Geotechnical Evaluation

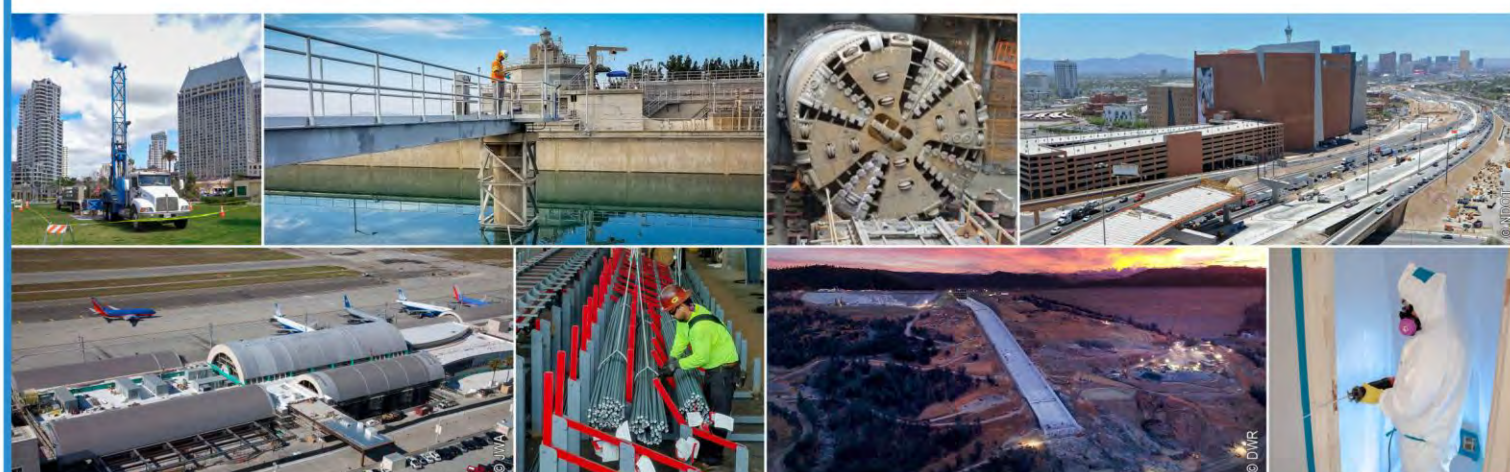
City of Alameda New Aquatic Center
Jean Sweeney Open Space Park

1100 Atlantic Avenue
Alameda, California 94501

City of Alameda

950 West Mall Square | Alameda, California 94501

May 30, 2025 | Project No. 403773009



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Ninyo & Moore

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1 INTRODUCTION

In accordance with your request and authorization, Ninyo & Moore has conducted a geotechnical evaluation for the proposed City of Alameda New Aquatic Center project at Jean Sweeney Open Space Park in Alameda, California (Figure 1). This report presents the findings from the subsurface exploration conducted for this evaluation, the conclusions from our review of geologic conditions at the site, and our geotechnical recommendations for the design and construction of the project.

2 SCOPE OF SERVICES

Our scope of services included the following:

- Review of readily available geologic and seismic literature pertinent to the project area including geologic maps and reports, regional fault maps, seismic hazard maps, water resources, and aerial photography.
- Performance of a site reconnaissance to mark the proposed subsurface exploration and percolation test locations and to observe general site conditions, including topographic features, drainage, and surficial geologic conditions and to review project limits and check equipment access.
- Coordination with Underground Service Alert (USA) to locate underground utilities in the vicinity of the subsurface exploration locations.
- Performance of a private utility survey to further check the exploration locations for potential conflicts with underground utilities.
- Procurement of Alameda County Public Works soil boring and well permits.
- Subsurface exploration consisting of ten cone penetrometer test (CPT) probes and four soil borings. The CPTs and borings were advanced to depths of up to about 53 feet and 50½ feet, respectively, below the existing grade. The CPTs and borings were backfilled with grout in accordance with the permit. Soil cuttings generated during drilling were drummed and off-hauled. The exploration locations are shown in Figure 2.
- Collecting of shear wave velocity measurements at 5-foot intervals at one cone penetration testing to evaluate the seismic site class.
- Installation of 2-inch diameter monitoring wells in two of the borings after geotechnical sampling to record groundwater levels.
- Percolation testing at two locations using the borehole method.
- Laboratory testing of selected soil samples to evaluate in-place soil moisture content and dry density, percentage of soil particles finer than the No. 200 sieve, Atterberg limits, consolidation characteristics, unconsolidated-undrained triaxial compressive strength, R-value, and soil corrosivity, as appropriate for the subsurface materials encountered.
- Data compilation and engineering analysis of the information obtained from our background review, subsurface evaluation, and laboratory testing.

- Preparation of this geotechnical evaluation report presenting our findings and conclusions regarding the subsurface conditions encountered at the project site, and our geotechnical recommendations for the design and construction of the project.

