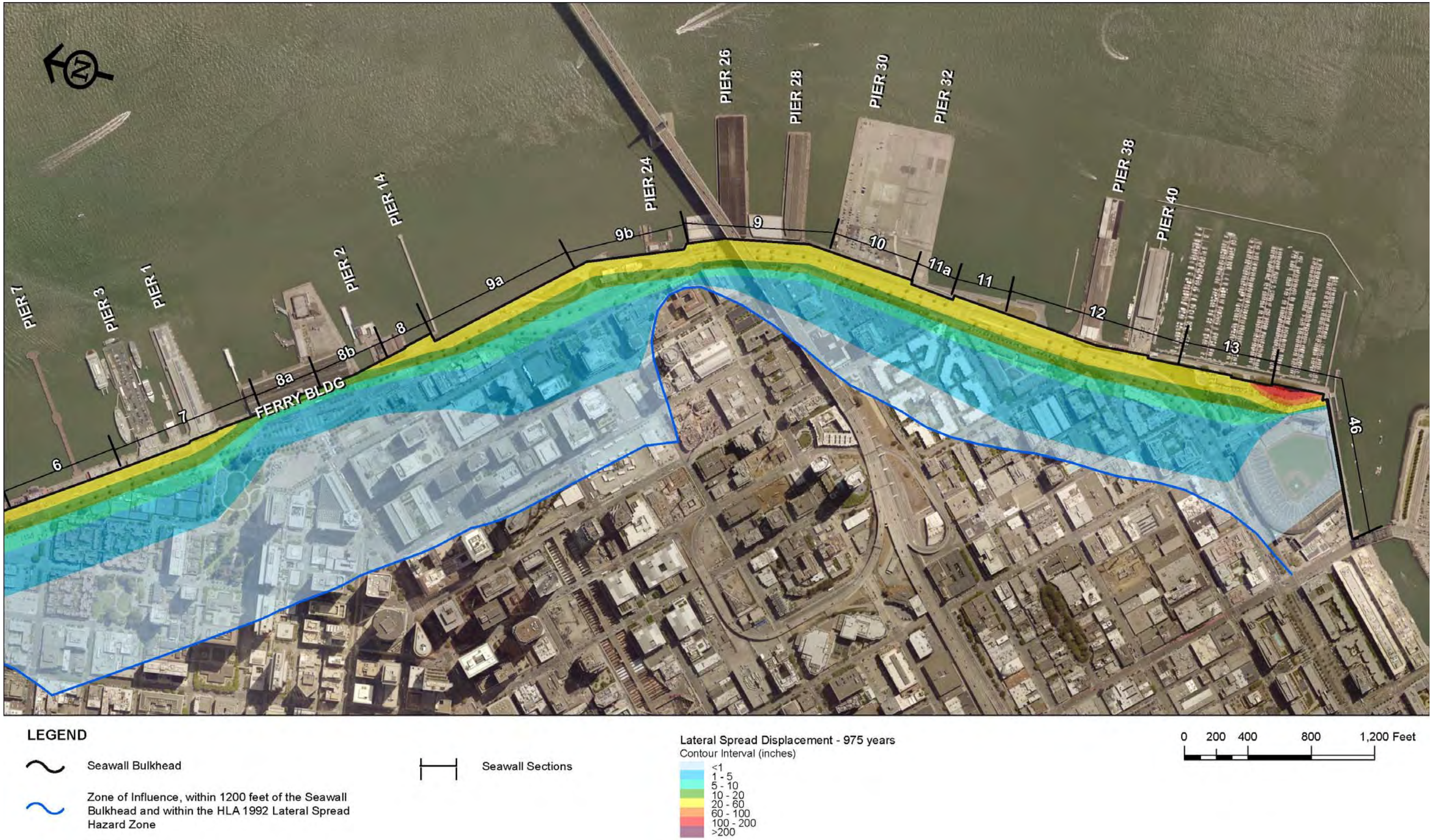


Figure 3-64: Lateral Spread Displacement from 975-Year Return Period Seismic Event – Seawall Sections 7 through 46

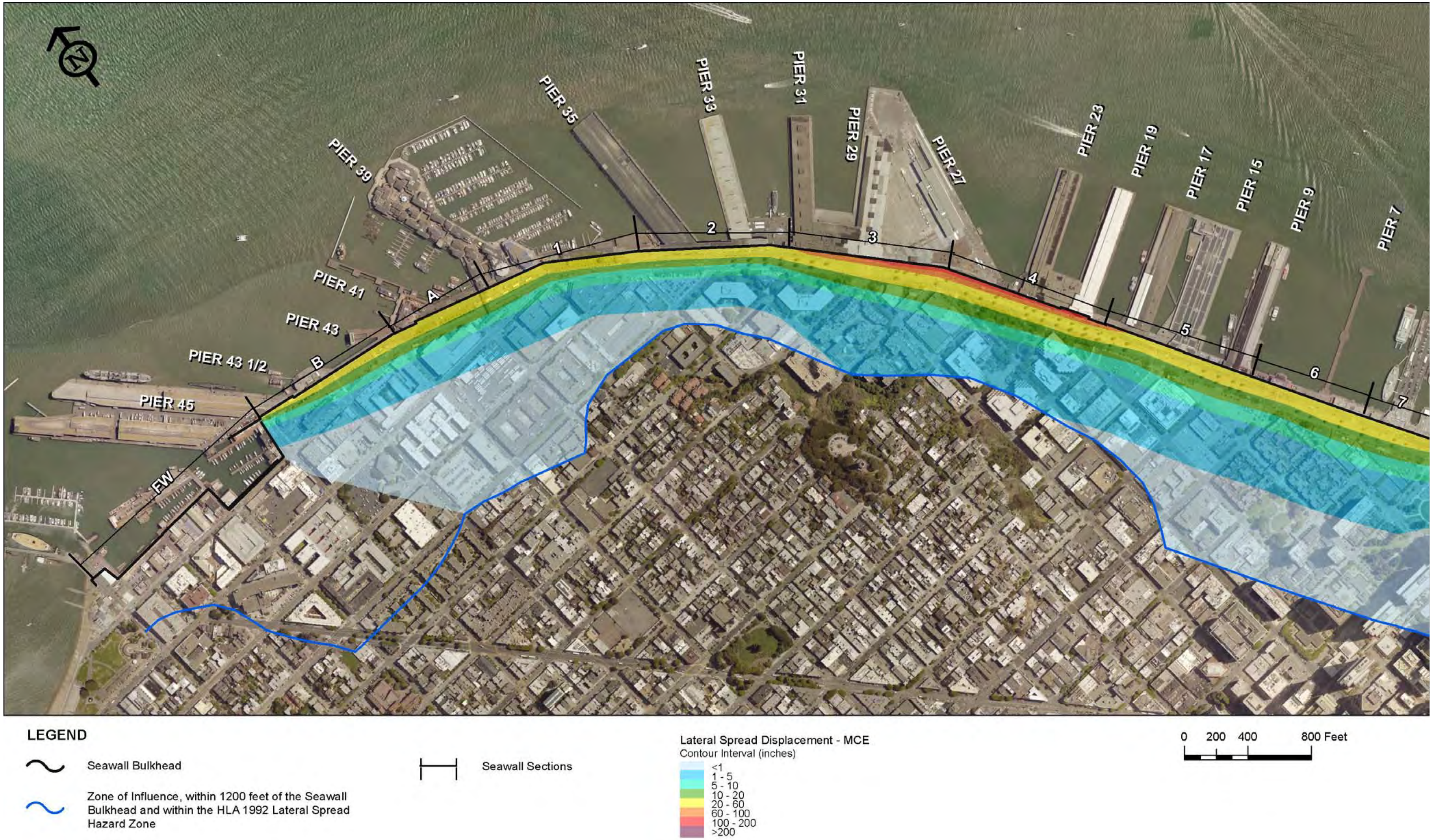


Figure 3-65: Lateral Spread Displacement from Maximum Considered Earthquake – Seawall Sections B through 6

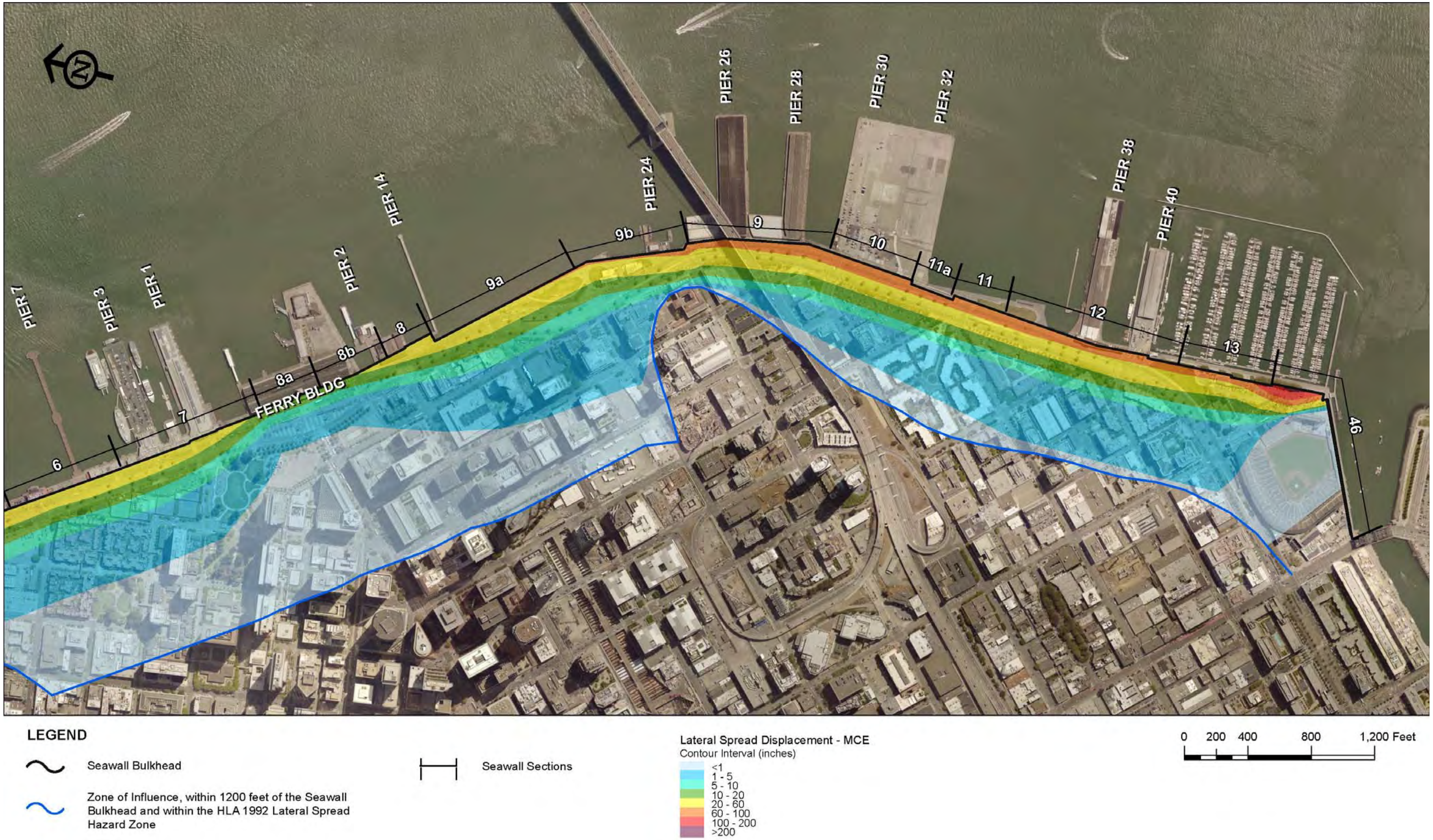


Figure 3-66: Lateral Spread Displacement from Maximum Considered Earthquake – Seawall Sections 7 through 46