3 SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The new Alameda Aquatics Center will be located at the western end of Jean Sweeney Open Space Park, near the crossroads of Wilma Chan Way and Atlantic Avenue in Alameda, California (Figure 1). The proposed project site is bounded by residential homes to the north and south, commercial businesses to the northeast, food stores and parking lots to the immediate west, and relatively open space with paved walkways and trails which transitions to the more developed portion of the park. In addition, a chain-linked fence was observed on the south boundary of the project site. During our subsurface exploration, the site was an open space park with grass fields and pedestrian asphalt concrete paved access walking trails. The project site is characterized by a low degree of topographic relief with a ground surface elevation that ranges between approximately 6 to 15 feet (NAVD88) from southwest corner to northeast corner of the project site and a ground surface elevation that ranges between 8 to 12 feet from northwest to southeast corner of the project site (ELS Architecture+Urban Design, 2025). There is an approximately 3-foot-tall mound of dirt in the eastern portion of the project.

Based on our review of the design drawings (ELS Architecture+Urban Design, 2025), the new Alameda Aquatics Center will be a swim facility that features a one-story building designed to support a 30-meter competitive swimming pool, a smaller activity pool, and spectator seating areas adjacent to the pools (Figure 3). The Competition Pool depth will range from 3’-6” to 7’-0”, and the Activity Pool depth will range from zero to 5’-0”. The primary structure has an L-shaped layout, with the north wing housing essential spaces such as pool mechanical equipment, an electrical room, locker rooms, and a lifeguard room, while the south wing contains administration offices and a multipurpose room. These two wings are connected by a covered breezeway, which serves as the primary entry to the facility. A secondary structure will accommodate pool storage. Anticipated structure loads provided by the design team (DL plus LL) are 32 kips on column footings and 1618 pounds per linear foot (plf) on strip footings. Surrounding the perimeter of the pool deck is fencing that stands at a minimum height of 10 feet for security, with an increased height of 15 feet along the western side to provide necessary wind protection for the pools. This fencing consists of a combination of solid and semi-porous materials. Other associated improvements include construction of an entry plaza, bicycle parking, and new parking lot and installation of underground utilities.

We understand that rough grading will include placement of up to about three feet of fill in the area of the proposed pools and structures, and cuts and fills are anticipated to generally balance the site as much as possible. Based on the preliminary pool depths, we anticipate the swimming pools will require excavations on the order of 10 to 12 feet deep. Excavations will be required for foundations. The mound of dirt in the eastern portion of the project will need to be removed.

We understand the settlement tolerance for the planned buildings is 1 inch maximum and a differential of ¼ inch over a distance of 30 feet. The planned pools and associated piping and surge chamber need to settle relatively uniformly with a goal of achieving a maximum differential settlement of ½ inch over the length/width of the pool. The goal is to also reduce differential settlement in pool decks as well as impacts to utilities entering/exiting the pools. Based upon engineering analyses and communications with the design team, we understand the desired goal of ½ inch differential across the length/width of the pool (80 to 100 feet) under combined static and dynamic settlement was deemed costly and may not be possible.

4 SUBSURFACE EVALUATION AND LABORATORY TESTING

4.1 Subsurface Exploration

Our field exploration included a site reconnaissance and subsurface exploration of the project site. The initial subsurface exploration was conducted on December 5th and 6th, 2023, and consisted of five CPT soundings (CPT-1 through CPT-5) and four exploratory borings (B-1 through B-4), two of which were completed as monitoring wells. Infiltration tests were performed at I-1 and I-2 locations. Supplemental subsurface exploration was conducted on October 23, 2024, and consisted of five CPT soundings (CPT-6 through CPT-10). The approximate locations of the CPT soundings, borings, monitoring wells, and infiltration tests are shown on Figure 2.

4.1.1 Cone Penetration Testing

The CPT soundings were advanced to depths up to about 53 feet below ground surface (bgs) using a truck-mounted rig with 20-ton reaction capacity. Penetration and pore water pressure data were collected and recorded electronically at intervals of approximately 2 inches while the sounding was being conducted. The soil behavior type index (I_c) of the materials encountered were assessed using correlations (Robertson, 2010) based on the cone penetration data including tip resistance and sleeve friction penetration. Shear wave velocity measurement at 5-foot intervals during cone penetration testing was collected at CPT-2 to assist with evaluation of the seismic site classification. The logs of CPT data and the interpreted soil behavior types are presented in Appendix A.

4.1.2 Geotechnical Borings

The exploratory borings were advanced with a truck-mounted drill rig using hollow-stem augers to depths up to about 50½ feet below the existing grade. One of the borings was located near a CPT to assist with correlation of subsurface data. A representative of Ninyo & Moore logged the subsurface conditions exposed in the borings and collected bulk and relatively undisturbed soil samples from the borings for laboratory testing. Soil was field-classified in accordance with the Unified Soil Classification System (USCS) using the visual-manual procedures in Standard D2488 by the American Society for Testing and Material (ASTM). Descriptions of the subsurface materials encountered are presented in the following sections. Detailed logs of the borings and sampling procedures are presented in Appendix B. The collected samples were transported to our geotechnical laboratory for testing. The borings were backfilled with cement grout shortly after drilling. The soil cuttings generated during drilling were drummed and off-hauled.

Two of the borings (B-1 and B-3) were completed as monitoring wells to maximum depths of approximately 26 feet bgs. Refer to Section 5.5 for additional information regarding monitoring well construction.

4.2 Laboratory Testing

Geotechnical laboratory testing of soil samples recovered from the borings included in-situ soil moisture content and dry density, percentage of soil particles finer than the No. 200 sieve, Atterberg limits, consolidation characteristics, unconsolidated-undrained triaxial compressive strength and R-value. The results of the in-place moisture content and dry density tests are shown at the corresponding sample depths on the boring logs in Appendix B. The results of the remaining laboratory tests performed are presented in Appendix C.

It was requested that we provide the average soil unit weight for the upper 14 feet of soil for the design of the pool. The table below summarizes the average soil unit weights based on our laboratory testing and CPT data.

Table 1 – Average Soil Unit Weight

General Description	Layer Depth (feet)	Average Wet Density (pcf)	Assumed Moisture Content (%)	Dry Density (pcf)
Fill	0 – 5	121	10	110
Bay Mud	5 – 14	100	55	65

Additionally, one sample of the near-surface soil was sent to CERCO Analytical (CERCO) in Concord, California for corrosivity analysis. The results of this testing, including a brief evaluation, are presented in Appendix D, and are discussed in Section 6.8.

4.3 Infiltration Testing

The permeability of the near surface soils was evaluated at two locations as shown on Figure 2. Infiltration testing was performed on December 6, 2023. Borehole percolation testing was performed in general accordance with the Alameda County Department of Environmental Health Onsite Wastewater Treatment Manual (2018) at depths of about 3 feet below the ground surface. The test holes were backfilled with soil cuttings and tamped using the drill rig shortly after completion of test.

The percolation test procedures and test data are presented in Appendix E and the test results are listed in Table 2 below. The test results at the two tested locations indicate infiltration rate of 0 inches per hour, which is considered low. Due to the variability of subsurface materials encountered during our exploration, variability in subsurface infiltration should be anticipated.

Table 2 – Percolation Test Results

Test (Boring)	Test Depth (ft.)	Subsurface Conditions	Percolation Rate (inch/hour)	Infiltration Rate[1] (inch/hour)
I-1	3.0	Lean Clay	0.0	0.0
I-2	3.0	Lean Clay	0.0	0.0

Note:
[1] Infiltration rate is percolation rate adjusted by a reduction factor to exclude percolation through sides of test hole.

5 GEOLOGIC AND SUBSURFACE CONDITIONS

Our findings regarding regional geologic setting, site geology, subsurface stratigraphy, and groundwater conditions at the subject site are provided in the following sections.

5.1 Regional Geologic Setting

The subject site is on the eastern margin of San Francisco Bay in the Coast Ranges geomorphic province of California. The Coast Ranges are comprised of several mountain ranges and structural valleys formed by tectonic processes commonly found around the Circum-Pacific belt. Basement rocks have been sheared, faulted, metamorphosed, and uplifted, and are separated by thick blankets of Cretaceous and Cenozoic sediments that fill structural valleys and line continental margins. The San Francisco Bay Area has several ranges that trend northwest, parallel to major strike-slip faults such as the San Andreas, Hayward, and Calaveras (Figure 4).

Major tectonic activity associated with these and other faults within the region consists primarily of right-lateral, strike-slip movement.

5.2 Site Geology

Regional geologic mapping indicates that the project site is underlain by artificial fill (Graymer, 2000). The material used as fill varies in composition depending upon the source of the material. Per Graymer (2000), some fills are compacted and quite firm, but fills placed before 1965 are typically loose and poorly compacted consisting of dumped materials.

Graymer (2000) maps the natural portions of Alameda Island as Dune Sand from the Holocene and Pleistocene, with deposition likely ending around 6,000 years ago. Radbruch (1957) classifies it as Merritt sand. The material is described as fine-grained, very well sorted (poorly graded), well drained eolian deposits, mainly occurring in large sheets or small hills. Graymer distinguishes the two units based on morphology, stating that the Merritt Sand displays yardang morphology. In both cases, the sand interfingers with Holocene Bay Mud deposits.

As described by Graymer (2000), Bay Mud consists of water saturated estuarine mud, predominantly gray, green, and blue clay and silty clay. The mud interfingers with and grades into fine-grained deposits at the distal edges of Holocene fans. Mapping of thickness of the Young Bay Mud deposit in the Bay Area (McDonald et. al., 1978) indicates the Bay Mud thickness at the project site is between zero and 20 feet with the southern limit of Bay Mud near the southern limit of the undeveloped area.