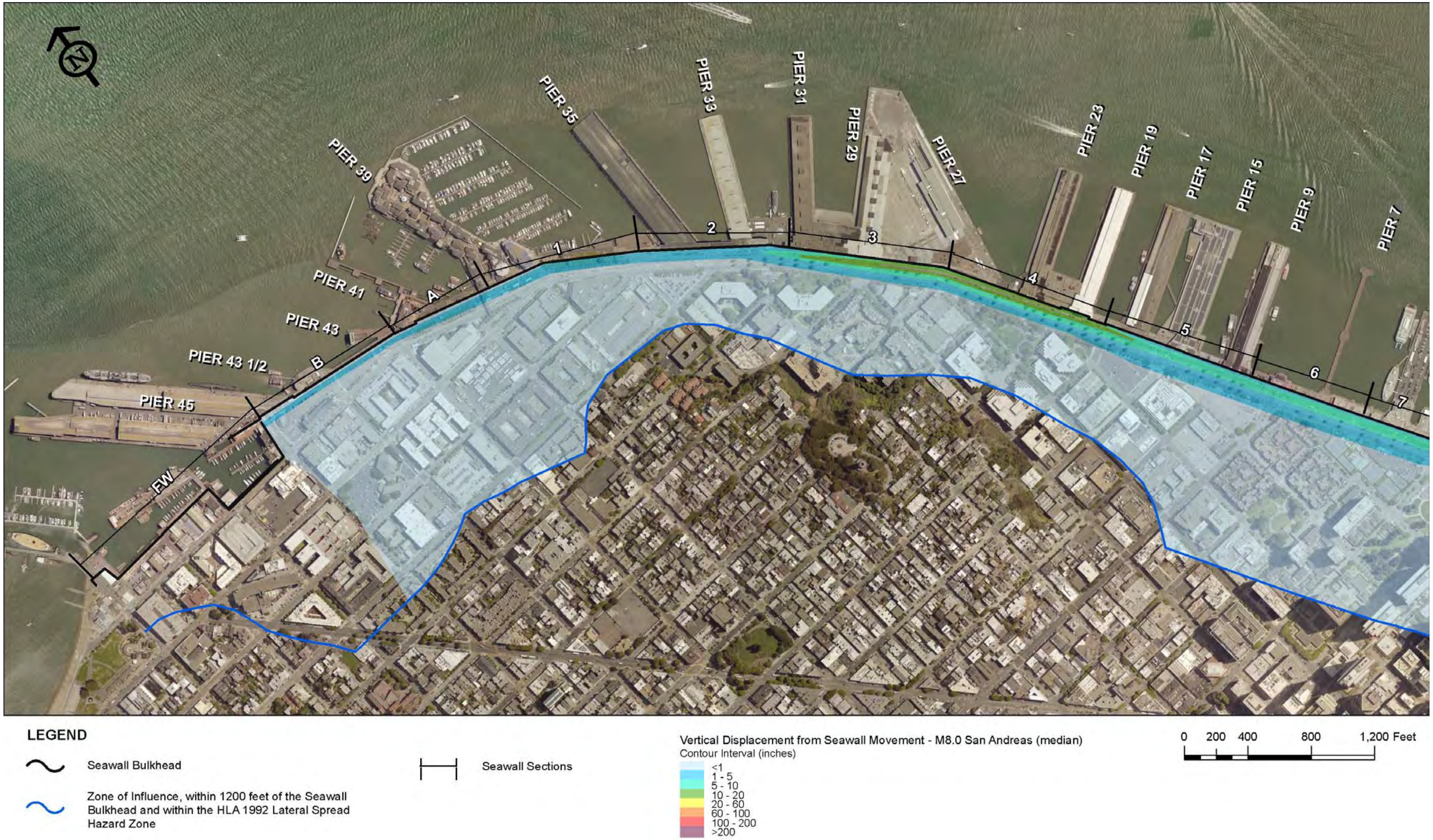


Figure 3-67: Vertical Displacement from Seawall Movement from M8.0 San Andreas (Median) Seismic Event – Seawall Sections B through 6

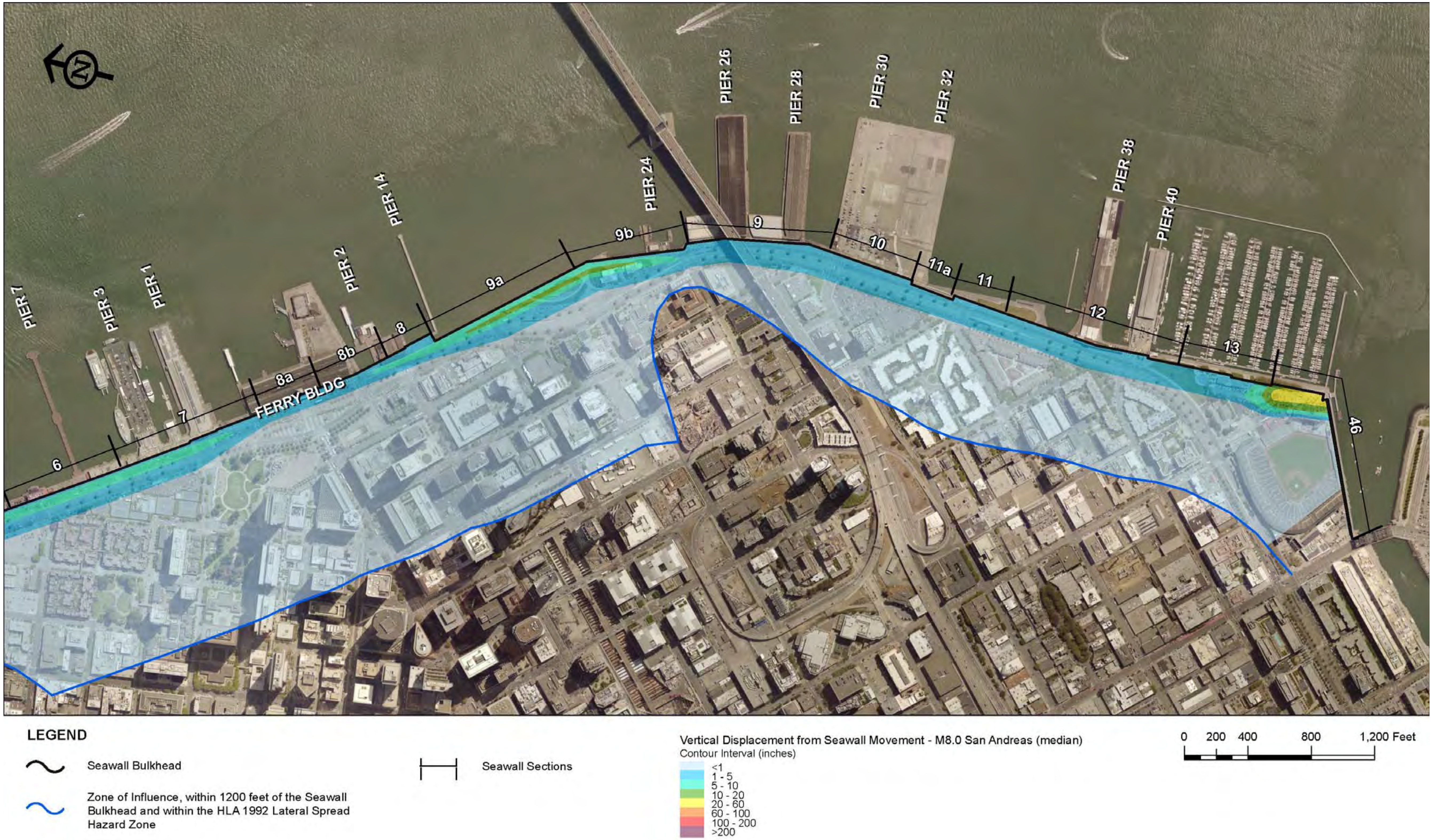


Figure 3-68: Vertical Displacement from Seawall Movement from M8.0 San Andreas (Median) Seismic Event – Seawall Sections 7 through 46

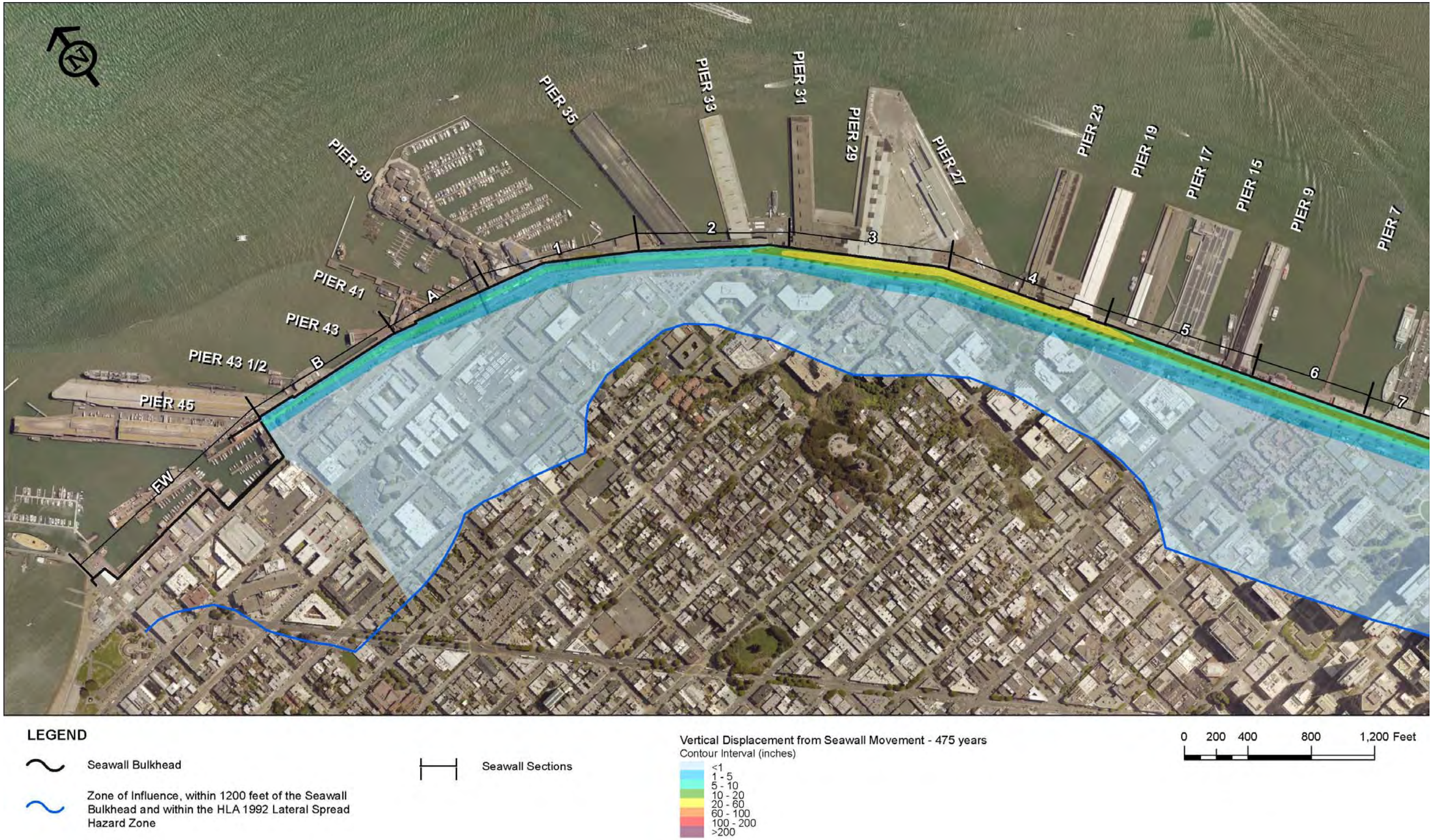


Figure 3-69: Vertical Displacement from Seawall Movement from 475-Year Return Period Seismic Event – Seawall Sections B through 6

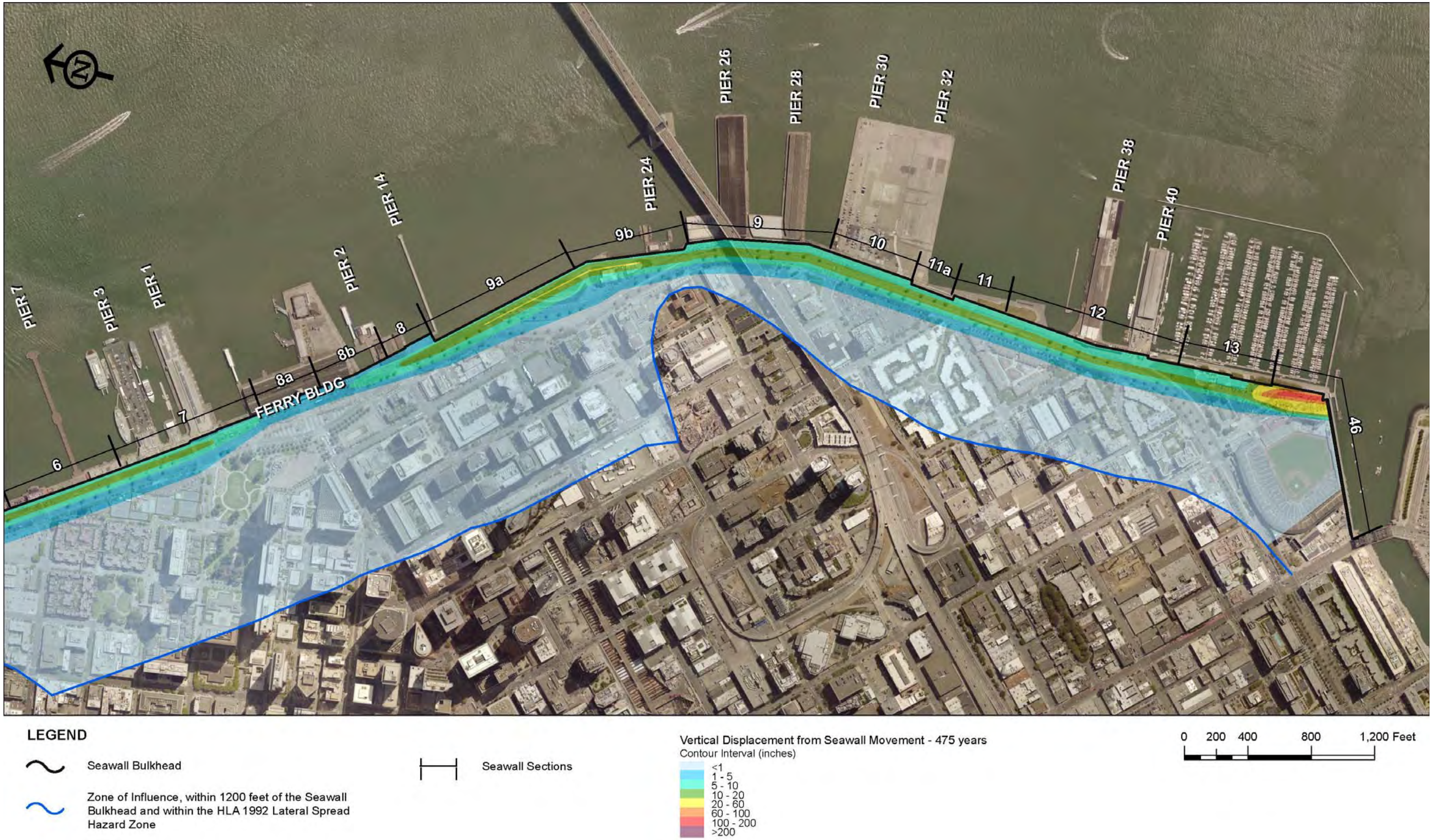


Figure 3-70: Vertical Displacement from Seawall Movement from 475-Year Return Period Seismic Event – Seawall Sections 7 through 46

Figure 3-71: Vertical Displacement from Seawall Movement from 975-Year Return Period Seismic Event – Seawall Sections B through 6

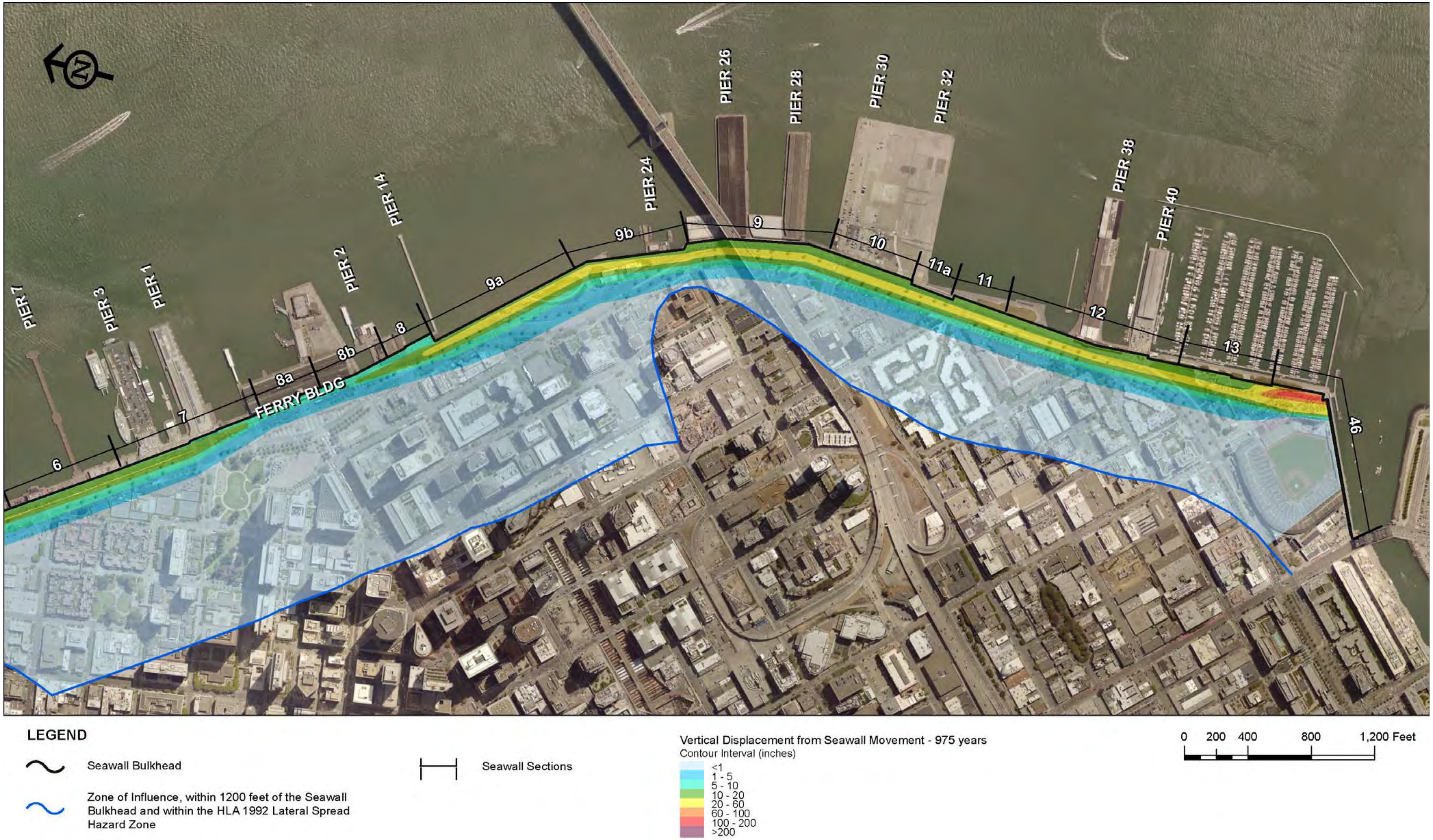


Figure 3-72: Vertical Displacement from Seawall Movement from 975-Year Return Period Seismic Event – Seawall Sections 7 through 46