The findings of our subsurface exploration, described below, indicate that the site is underlain by fill and Young Bay Mud. Underlying the Young Bay Mud are Dune Sand and Old Bay Mud, both of which include an interlayered soil profile including sand, clay and silt mixtures. A regional geologic map for the site and surrounding area is presented on Figure 5.

5.3 Subsurface Conditions

The following sections provide a generalized description of the geologic units encountered during our subsurface evaluation at the project site. More detailed descriptions are presented on the CPT logs in Appendix A and boring logs in Appendix B.

5.3.1 Artificial Fill

Artificial fill was encountered in the borings and CPTs at the surface and extended to depths varying from about 3 to 9 feet below existing grade. As encountered in the borings, the artificial fill generally consisted of brown and gray, moist to wet, loose to dense silty and clayey sand and grayish brown, moist, very stiff sandy lean clay. The behavior index of the fill as encountered in the CPT soundings indicate the fill generally consists of silty and clayey sand and clay.

5.3.2 Young Bay Mud

Young Bay Mud was encountered below the artificial fill in Borings B-1, B-2, B-3, and in the CPT soundings. The Young Bay Mud varies in thickness between 8 to 13 feet with the bottom of the Young Bay Mud varying up to about 19 feet below existing grade. In general, the Young Bay Mud as encountered in the borings consisted of black, gray, and bluish-gray, moist to wet, soft to stiff clay, and medium dense, silty sand. The behavior index of the Young Bay Mud as encountered in the CPT soundings indicate that the Bay Mud generally consisted of clay and silty clay and silty and clayey sand. As encountered in the subsurface exploration, the soft clay portion of the Young Bay Mud is less than 10 feet thick and bottomed about 12 to 14 feet below grade.

5.3.3 Dune Sand

Dune Sand was encountered below the Young Bay Mud in Borings B-1, B-2, B-3 and in the CPT soundings. The Dune Sand, as encountered in the borings extended to a maximum depth of 39 feet below existing grade. The Dune Sand, as encountered in the borings, generally consisted of olive brown and brown, wet, loose to very dense, silty and clayey sand. As encountered in the CPT soundings, the Dune Sand generally consisted of interlayered silty clay, clayey silt, and silty sand.

5.3.4 Old Bay Mud

Old Bay Mud was encountered below the Dune Sand in Boring B-1 at a depth of about 39 feet below existing grade. Based on CPT soundings, we anticipate that Old Bay Mud is present; however, it is difficult to distinguish between the Dune Sand and Old Bay Mud. In general, the Old Bay Mud as encountered in the boring consisted of bluish-gray, wet, dense to very dense silty sand.

5.3.5 CPT Summary

The CPT data recorded in CPT-1 is generally consistent with the subsurface materials encountered in Boring B-1 which is close to CPT-1.

In general, the CPT data indicates materials of low tip resistance and/or low friction, consistent with low strength clayey or organic and loose sandy materials, extending to depths up to about 18 feet below existing grade. The subsurface materials below this upper weaker zone generally consist of interbedded fine-grained and sandy layers. The CPT soundings 1 through 5, 7 and 9 encountered refusal at depths varying from about 40 to 53 feet below grade suggesting the presence of dense zones or inclusions which are restrictive to penetration.

5.4 Groundwater

Groundwater was encountered during our subsurface exploration at depths ranging between 5 feet and 13¾ feet below the ground surface in the CPT soundings, and approximately between 13 feet and 14 feet below the ground surface in the borings. Groundwater may rise to a higher elevation than was encountered in the exploratory borings due to the short time available for seepage of water into the boring. Regional groundwater records compiled by the California Geological Survey (CGS, 2003a), indicate that the historic high groundwater level at the site is about 5 feet below the ground surface. Groundwater monitoring at the site during the period between January 2024 to March 2025 indicated groundwater levels within the monitoring wells varied from about 3.5 to 5.0 feet below grade (See Section 5.5).

Variations or fluctuations in the groundwater levels across the site and over time may occur due to seasonal precipitation, spatial variations in topography or subsurface hydrogeologic conditions, or as a result of tidal variations or other factors. In addition, seeps may be encountered at elevations above the groundwater levels encountered due to perched groundwater conditions, leaking pipes, preferential drainage, or other factors not evident at the time of our exploration. Monitoring wells were installed to further evaluate the depth to groundwater in the study area and fluctuation in groundwater levels.