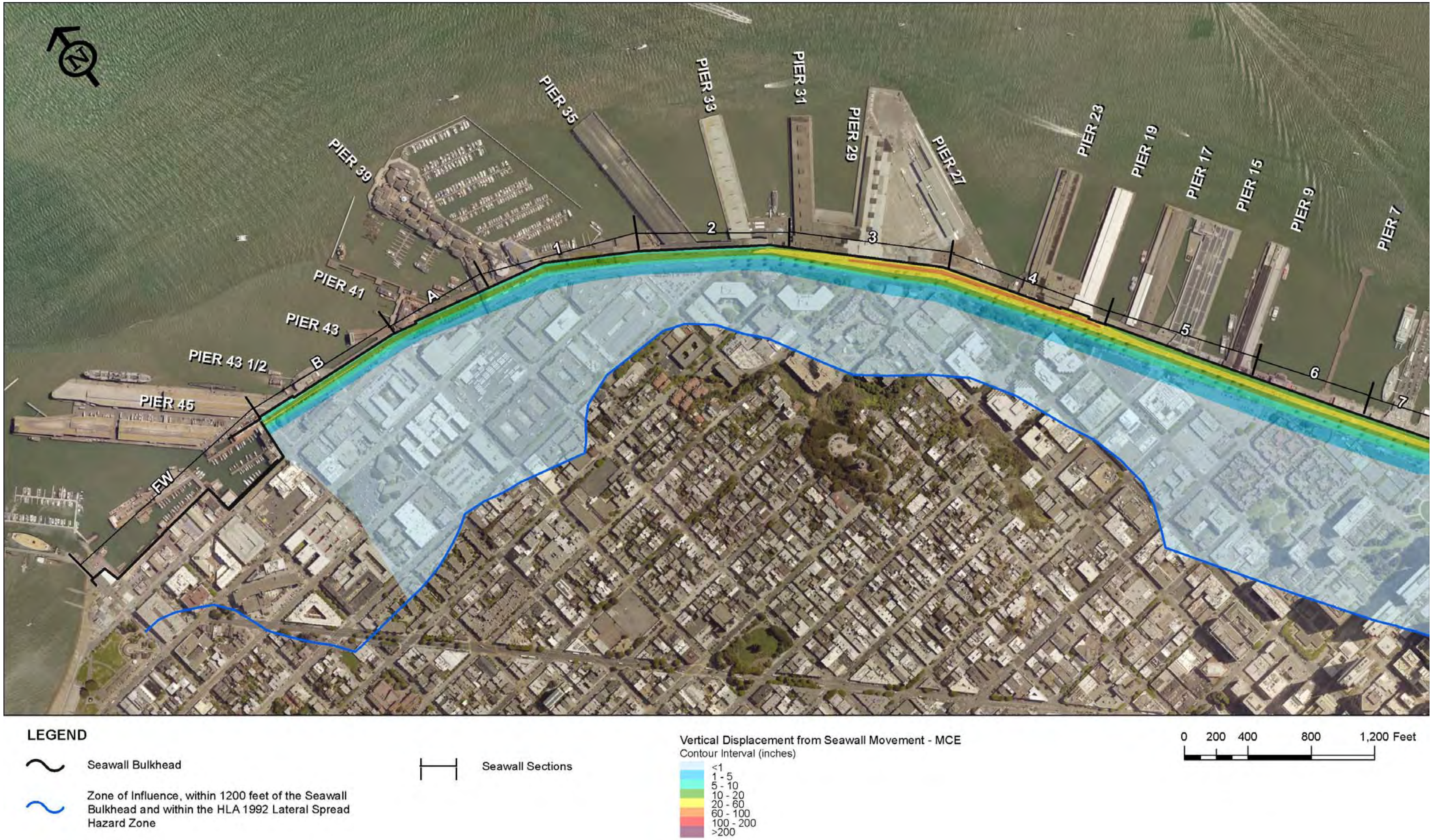


Figure 3-73: Vertical Displacement from Seawall Movement from Maximum Considered Earthquake – Seawall Sections B through 6

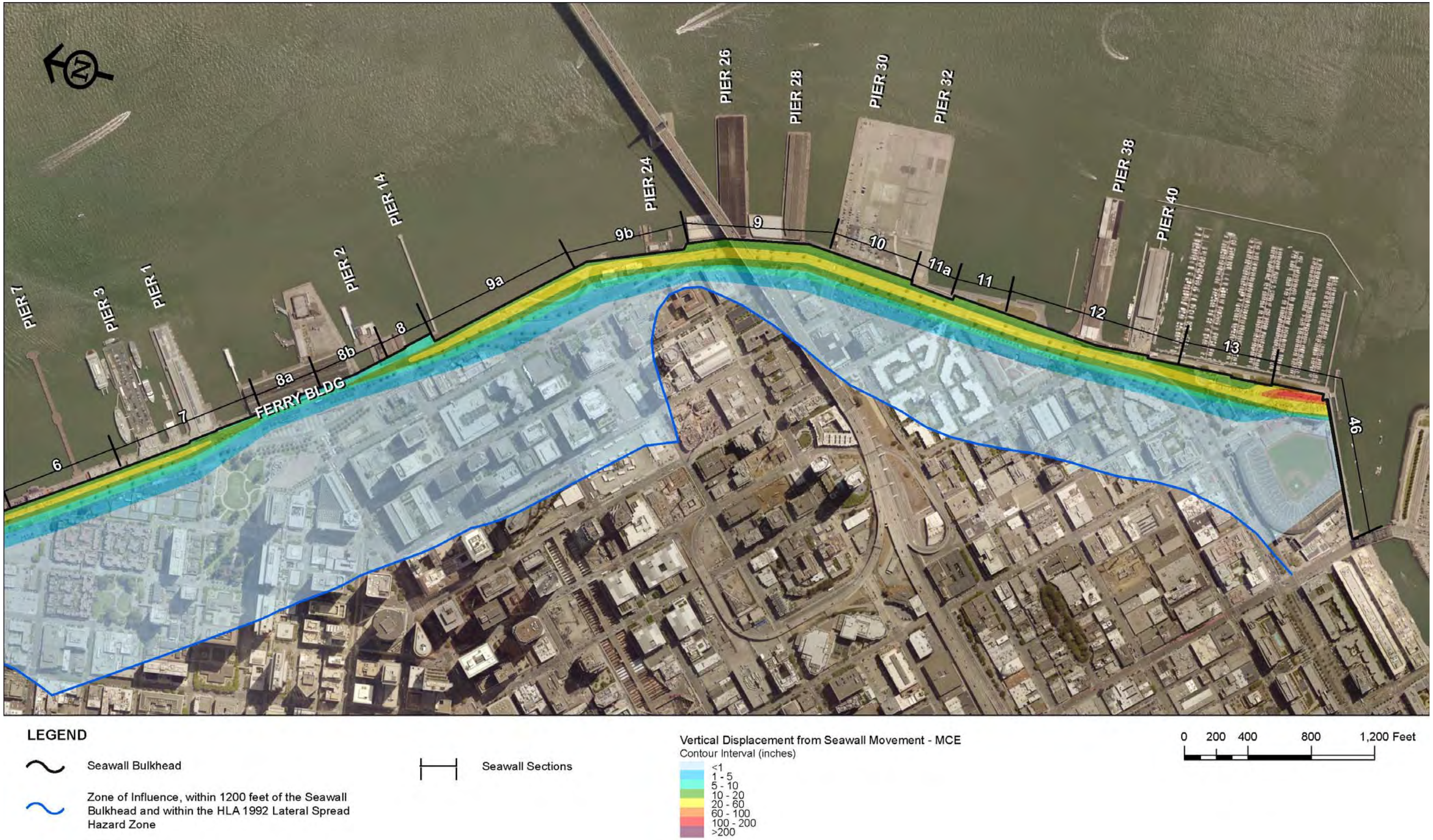


Figure 3-74: Vertical Displacement from Seawall Movement from Maximum Considered Earthquake – Seawall Sections 7 through 46

Figure 3-75: Total Vertical Displacement from M8.0 San Andreas (Median) Seismic Event – Seawall Sections B through 6

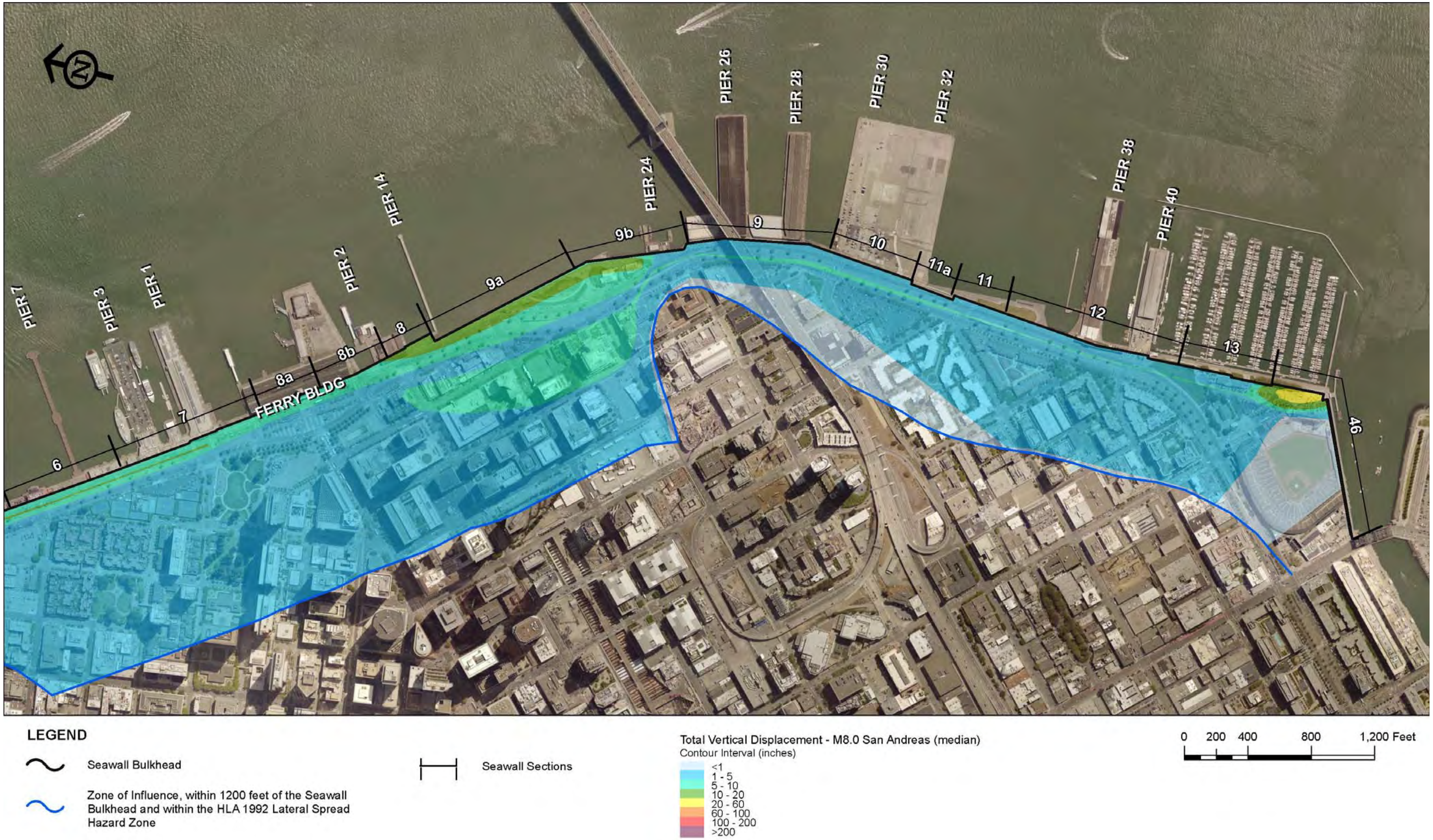


Figure 3-76: Total Vertical Displacement from M8.0 San Andreas (Median) Seismic Event – Seawall Sections 7 through 46

Figure 3-77: Total Vertical Displacement from 475-Year Return Period Seismic Event – Seawall Sections B through 6

Figure 3-7816: Total Vertical Displacement from 475-Year Return Period Seismic Event – Seawall Sections 7 through 46

Figure 3-7917: Total Vertical Displacement from 975-Year Return Period Seismic Event – Seawall Sections B through 6

Figure 3-80 Total Vertical Displacement from 975-Year Return Period Seismic Event – Seawall Sections 7 through 46

Figure 3-81: Total Vertical Displacement from Maximum Considered Earthquake – Seawall Sections B through 6

Figure 3-82: Total Vertical Displacement from Maximum Considered Earthquake – Seawall Sections 7 through 46

3.6. Geotechnical Design Parameters for Structural Assessment

In addition to the analyses provided in previous sections, GTC and NA evaluated geotechnical design parameters in order for the structural engineering team to evaluate bulkhead wall stability and bulkhead wharf behavior. The following items were evaluated, and the results are presented in the following sections of this report.

1. Bulkhead Seawall
    a. Static
        i. At rest/active soil pressures on walls.
        ii. Passive soil pressures on walls.
        iii. Surcharge pressures on walls, as a percentage of surface surcharge.
        iv. Bearing capacity of rock dike at base of bulkhead walls for static loads.
    b. Seismic for four earthquake levels (M8.0 San Andreas - median, 475-yr, 975-yr, MCE).
        i. Seismic (dynamic) soil pressures on seawalls.
        ii. Seismic acceleration at wall location to determine wall inertial loads.
        iii. Bearing capacity of rock dike at base of bulkhead walls for seismic loads.
2. Bulkhead Wharf
    a. Lateral load deflection behavior (p-y data) in the form of LPILE input parameters for soil strata.
    b. Pile axial skin friction and end bearing resistance.

Bulkhead Seawall Stability

<u>Lateral Earth Pressures</u>

We evaluated lateral earth pressures on the existing concrete seawalls. The lateral earth pressure diagrams are included on ***Figures 3-83 and 3-84 – Lateral Earth Pressures***.

Active earth pressures are imposed by the soil on walls that are unrestrained so that the top of the wall is free to translate or rotate at least 0.004H, where H is the height of the wall. The active earth pressure varies with effective stress in the soil and the soil type. Our recommended distribution of active earth pressure with depth is provided on ***Figure 3-83***.

Unbalanced hydrostatic pressures should be accounted for in the stability evaluation of the wall. We estimate that the maximum unbalanced hydrostatic level between the tide level in the bay and the groundwater level behind the wall is approximately 3.84 feet based on an average groundwater level behind the wall at approximately elevation +3.5 feet (NAVD88). The Mean Lower Low Water level is at approximately -0.34 feet, which was adopted as the tide level in the bay. The pressure distribution caused by this unbalanced hydrostatic pressure is provided on ***Figure 3-83***.

The seawall may experience additional lateral loads imposed by traffic, construction, storage of materials, or other at-grade applied vertical surcharge loads. The lateral distribution of surcharge loads depends on the load magnitude, location, and surface dimensions of the load footprint (e.g., point loads, areal loads). For uniform areal loads over a large footprint, we recommend that a uniform lateral pressure increment of 0.30 x q be applied to the seawall, where q is the applied surface pressure. This load should be applied in a rectangular pressure distribution on the seawall as shown on ***Figure 3-83***.

In addition to the active earth pressures, the seawall should be designed to consider additional earth pressures due to earthquake loading. The seawall stability should be evaluated for two conditions: i) retained soil is non-liquefiable in which case the seismic earth pressure can be evaluated using the Mononobe-Okabe (M-O) seismic coefficient analysis (Seed and Whitman, 1970); and ii) retained soil below the groundwater table liquefies in which case the soil pressures below the water level act as a heavy fluid. For the M-O analysis, the seismic earth pressure increment is a function of the peak ground acceleration. The peak ground accelerations used in the M-O analyses are based on the site-specific ground motions and microzonation performed for the project and detailed in *Section 3.5 – Seismic Ground Motions*. The peak ground accelerations vary across the waterfront depending on subsurface conditions, so each seawall section was evaluated separately. In accordance with recommendations to use seismic coefficients corresponding to $^{1}/_{3}$ to ½ of the peak acceleration of the design earthquake (Whitman, 1990), we used a horizontal seismic coefficient, k_h, of one-half of the peak ground acceleration of the various seismic events. The pressure distribution for the seismic earth pressure may be taken as a rectangular distribution and the pressures vary from 7H pounds per square foot (psf) to 13H psf across all retaining wall sections and the five selected earthquake scenarios (Maximum Considered Earthquake (MCE), Design Earthquake (DE), 475-year and 975-year return period, and the median estimate of a M8.0 San Andreas seismic event). The distribution of the earth pressure due to seismic loading is provided on ***Figure 3-83***. The magnitude of the seismic earth pressures are presented in ***Table 3-3***. The distribution of earth pressure in the event of liquefaction of the sandy fill below the groundwater table is provided on ***Figure 3-84***.

Table 3-3: Seismic Increment Pressures

Earthquake Ground Motion Level	**Seismic Increment Pressures (psf)**		
	Zone A1/A2	**Zone B1/B2**	**Zone C2**
MCE	13H	9H	9H
DE	8H	8H	7H
975 year return period	11H	9H	8H
475 year return period	10H	9H	8H
M8.0 San Andreas - median	7H	7H	7H

Note: H is the height of the concrete seawall in feet.

Lateral loads on the seawall can be resisted by a combination of passive pressures and either the shear and bending moment resistance of the seawall piles or shear resistance along the base. While shear resistance is mobilized with small lateral movements, the passive pressure depends on more significant lateral displacement of the wall. Soft clays and loose sands generally require more displacement to mobilize the ultimate passive resistance than stiff clays, dense sands and gravels. For the material types existing at the face of the embedded portion of the seawall, we estimate that the displacement to achieve ultimate passive pressure resistance is approximately 5 percent of the depth of embedment of the seawall, Z. The distribution of passive earth pressure with depth is provided on ***Figure 3-83***. As presented, the ultimate passive pressure resistance may be considered to be somewhat conservative since we are unsure of the current condition of the bulkhead retaining walls in the rock dike, and the potential steep angle of the rock dike down and away from the toe of the wall. The ultimate passive earth pressures presented are based on the unit weight and shear strength profile of bay mud. These are Rankine earth pressures and neglect wall friction. Oftentimes, the displacement to achieve ultimate passive earth pressures exceeds the allowable displacement of the structure. We estimate that approximately 85 percent of the ultimate passive resistance will be mobilized with a displacement of 2.5 percent of Z, and 50 percent of the ultimate passive resistance will be mobilized with a displacement of 0.5 percent of Z.

Along portions of the seawall, riprap revetment has been placed. The lateral resistance due to passive earth pressures from the revetment is provided on ***Figure 3-85 – Passive Pressure of Riprap Revetment***. The earth pressures are based on Rankine earth pressure theory and neglect wall friction. Riprap revetment was estimated to have a unit weight of 132 pcf and a friction angle of 42 degrees. To achieve 100 percent of the ultimate passive resistance, the bench width extending from the concrete seawall at constant elevation should be at least two times the riprap height, D. A reduced ultimate passive resistance is also provided on ***Figure 3-85*** for riprap that is sloped at 3 to 1 (horizontal to vertical) starting at the face of the seawall. For riprap, we estimate that the displacement to achieve ultimate passive pressure resistance is approximately 2 percent of the depth of embedment of the seawall below the top of riprap, Z. Oftentimes, the displacement to achieve ultimate passive earth pressures exceeds the allowable displacement of the structure. We estimate that approximately 85 percent of the ultimate passive resistance will be mobilized with a displacement of 1 percent of Z, and 50 percent of the ultimate passive resistance will be mobilized with a displacement of 0.3 percent of Z.

The displacement of the seawall is also resisted by the lateral resistance of the existing piles and/or friction along the seawall base. A lateral load versus displacement analysis (e.g. LPILE analysis) can be performed to evaluate the contribution of the piles to seawall stability if the piles are judged to be in sound condition. The LPILE parameters provided in the following section may be used to perform such an analysis. In the event that the existing piles fail because of exceeding shear or bending moment capacities, the friction along the base of the wall can be utilized to assess seawall stability. A coefficient of friction of 0.35 may be used for estimating the resistance due to base friction. The coefficient should be multiplied by the dead load only, and should account for the buoyant weight of the seawall for the submerged portion.

Bearing Capacity

The base of the seawall bulkhead is generally founded on rock dike material that has good supporting capacity and is not prone to strength loss during earthquake shaking. However, it should be noted that the rock dike material is widely varying and includes zones of loose sand, clay, and other miscellaneous fill materials and construction debris. For the majority of rock dike fill, the ultimate bearing capacity of the walls are anticipated to be 10 kips per square foot (ksf) or higher. Generally, a lesser bearing capacity is used under static loading conditions to limit the settlement to an acceptable level. We recommend a factor of safety of 3 to limit settlement to 1 inch. For seismic loading conditions, a factor of safety of 1.5 to 2 would be appropriate to limit settlements to acceptable levels. Lower factors of safety will result in additional settlement of the seawall bulkhead but will not result in a bearing capacity failure unless the ultimate bearing capacity is exceeded. The rock dike material is generally founded on either young bay mud and/or sandy fill or natural sandy deposits that will likely experience permanent soil deformation during moderate to strong ground shaking, and therefore the seawall bulkhead will displace along with the movement of the rock dike mass.