5.5 Groundwater Monitoring and Waste Disposal

5.5.1 Monitoring Well Installation and Monitoring

On December 6, 2023, Ninyo & Moore personnel oversaw the installation of groundwater monitoring wells MW-1 and MW-2 in geotechnical borings B-1 and B-3 at the site, respectively. Following soil sampling and lithologic logging, the wells were constructed with 2-inch diameter schedule 40 polyvinyl chloride (PVC) blank casing and 0.010-inch slotted PVC well screen. The screened intervals for the wells are 20 feet in length, extending from approximately 6 to 26 feet bgs. Well filter packs consist of #2/12 sand placed within the annulus of each boring from the bottom of each boring to approximately 1 foot above the top of each well screen, followed by an approximate 2 foot well transition seal consisting of bentonite. The remaining open borehole annulus in each well was sealed with neat cement to near ground surface. The boring logs are presented in Appendix B. Upon well completion, each wellhead was finished at the ground surface with a locking well cap and traffic-rated bolt-down well vault. The vaults were installed and finished with a concrete apron.

Groundwater levels within the monitoring wells varied from about 3.5 to 5.0 feet below grade during the period between January 2024 to March 2025 as indicated in Table 3.

Table 3 – Measured Groundwater Depth from Monitoring Wells		
Date	**Groundwater Depth at MW-1 (ft.)**	**Groundwater Depth at MW-2 (ft.)**
January 15, 2024	3.9	3.5
April 29, 2024	4.5	3.5
June 26, 2024	5.0	4.5
August 1, 2024	5.0	4.5
September 4, 2024	5.0	4.8
October 21, 2024	5.0	5.0
March 25, 2025	4.5	3.5

5.5.2 Waste Disposal

Investigation-derived waste (IDW) was temporarily stored on-site in 55-gallon, U.S. Department of Transportation–approved 17H drums, pending characterization and disposal. The IDW was characterized in accordance with waste disposal and recycling facility acceptance requirements. Ninyo & Moore coordinated the transport and disposal of the IDW with Belshire Environmental Services (Belshire) of Lake Forest, California, an appropriately licensed transporter to an approved waste facility. The waste was transported offsite by Belshire on December 29, 2023 to Soil Safe of Adelanto, California for disposal. A copy of the soil disposal certificate is attached in Appendix F.

6 GEOLOGIC HAZARDS AND GEOTECHNICAL CONSIDERATIONS

This study considered a number of potential issues relevant to the proposed project including seismic hazards, flooding and dam failure inundation, landsliding and slope stability, naturally occurring asbestos, collapsible soil, regional land subsidence, consolidation and static settlement, corrosive soil, expansive soil, and excavation characteristics. These issues are discussed in the following subsections.

6.1 Seismic Hazards

The seismic hazards considered in this study include the potential for ground surface fault rupture, seismic ground shaking, liquefaction and cyclic softening, dynamic settlement, sand boil ground subsidence, lateral spreading, seismic slope stability, and tsunamis and seiches. These potential hazards are discussed in the following subsections.

6.1.1 Historical Seismicity

The site is located in a seismically active region, as is much of northern California. Figure 4 presents the location of the site relative to the epicenters of historic earthquakes with magnitudes of 5.5 or more from 1800 to 2000. Records of historic ground effects related to seismic activity (e.g. liquefaction, sand boils, lateral spreading, ground cracking) compiled by Knudsen et al. (2000), indicate that no ground effects related to historic seismic activity have been reported for the site vicinity.

6.1.2 Ground Surface Fault Rupture

The site is not located within an Earthquake Fault Zone established by the state geologist (CGS, 1982) (formerly known as Alquist-Priolo Special Studies Zones) to delineate regions of potential ground surface rupture adjacent to active faults. As defined by the CGS, active faults are faults that have caused surface displacement within Holocene time, or within approximately the last 11,700 years (CGS, 2018). The closest fault rupture hazard zone is associated with the Hayward Fault. This hazard zone is approximately 5½ miles from the site to the northeast.

Based on our review of the referenced seismic hazard and geologic maps, known active faults are not mapped on the site, and the site is not located within a fault-rupture hazard zone. Therefore, the probability of damage from surface fault rupture is considered to be low.

6.1.3 Seismic Ground Motion and Site Classification

Considering the proximity of the site to historic and Holocene active faults (Figure 4), the potential for future strong seismic ground shaking at the site is significant. Seismic design criteria to address ground shaking are provided in Section 8.1. The peak ground acceleration (PGA) associated with the Maximum Considered Earthquake Geometric Mean (MCEG) was calculated in accordance with the American Society of Civil Engineers (ASCE) 7-16 Standard and the 2022 California Building Code (CBC). The MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) was calculated as 0.724g using the seismic design tool developed by the Structural Engineers Association of California in conjunction with the Office of Statewide Health Planning and Development (SEAOC & OSHPD, 2023). The calculated PGA_M is based a mapped MCE_G peak ground acceleration of 0.658g for the site and a site coefficient (F_{PGA}) of 1.1 for Site Class D. We performed one seismic CPT sounding at the site (CPT-2) that was terminated at a depth of about 52½ feet due to cone refusal. A shear wave velocity profile with respect to depth was obtained from this sounding that yielded an average shear wave velocity of about 940 feet per second. This shear wave velocity value correlates to a seismic Site Class D for the subject site (in accordance with ASCE 7-16). Considering the anticipated deep soil profile at the site, regional mapped Vs30 values (CGS, 2015), and soft clay soils less than 10 feet thick, we judge that Site Class D is appropriate for the project site.