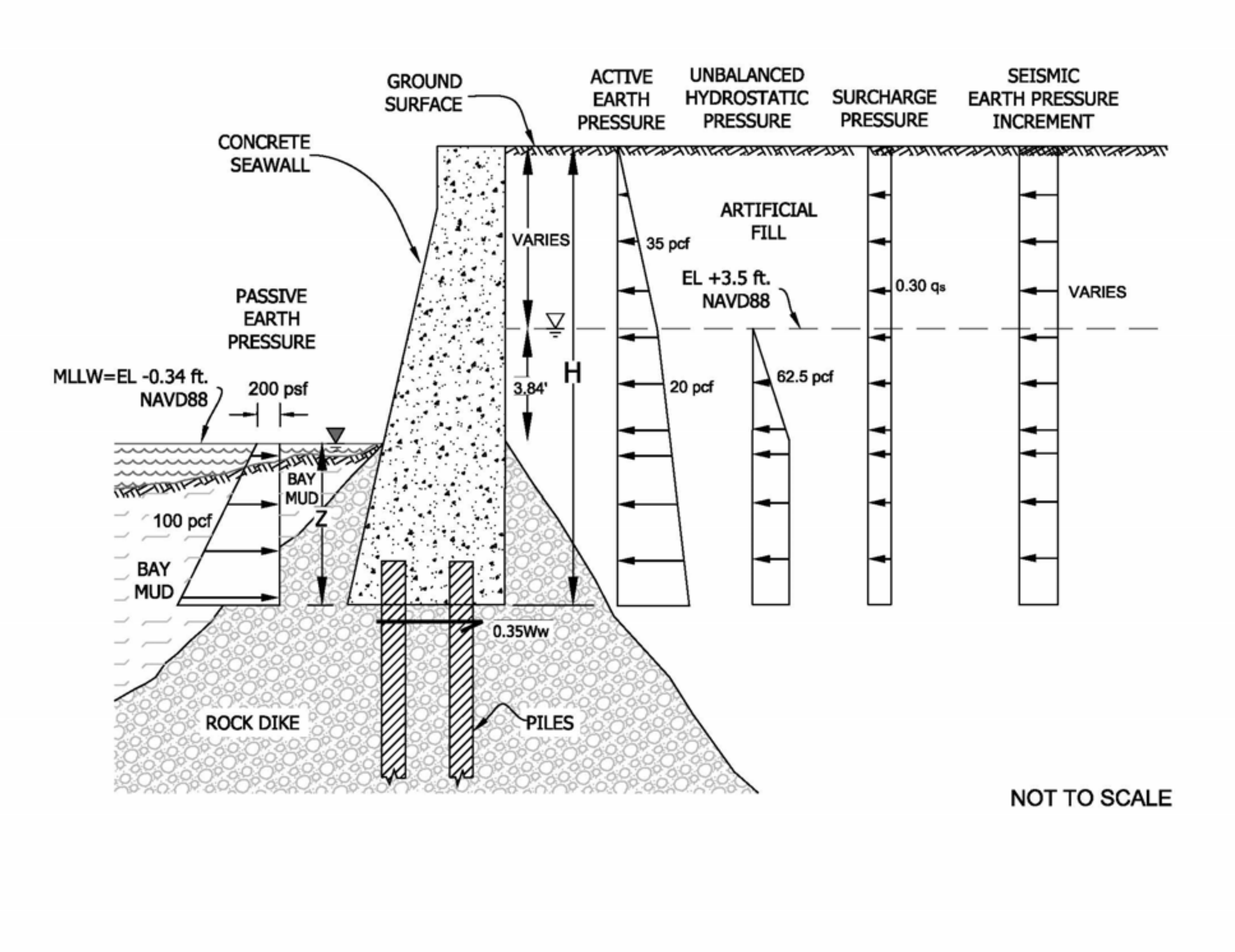


Figure 3-83:Lateral Earth Pressures – Non-Liquefied Fill

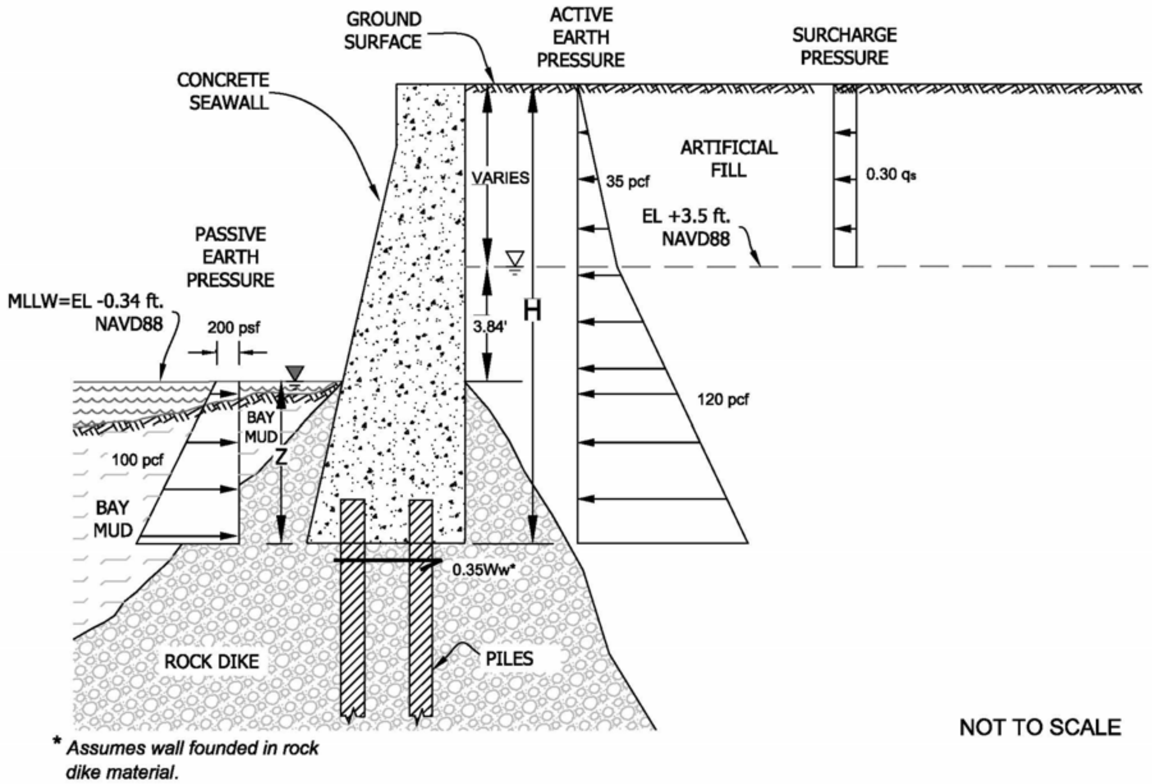


Figure 3-8418: Lateral Earth Pressures – Fill Below Groundwater Liquefied

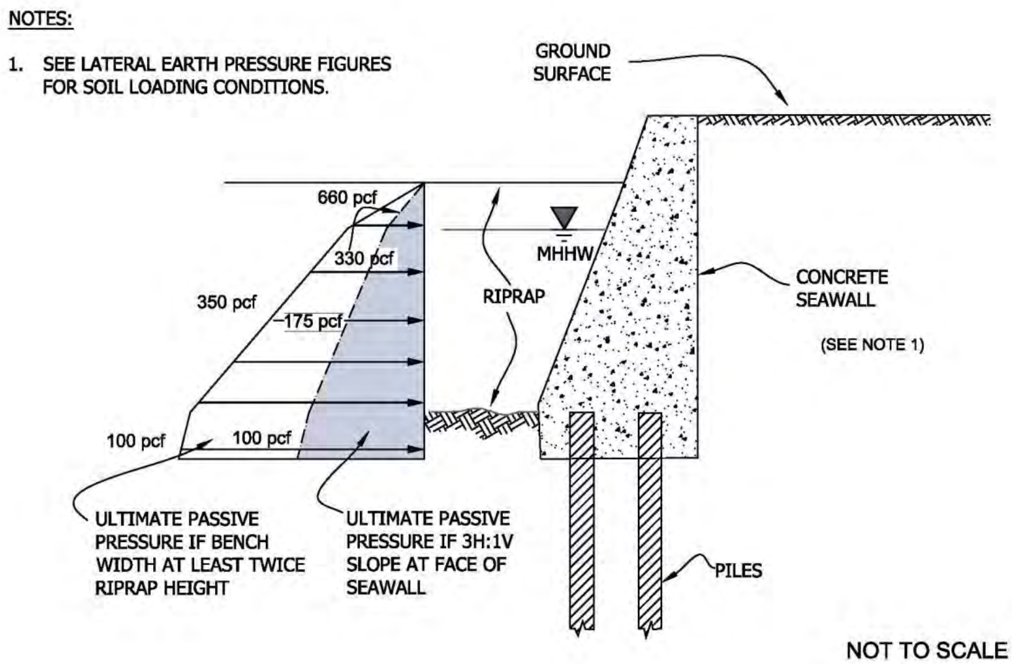


Figure 3-85:Passive Earth Pressure of Riprap Revetment

Bulkhead Wharf

The bulkhead wharves are pile-supported structures. Portions of the waterfront have a marginal wharf on which there are no significant, large structures, while other portions have a bulkhead building with multiple uses including office space, restaurants and other commercial and maritime uses. The following sections provide the generalized lateral and axial capacity of existing driven piles on the landward and near-shore area of the Northern Waterfront Seawall. The subsurface conditions were idealized at nine section locations that are considered to be representative of the project area. The recommended lateral and axial parameters are not applicable to drilled-in-place foundations nor for offshore piles that are outside of the seawall zone of influence. This also does not address the important issue of kinematic loading on existing pile foundations caused by horizontal permanent ground displacements. These issues can be evaluated and provided at specific structure locations, as required.

Lateral Load Capacities of Driven Piles

Resistance to lateral loading will be provided by passive resistance of the soil against the pile. In lateral load analyses, non-linear soil springs are applied at each depth increment, and are represented by soil resistance, p, at a lateral deflection, y. Soil parameters to generate "p-y" springs in the computer program LPILE are provided for nine idealized soil profiles in ***Tables 3-6 through 3-14 – Preliminary LPILE Parameters for Lateral Pile Analysis***. These springs can then be used to evaluate the load-deflection response of individual piles.

The "p-y" springs generated using the soil parameters provided in ***Tables 3-6 through 3-14*** are applicable for individual piles only. Lateral response of piles in a group is affected by pile spacing, pile orientation, and direction of loading. If the spacing of individual piles is at least five pile diameters, the piles may be assumed to develop their full capacity. If the piles are spaced three pile diameters, there will be overlapping of shear failure planes, and the ultimate resistance of the piles in a group will be less than the summation of the ultimate resistance of individual piles. At a spacing of three pile diameters in the direction of loading, we recommend the ultimate soil resistance, p_{ult}, should be reduced to 90 percent of p_{ult} of an individual pile for the leading row of piles in the group and to 70 percent of p_{ult} for the trailing piles in a group. If the side-to-side spacing is also at three pile diameters, an additional reduction factor should be applied so that the leading piles are 85 percent of p_{ult} and the trailing piles are 65 percent of p_{ult}. For spacing between three and five pile diameters, the ultimate soil resistance reduction factors may be obtained by interpolation. Some manipulation of the soil strength parameters or manually inputting "p-y" springs will be necessary to correctly model piles in a group using the LPILE program. Alternatively, computer program GROUP can be used to model a segment of the wharf structure, which will internally generate group efficiency factors.