6.1.4 Liquefaction and Cyclic Softening

The strong vibratory motions generated by earthquakes can trigger a rapid loss of shear strength in saturated, loose, granular soils of low plasticity (liquefaction) or in wet, sensitive, cohesive soil (cyclic softening). Liquefaction and strain softening can result in a loss of foundation bearing capacity or lateral spreading of sloping or unconfined ground. Liquefaction can also cause settlement of buildings on shallow foundation and generate sand boils leading to subsidence at the ground surface. Liquefaction (or cyclic softening) is generally not a concern at depths more than 50 feet bgs.

The project site is located within a liquefaction hazard zone (Figure 6) established by the California Geological Survey (CGS, 2003). Regional studies of liquefaction susceptibility indicate that the site is in an area considered to be very highly susceptible to liquefaction (Witter et al., 2006, and Knudsen et al., 2000).

Ninyo & Moore performed an analysis to evaluate the potential for liquefaction and cyclic softening using the CPT data collected during the subsurface exploration for this study and the methodology presented in Boulanger & Idriss (2014) and Robertson (2009) for liquefaction and cyclic softening, respectively. A groundwater level approximately 5 feet below the ground surface was assumed for the analysis which considered a seismic event producing a PGA of 0.724g resulting from a Magnitude 7.51 earthquake on the Hayward Fault (USGS, 2014). Corrections for thin layers and transitions zones were applied to account for reductions in measured penetration resistance where thin layers of sand are underlain by clay. Based on a comparison of borings and CPT soundings that were performed in close proximity to one another, material that was identified as having a behavior type index (Ic) of between 2.4 or 2.6 in the soundings generally correlated with sand, clay, and silt mixtures. A soil behavior type index (I_c) of 2.6 was selected as the cutoff for the liquefaction analysis. Soil with an I_c exceeding 2.6 was evaluated for cyclic softening. The results of the analysis, presented in Appendix G, indicate that layers of sandy soil within the artificial fill, Young Bay

Mud, and Dune Sand will liquefy to depths varying from about 5 to 34 feet below existing grade, and the weaker clayey Young Bay Mud layers will be subject to cyclic softening under the considered ground motion based on a computed factor of safety of less than one. High tip resistance and skin friction were encountered below depths of 34 feet, and refusal was encountered in CPT soundings 1 through 5, 7, and 9 at depths ranging between approximately 40½ and 53 feet below the ground surface. Furthermore, based on soil behavior type and calculated $N_{1(60)}$ from the CPT data, we judge the risk of liquefaction is low for subsurface materials below a depth of 34 feet.

Foundation type selection to mitigate liquefaction concerns and cyclic softening is provided in Section 8.3.1. Other consequences of liquefaction, including dynamic settlement, sand-boil-induced ground subsidence, and lateral spreading, are addressed in the following sections.

6.1.5 Dynamic Settlement

Earthquake ground shaking can dynamically compact loose granular soil leading to surficial settlements. Dynamic or volumetric-induced settlement is not limited to the near surface environment and may occur in both dry and saturated sandy soil. Cohesive soil is not typically susceptible to dynamic settlement.

Ninyo & Moore evaluated the potential for dynamic settlement due to strong ground motion of dry sandy soil and liquefaction of saturated soil using the computer program CLiq (GeoLogismiki, 2018) to evaluate the CPT data collected during our field investigation. The analysis estimates dynamic settlement due to seismic shaking for sandy soil layers with an I_c of 2.6 or less using the CPT data from the subsurface exploration for this study and methods by Boulanger & Idriss (2014) for saturated soil subject to liquefaction and by Robertson & Shao (2010) for dry sand settlement. A groundwater level approximately 5 feet below the ground surface was assumed for the analysis which considered a seismic event producing a PGA of 0.724g resulting from a Magnitude 7.51 earthquake. The results of the analysis indicate that the estimated total free-field dry and saturated dynamic settlement at the site following the considered seismic event is on the order of about 1¾ to 3½ inches occurring within the upper 5 to 34 feet below existing grade. Differential dynamic settlement is estimated to be up to about 2 inches over a horizontal distance of approximately 50 feet.

6.1.6 Sand Boil Ground Subsidence

In addition to dynamic or volumetric-induced settlement, sand boils that occur when liquefied, near-surface soil escapes to the ground surface, can result in additional ground subsidence due to loss of material. Ishihara (1985) concluded based on case study findings that the potential for surface manifestation of liquefaction or sand boils is low where the SPT penetration resistance of soil susceptible to liquefaction exceeds 10 blows per foot. Ninyo & Moore calculated the Ishihara-inspired Liquefaction Potential Index (LPI) described by Maurer et al (2015) from the results of the liquefaction analysis for liquefiable soil with an equivalent ratio SPT penetration resistance for 60 percent energy ratio (N_{60}) of 10 or less based on the CPT data near the proposed improvements to evaluate the potential for surface manifestation of liquefaction such as sand boils. The computed value of the index is approximately 9 to 18 which indicates that the potential for surface manifestation of liquefaction or sand boils is moderate to high. Foundation type selection to mitigate sand-boil ground subsidence is provided in Section 8.3.1.