Table 3-4: Preliminary LPILE Parameters for Lateral Pile Analysis – Section B

SECTION B
PRELIMINARY LPILE PARAMETERS FOR LATERAL PILE ANALYSIS

Modeled Soil Unit	Elevation	Total Unit Weight	Effective Unit Weight	Soil Strength		Lateral Pile Capacity Assessment		
				c	ϕ	Soil Type for p-y curve	k	ε_{50}
	(feet, NAVD88)	(pcf)	(pcf)	(psf)	(deg.)		(pci)	
Seawall Rock Dike	9 to -15	120	58	0	36	"Sand"	90	-
Marine Deposit - Sand	-15 to -20	132	70	0	32	"Sand"	20	-
Marine Deposit – Sand LIQUEFIED	-15 to -20	132	70	400	0	"Soft Clay"	-	0.04
Young Bay Mud	-20 to -33	112	50	650 top - 850 bot.	-	"Soft Clay"	-	0.01
Upper Layered Sediments - Sand	-33 to -46 and Below -57	139	77	0	40	"Sand"	125	-
Upper Layered Sediments - Clay	-46 to -57	138	76	1,500	-	"Stiff Clay w/o Free Water"	-	0.005

NOTES:

1. ***c*** *= cohesion,* ϕ *= friction angle,* ***k*** *= modulus of horizontal subgrade reaction for LPILE analysis, and* ε_{50} *= value of strain at 50% maximum stress for clays in LPILE analysis.*

Table 3-5: Preliminary LPILE Parameters for Lateral Pile Analysis – Sections A and 1

SECTIONS A AND 1
PRELIMINARY LPILE PARAMETERS FOR LATERAL PILE ANALYSIS

Modeled Soil Unit	Elevation	Total Unit Weight	Effective Unit Weight	Soil Strength		Lateral Pile Capacity Assessment		
				c	ϕ	Soil Type for p-y curve	k	ε_{50}
	(feet, NAVD88)	(pcf)	(pcf)	(psf)	(deg.)		(pci)	
Seawall Rock Dike	7 to -27	120	58	0	36	"Sand"	90	-
Marine Deposit - Clay	-27 to -34	105	43	500	-	"Soft Clay"	-	0.02
Marine Deposit - Sand	-34 to -54	115	53	0	32	"Sand"	20	-
Upper Layered Sediments - Sand	-54 to -71	132	50	0	40	"Sand"	125	-
Old Bay Clay	Below -71	112	50	2,000	-	"Stiff Clay w/o Free Water"	-	0.005

NOTES:

1. *c = cohesion, ϕ = friction angle, k = modulus of horizontal subgrade reaction for LPILE analysis, and ε_{50} = value of strain at 50% maximum stress for clays in LPILE analysis.*

Table 3-6: Preliminary LPILE Parameters for Lateral Pile Analysis – Sections 2, 3 4 and 5

SECTIONS 2, 3, 4 AND 5
PRELIMINARY LPILE PARAMETERS FOR LATERAL PILE ANALYSIS

Modeled Soil Unit	Elevation	Total Unit Weight	Effective Unit Weight	Soil Strength		Lateral Pile Capacity Assessment		
				c	ϕ	Soil Type for p-y curve	k	ε_{50}
	(feet, NAVD88)	(pcf)	(pcf)	(psf)	(deg.)		(pci)	
Seawall Rock Dike	5 to -20	120	58	0	36	"Sand"	90	-
Young Bay Mud	-20 to -58	102	40	650 top - 1,100 bot.	-	"Soft Clay"	-	0.01
Upper Layered Sediments - Clay	-58 to -65	130	68	1,200	-	"Stiff Clay w/o Free Water"	-	0.007
Old Bay Clay	-65 to -107	110	48	1,500	-	"Stiff Clay w/o Free Water"	-	0.005
Lower Layered Sediments - Sand	Below -107	125	63	0	35	"Sand"	90	-

NOTES:

1. c = *cohesion,* ϕ = *friction angle,* k = *modulus of horizontal subgrade reaction for LPILE analysis, and* ε_{50} = *value of strain at 50% maximum stress for clays in LPILE analysis.*

Table 3-7: Preliminary LPILE Parameters for Lateral Pile Analysis – Sections 6 and 7

SECTIONS 6 AND 7
PRELIMINARY LPILE PARAMETERS FOR LATERAL PILE ANALYSIS

Modeled Soil Unit	Elevation	Total Unit Weight	Effective Unit Weight	Soil Strength		Lateral Pile Capacity Assessment		
				c	ϕ	Soil Type for p-y curve	k	ε_{50}
	(feet, NAVD88)	(pcf)	(pcf)	(psf)	(deg.)		(pci)	
Seawall Rock Dike	5 to -40	120	58	0	36	"Sand"	90	-
Young Bay Mud	-40 to -109	102	40	1,000 top - 1,700 bot.	-	"Soft Clay"	-	0.007
Upper Layered Sediments - Clay	-109 to -118	126	64	1,500	-	"Stiff Clay w/o Free Water"	-	0.005
Upper Layered Sediments - Sand	-118 to -136	132	70	0	40	"Sand"	125	-
Old Bay Clay	Below -136	126	64	2,000	-	"Stiff Clay w/o Free Water"	-	0.005

NOTES:

1. c *= cohesion,* ϕ *= friction angle,* k *= modulus of horizontal subgrade reaction for LPILE analysis, and* ε_{50} *= value of strain at 50% maximum stress for clays in LPILE analysis.*

Table 3-8: Preliminary LPILE Parameters for Lateral Pile Analysis – Sections 8a and 8b

SECTIONS 8a AND 8b
PRELIMINARY LPILE PARAMETERS FOR LATERAL PILE ANALYSIS

Modeled Soil Unit	Elevation	Total Unit Weight	Effective Unit Weight	Soil Strength		Lateral Pile Capacity Assessment		
				c	ϕ	Soil Type for p-y curve	k	ε_{50}
	(feet, NAVD88)	(pcf)	(pcf)	(psf)	(deg.)		(pci)	
Seawall Rock Dike	0 to -10	120	58	0	36	"Sand"	90	-
Seawall Sand Fill	-10 to -28	130	68	0	30	"Sand"	20	-
Seawall Sand Fill LIQUEFIED	-10 to -28	130	68	400	0	"Soft Clay"	-	0.04
Young Bay Mud	-28 to -98	102	40	850 top - 1,600 bot.	-	"Soft Clay"	-	0.007
Upper Layered Sediments - Clay	-98 to -118	132	70	2,000	-	"Stiff Clay w/o Free Water"	-	0.005
Upper Layered Sediments - Sand	-118 to -133	132	70	0	40	"Sand"	125	-
Old Bay Clay	Below -133	112	50	2,000	-	"Stiff Clay w/o Free Water"	-	0.005

NOTES:

1. ***c*** *= cohesion,* ϕ *= friction angle,* ***k*** *= modulus of horizontal subgrade reaction for LPILE analysis, and* ε_{50} *= value of strain at 50% maximum stress for clays in LPILE analysis.*

Table 3-9: Preliminary LPILE Parameters for Lateral Pile Analysis – Sections 8 and 9a

SECTIONS 8 AND 9a
PRELIMINARY LPILE PARAMETERS FOR LATERAL PILE ANALYSIS

Modeled Soil Unit	Elevation	Total Unit Weight	Effective Unit Weight	Soil Strength		Lateral Pile Capacity Assessment		
				c	ϕ	Soil Type for p-y curve	k	ε_{50}
	(feet, NAVD88)	(pcf)	(pcf)	(psf)	(deg.)		(pci)	
Seawall Rock Dike	5 to -38.5	120	58	0	36	"Sand"	90	-
Young Bay Mud	-38.5 to -95	102	40	950 top - 1,600 bot.	-	"Soft Clay"	-	0.007
Upper Layered Sediments - Sand	Below -95	132	70	0	40	"Sand"	125	-

NOTES:

1. ***c*** *= cohesion,* **ϕ** *= friction angle,* ***k*** *= modulus of horizontal subgrade reaction for LPILE analysis, and* $\boldsymbol{\varepsilon_{50}}$ *= value of strain at 50% maximum stress for clays in LPILE analysis.*

Table 3-10: Preliminary LPILE Parameters for Lateral Pile Analysis – Sections 9B and 9

SECTIONS 9b AND 9
PRELIMINARY LPILE PARAMETERS FOR LATERAL PILE ANALYSIS

Modeled Soil Unit	Elevation	Total Unit Weight	Effective Unit Weight	Soil Strength		Lateral Pile Capacity Assessment		
				c	ϕ	Soil Type for p-y curve	k	ε_{50}
	(feet, NAVD88)	(pcf)	(pcf)	(psf)	(deg.)		(pci)	
Seawall Rock Dike	-4 to -44	120	58	0	36	"Sand"	90	-
Young Bay Mud	-44 to -59	90	28	1,000 top - 1,200 bot.	-	"Soft Clay"	-	0.007
Upper Layered Sediments - Clay	Below -59	131	69	2,000	-	"Stiff Clay w/o Free Water"	-	0.005

NOTES:

1. ***c*** *= cohesion,* ϕ *= friction angle,* ***k*** *= modulus of horizontal subgrade reaction for LPILE analysis, and* ε_{50} *= value of strain at 50% maximum stress for clays in LPILE analysis.*

Table 3-11: Preliminary LPILE Parameters for Lateral Pile Analysis – Sections 10, 11a, 11, 12 and 13

SECTIONS 10, 11a, 11, 12 AND 13
PRELIMINARY LPILE PARAMETERS FOR LATERAL PILE ANALYSIS

Modeled Soil Unit	Elevation	Total Unit Weight	Effective Unit Weight	Soil Strength		Lateral Pile Capacity Assessment		
				c	φ	Soil Type for p-y curve	k	ε_{50}
	(feet, NAVD88)	(pcf)	(pcf)	(psf)	(deg.)		(pci)	
Seawall Rock Dike	2 to -27	120	58	0	36	"Sand"	90	-
Young Bay Mud	-27 to -33	90	28	800	-	"Soft Clay"	-	0.01
Upper Layered Sediments - Sand	-33 to -60 and -79 to -92	135	73	0	36	"Sand"	90	-
Upper Layered Sediments - Clay	-60 to -79	131	69	2,000	-	"Stiff Clay w/o Free Water"	-	0.005
Old Bay Clay	-92 to -112	112	50	2,000	-	"Stiff Clay w/o Free Water"	-	0.005
Lower Layered Sediments - Sand	Below -112	138	76	0	42	"Sand"	125	-

NOTES:

1. ***c*** *= cohesion,* φ *= friction angle,* ***k*** *= modulus of horizontal subgrade reaction for LPILE analysis, and* ε_{50} *= value of strain at 50% maximum stress for clays in LPILE analysis.*

Table 3-12: Preliminary LPILE Parameters for Lateral Pile Analysis – Section 46

SECTION 46
PRELIMINARY LPILE PARAMETERS FOR LATERAL PILE ANALYSIS

Modeled Soil Unit	Elevation (feet, NAVD88)	Total Unit Weight (pcf)	Effective Unit Weight (pcf)	Soil Strength *c* (psf)	Soil Strength ϕ (deg.)	Lateral Pile Capacity Assessment: Soil Type for p-y curve	k (pci)	ε_{50}
Seawall Rock Dike	8.5 to -8	120	58	0	36	"Sand"	90	-
Seawall Sand Fill	-8 to -35	130	68	0	30	"Sand"	20	-
Seawall Sand Fill LIQUEFIED	-8 to -35	130	68	400	0	"Soft Clay"	-	0.04
Upper Layered Sediments - Sand	-35 to -52	135	73	0	38	"Sand"	105	-
Old Bay Clay	-52 to -65	112	50	2,000	-	"Stiff Clay w/o Free Water"	-	0.005
Lower Layered Sediments - Sand	Below -65	138	76	0	42	"Sand"	125	-

NOTES:

1. ***c*** *= cohesion,* ϕ *= friction angle,* ***k*** *= modulus of horizontal subgrade reaction for LPILE analysis, and* ε_{50} *= value of strain at 50% maximum stress for clays in LPILE analysis.*

Axial Capacities of Driven Piles

We evaluated the axial compressive and axial tensile capacities of existing driven piles. The piles will gain their resistance in both side resistance along the length of the pile and in end bearing. The allowable axial capacities of the driven piles were evaluated using computer program APILE v2014 (Ensoft, 2014). We evaluated pile capacities by two analysis methods: American Petroleum Institute Recommended Practice 2A (RP2A) and Federal Highway Administration (FHWA) methods. The analysis results were similar with the values obtained by the RP2A method slightly less for the soil conditions at the site.