6.1.7 Lateral Spreading

In addition to vertical displacements, seismic ground shaking can induce horizontal displacements as surficial soil deposits spread laterally by floating atop liquefied subsurface layers. Lateral spread can occur on sloping ground or on flat ground adjacent to an exposed face. Lateral spreading is generally not significant where the clean-sand-equivalent, overburden normalized, and corrected SPT penetration resistance (N_{160cs}) of the soil that can liquefy is more than 15 blows per foot (Caltrans, 2020). Based on the results of our subsurface exploration and liquefaction analysis, layers of liquefiable soil with an equivalent N_{160cs} less than 15 are present at depths ranging between approximately 5 and 19 feet below the ground surface.

Ninyo & Moore calculated the Lateral Displacement Index (LDI) as described by Zhang et al (2004) from the CPT data near the proposed improvements for liquefiable soil layers based on the liquefaction analysis with an N_{160cs} of 15 or less. Estimates of lateral spread displacement based on the computed LDI at each CPT sounding for an average ground slope of 0.7 percent toward San Francisco Bay to the southwest, have been performed without consideration for laterally discontinuous liquefiable soil layers between soundings or constraints resulting from adjacent down gradient areas where less lateral displacement is expected. The estimated lateral spread displacement of up to approximately 1 inch from our analysis conforms with the upper limit on lateral displacement for shallow foundations in Table 12.13-2 of ASCE 7-16 on projects with a Risk Category of I, II, or III. Foundation type selection is provided in the recommendation section to mitigate differential lateral spread displacement for structural elements.

6.1.8 Seismic Slope Stability

The site is not located within a hazard zone for earthquake-induced landslides on the Seismic Hazard Zones Map (Figure 6) prepared by the CGS (2003). As such, we do not regard seismic slope stability as a design consideration. Slope stability and landsliding are further addressed in Section 6.3.

6.1.9 Tsunamis and Seiches

Tsunamis are long wavelength seismic sea waves (long compared to ocean depth) generated by the sudden movements of the ocean floor during submarine earthquakes, landslides, or volcanic activity. The project location is within a tsunami hazard area as shown on the Tsunami Hazard Area Map for the County of Alameda (Kate Thomas, CGS, 2021). The Tsunami Inundation Map for the project site is presented on Figure 7.

Seiches are waves generated in a large enclosed body of water. Based on the lack of large enclosed bodies of water adjacent to the site, the potential for inundation due to seiches is not a consideration but the site may be subject to inundation by a tsunami.

6.2 Flood Hazards and Dam Failure Inundation

Our review of Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FEMA, 2018) indicates that the project site location is in a Future Conditions 1% Annual flood hazard area (Zone X) (Figure 8).

Properties located downstream of dams can be inundated with flood waters if the dams were to fail. Dam owners are required to prepare inundation maps showing the limits of flooding caused by dam failure. Our review of dam breach inundation maps on file with the Division of Safety of Dams at the California Department of Water Resources (DWR, 2020) indicates that the site is outside the inundation boundaries on the available inundation maps and therefore the potential for inundation at the site due to dam failure is very low. Figure 9 presents the Dam Inundation Map for a conjectured sunny day breach of the Central dam about 2.9 miles northeast of the site.

6.3 Landsliding and Slope Stability

The site in not within a seismic hazard zone for earthquake-induced landslides (Figure 6) as mapped by the State Geologist (CGS, 2021) and no existing landslides are mapped on the site based on the regional inventory in the seismic hazard zone report (CGS, 2003a). The site and vicinity are relatively flat with low topographic relief of about 1 foot on site and an average slope gradient of approximately 2.3 percent. No significant slopes are proposed for the project under consideration. Based on the existing topography and our review of existing maps and literature, we do not regard landslides or slope stability as design considerations for the proposed project.

6.4 Unsuitable Materials

Fill materials that were not placed and compacted in lifts with geotechnical observation and testing, or fill materials lacking documentation of such observation and testing, are considered non-engineered or undocumented fill. Non-engineered or undocumented fill is generally unsuitable as a bearing material below foundations and new fill due to the potential for differential settlement resulting from variable support characteristics or the potential inclusion of deleterious materials. The artificial fill encountered in our exploratory borings at the proposed project improvement areas are considered undocumented and may include or cover unsuitable material.

Recommendations for remedial grading are provided to mitigate surficial concerns related to undocumented fill and weak or soft soils at shallow depth which may impact access for construction equipment.