Based on these analyses, we generalized the frictional resistance and the end bearing capacity of the piles within each of the soil layers. Soil parameters to evaluate axial capacities of existing driven piles are provided for nine idealized soil profiles in ***Tables 3-15 through 3-23 – Preliminary Parameters for Axial Pile Analysis***. It should be noted that end bearing resistance is affected by soil layers within about ten pile diameters of the pile tip, so weaker (or stronger) layers within close proximity to the pile tip will affect the pile capacity and is not accounted for in this generalized approach. For piles in tension, 70 percent of the unit skin friction resistance in compression should be used and the end bearing resistance should be ignored. The uplift resistance afforded by the buoyant weight of the piles can be added at the discretion of the structural engineer. For a performance-based structural evaluation of the bulkhead wharf, the capacities provided require approximately ¼ inch to mobilize, and the ultimate end bearing capacity has been decreased by a factor of safety of about 2 to account for the larger pile movement required to mobilize end bearing than frictional resistance. The capacities are best estimates, and a sensitivity analysis with ½ to 2 times the recommended parameters should be performed to account for soil variability and uncertainty. For simplified structural evaluations, we recommend a factor of safety of 2 to evaluate allowable capacities of existing piles for static and normal duration live loads. Generally, lower factors of safety are allowed when considering short duration seismic loads, in which a factor of safety of 1.5 is recommended.

Piles that are spaced at least three pile diameters center to center will achieve 100 percent of their allowable axial capacities. Axial group reduction factors for allowable capacities can be provided for piles that are spaced more closely, upon request.

Table 3-13: Preliminary Parameters for Axial Pile Analysis – Section B

SECTION B
PRELIMINARY PARAMETERS FOR AXIAL PILE ANALYSIS

Modeled Soil Unit	Elevation (feet, NAVD88)	Total Unit Weight (pcf)	Effective Unit Weight (pcf)	Axial Pile Parameters in Compression [1] Unit Skin Friction (ksf)	Axial Pile Parameters in Compression [1] Unit End Bearing (ksf)
Seawall Rock Dike [2]	9 to -15	120	58	0.4	15
Marine Deposit - Sand	-15 to -20	132	70	0.65	15
Marine Deposit – Sand LIQUEFIED	-15 to -20	132	70	0.1	2.5
Young Bay Mud	-20 to -33	112	50	0.6	2.5
Upper Layered Sediments - Sand	-33 to -46	139	77	2.7	150
Upper Layered Sediments - Clay	-46 to -57	138	76	1.2	10
Upper Layered Sediments - Sand	Below -57	139	77	4.0	200

NOTES:
1. *For axial pile parameters in tension, use 70 percent of the unit skin friction resistance in compression and ignore the end bearing resistance.*
2. *Soils above a liquefied soil layer will impose downdrag loads on the pile after the seismic event.*

Table 3-14: Preliminary Parameters for Axial Pile Analysis – Section A and 1

SECTIONS A AND 1
PRELIMINARY PARAMETERS FOR AXIAL PILE ANALYSIS

Modeled Soil Unit	Elevation (feet, NAVD88)	Total Unit Weight (pcf)	Effective Unit Weight (pcf)	Axial Pile Parameters in Compression [1]	
				Unit Skin Friction (ksf)	Unit End Bearing (ksf)
Seawall Rock Dike	7 to -27	120	58	0.3	15
Marine Deposit - Clay	-27 to -34	105	43	0.5	2.5
Marine Deposit - Sand	-34 to -54	115	53	1.2	35
Upper Layered Sediments - Sand	-54 to -71	132	50	3.0	200
Old Bay Clay	Below -71	112	50	1.5	10

NOTES:

1. *For axial pile parameters in tension, use 70 percent of the unit skin friction resistance in compression and ignore the end bearing resistance.*

Table 3-15: Preliminary Parameters for Axial Pile Analysis – Sections 2, 3, 4 and 5

SECTIONS 2, 3, 4 AND 5
PRELIMINARY PARAMETERS FOR AXIAL PILE ANALYSIS

Modeled Soil Unit	Elevation	Total Unit Weight	Effective Unit Weight	Axial Pile Parameters in Compression [1]	
				Unit Skin Friction	Unit End Bearing
	(feet, NAVD88)	(pcf)	(pcf)	(ksf)	(ksf)
Seawall Rock Dike	5 to -20	120	58	0.55	15
Young Bay Mud	-20 to -58	102	40	0.6	2.5
Upper Layered Sediments - Clay	-58 to -65	130	68	1.1	5
Old Bay Clay	-65 to -107	110	48	1.2	10
Lower Layered Sediments - Sand	Below -107	125	63	3.2	100

NOTES:

1. *For axial pile parameters in tension, use 70 percent of the unit skin friction resistance in compression and ignore the end bearing resistance.*

Table 3-16: Preliminary Parameters for Axial Pile Analysis – Sections 6 and 7

SECTIONS 6 AND 7
PRELIMINARY PARAMETERS FOR AXIAL PILE ANALYSIS

Modeled Soil Unit	Elevation	Total Unit Weight	Effective Unit Weight	Axial Pile Parameters in Compression [1]	
				Unit Skin Friction	Unit End Bearing
	(feet, NAVD88)	(pcf)	(pcf)	(ksf)	(ksf)
Seawall Rock Dike	5 to -40	120	58	1.0	15
Young Bay Mud	-40 to -109	102	40	1.2	7.5
Upper Layered Sediments - Clay	-109 to -118	126	64	1.4	7.5
Upper Layered Sediments - Sand	-118 to -136	132	70	4.5	200
Old Bay Clay	Below -136	126	64	2.3	10

NOTES:

1. *For axial pile parameters in tension, use 70 percent of the unit skin friction resistance in compression and ignore the end bearing resistance.*

Table 3-17: Preliminary Parameters for Axial Pile Analysis – Sections 8a and 8b

SECTIONS 8a AND 8b
PRELIMINARY PARAMETERS FOR AXIAL PILE ANALYSIS

Modeled Soil Unit	Elevation	Total Unit Weight		Effective Unit Weight	Axial Pile Parameters in Compression [1]	
					Unit Skin Friction	Unit End Bearing
	(feet, NAVD88)	(pcf)		(pcf)	(ksf)	(ksf)
Seawall Rock Dike [2]	0 to -10	120		58	0.3	15
Seawall Sand Fill	-10 to -28	130		68	0.65	10
Seawall Sand Fill LIQUEFIED	-10 to -28	130		68	0.1	2.5
Young Bay Mud	-28 to -98	102		40	1.0	7.5
Upper Layered Sediments - Clay	-98 to -118	132		70	1.6	10
Upper Layered Sediments - Sand	-118 to -133	132		70	4.5	200
Old Bay Clay	Below -133	112		50	2.1	10

NOTES:
1. *For axial pile parameters in tension, use 70 percent of the unit skin friction resistance in compression and ignore the end bearing resistance.*
2. *Soils above a liquefied soil layer will impose downdrag loads on the pile after the seismic event.*

Table 3-181: Preliminary Parameters for Axial Pile Analysis – Sections 8 and 9

SECTIONS 8 AND 9a
PRELIMINARY PARAMETERS FOR AXIAL PILE ANALYSIS

Modeled Soil Unit	Elevation (feet, NAVD88)	Total Unit Weight (pcf)	Effective Unit Weight (pcf)	Axial Pile Parameters in Compression [1] Unit Skin Friction (ksf)	Unit End Bearing (ksf)
Seawall Rock Dike	5 to -38.5	120	58	0.95	15
Young Bay Mud	-38.5 to -95	102	40	1.0	5
Upper Layered Sediments - Sand	Below -95	132	70	3.2	200

NOTES:
1. *For axial pile parameters in tension, use 70 percent of the unit skin friction resistance in compression and ignore the end bearing resistance.*

Table 3-19: Preliminary Parameters for Axial Pile Analysis – Sections 9b and 9

SECTIONS 9b AND 9
PRELIMINARY PARAMETERS FOR AXIAL PILE ANALYSIS

Modeled Soil Unit	Elevation	Total Unit Weight	Effective Unit Weight	Axial Pile Parameters in Compression [1]	
				Unit Skin Friction	Unit End Bearing
	(feet, NAVD88)	(pcf)	(pcf)	(ksf)	(ksf)
Seawall Rock Dike	-4 to -44	120	58	1.1	15
Young Bay Mud	-44 to -59	90	28	0.75	2.5
Upper Layered Sediments - Clay	Below -59	131	69	1.2	10

NOTES:
1. *For axial pile parameters in tension, use 70 percent of the unit skin friction resistance in compression and ignore the end bearing resistance.*

Table 3-20: Preliminary Parameters for Axial Pile Analysis – Sections 10, 11, 11a, 12 and 13

SECTIONS 10, 11a, 11, 12 AND 13
PRELIMINARY PARAMETERS FOR AXIAL PILE ANALYSIS

Modeled Soil Unit	Elevation	Total Unit Weight	Effective Unit Weight	Axial Pile Parameters in Compression [1]	
				Unit Skin Friction	Unit End Bearing
	(feet, NAVD88)	(pcf)	(pcf)	(ksf)	(ksf)
Seawall Rock Dike	2 to -27	120	58	0.8	15
Young Bay Mud	-27 to -33	90	28	0.65	7.5
Upper Layered Sediments - Sand	-33 to -60	135	73	1.7	75
Upper Layered Sediments - Clay	-60 to -79	131	69	1.5	10
Upper Layered Sediments - Sand	-79 to -92	135	73	3.3	100
Old Bay Clay	-92 to -112	112	50	1.8	10
Lower Layered Sediments - Sand	Below -112	138	76	5.2	300

NOTES:

1. *For axial pile parameters in tension, use 70 percent of the unit skin friction resistance in compression and ignore the end bearing resistance.*

Table 3-21: Preliminary Parameters for Axial Pile Analysis – Section 46

SECTION 46
PRELIMINARY PARAMETERS FOR AXIAL PILE ANALYSIS

Modeled Soil Unit	Elevation	Total Unit Weight	Effective Unit Weight	Axial Pile Parameters in Compression [1]	
				Unit Skin Friction	Unit End Bearing
	(feet, NAVD88)	(pcf)	(pcf)	(ksf)	(ksf)
Seawall Rock Dike [2]	8.5 to -8	120	58	0.3	15
Seawall Sand Fill	-8 to -35	130	68	0.85	15
Seawall Sand Fill LIQUEFIED	-8 to -35	130	68	0.1	2.5
Upper Layered Sediments - Sand	-35 to -52	135	73	2.1	125
Old Bay Clay	-52 to -65	112	50	1.6	10
Lower Layered Sediments - Sand	Below -65	138	76	3.2	300

NOTES:

1. *For axial pile parameters in tension, use 70 percent of the unit skin friction resistance in compression and ignore the end bearing resistance.*
2. *Soils above a liquefied soil layer will impose downdrag loads on the pile after the seismic event.*

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