6.5 Collapsible Soil

Loose, dry, low-density soil can “collapse” or compact with the addition of water under foundation loads or the weight of overlying soil. Ground settlement occurs when the collapsible soil is first saturated or is saturated to depths greater than those achieved by typical rain events. Non-engineered or undocumented fill, young alluvial fans, debris flow sediments, and deposits of wind-blown soil may include collapsible soils, particularly in arid or semi-arid environments. The subsurface conditions encountered during the exploration indicate that collapsible soil is not a consideration for the site. If present, foundation type selection and remedial pad grading are provided in the recommendation section to mitigate differential settlement resulting from collapsible soils.

6.6 Regional Land Subsidence

The site is not in an area known for regional land subsidence due to groundwater withdrawal, peat loss, or oil extraction (USGS, 2018). As such we do not regard land subsidence as a design consideration.

6.7 Static Settlement and Settlement Mitigation

Based on the information available regarding the proposed project improvements, anticipated loads, and the subsurface conditions encountered, we anticipate the following considerations related to static settlement.

Considering the relatively low density of Bay Mud, the weight of an 7-foot deep pool is estimated to induce an additional new load on the order of 170 pounds per square foot (psf) resulting in estimated static settlement of the pools underlain by unmitigated soil on the order of several inches.

The estimated fill placement of uniform new aerial fill of about 3 feet is estimated to induce static settlement of several inches due to consolidation of the un-mitigated Young Bay Mud. The amount of settlement will depend upon the thickness of new fill and the variability of the subsurface materials below the new fill. The static settlement due to anticipated new building loads (32 kips on isolated footings and 1618 plf on strip footings (DL + LL) on un-mitigated subsurface materials is estimated to be on the order of 1 inch or more assuming an allowable bearing capacity of 1500 psf on isolated or continuous footings or 200 psf average load over the footprint of a reinforced concrete mat foundation.

As previously indicated, liquefaction densification settlement of un-mitigated subsurface materials is estimated to be on the order of about 1¾ to 3½ inches occurring within the upper 5 to 34 feet below existing grade. Differential dynamic settlement is estimated to be up to about 2 inches over a horizontal distance of approximately 50 feet for un-mitigated soils.

Because of the compressibility and liquefaction susceptibility of the subsurface materials and the estimated total settlements anticipated from new loads and seismic densification, we recommend deep foundations and ground improvement, or a combination of these to mitigate settlement of proposed pools and structures. Ground improvement can be implemented in areas adjacent to new structures or the pools to mitigate differential settlement which may impact utilities or abrupt differential movement between mitigated and un-mitigated areas.

For ground improvement at the project site to reduce total and differential settlements, we recommend that ground improvement could include rigid inclusion, deep soil mixing (DSM), drilled displacement columns (DDC), stone columns, aggregate piers, pressure grout, or other ground improvement methods that achieve the performance requirements of the project. The selection of ground improvement method should take into consideration the shallow groundwater, Young Bay Mud deposits which typically experience strength loss due to remolding, sandy and/or weak soil layers which may be prone to caving, and sandy layers which are locally dense and may be encountered above the target depth of mitigation.

The ground improvement should extend to a minimum depth of 20 feet below the existing ground surface to mitigate total static settlement to ½ inch. Seismic settlement below depth of 20 feet is up to 2.6 inches with differential static plus seismic settlement up to about 2 inches over 50 feet. Ground improvement to a depth of 34 feet reduces total static plus seismic settlement to about 1 inch with differential settlement up to about ½ inches over 50 feet. We anticipate that the ground improvement elements themselves would typically be heavier that the existing soils and are anticipated to induce settlement of the subsurface materials below the ground improvement. Ground improvement may consider the use of light-weight materials to reduce the settlement induced by the ground improvement elements. Design of the ground improvement system should also consider the drag loads induced by consolidation and seismic settlement.

6.8 Corrosive/Deleterious Soil

Laboratory testing was performed to evaluate the corrosivity of on-site soil. A soil sample collected during the subsurface exploration was submitted to CERCO Analytical of Concord, California to perform laboratory testing and evaluate the corrosivity of the samples on the basis of tests to quantify pH, redox potential, electrical resistivity, chloride content, and soluble sulfate content. The results of the testing and the findings from the corrosivity evaluation are presented in Appendix D.

California Department of Transportation (Caltrans) defines a corrosive environment for structures as an area where the soil has a chloride concentration of 500 parts per million (ppm) or greater, soluble sulfate concentration of 0.15 percent (1,500 ppm) or greater, or a pH of 5.5 or less (Caltrans, 2021). The criteria used to evaluate the deleterious nature of soil on concrete are listed in Table 4. Based on these criteria and the results of the testing, the near-surface soil as encountered at the tested location does not meet the definition of a corrosive environment for structures, and the sulfate exposure to concrete is negligible with an exposure classification for sulfate of S0. As noted in Appendix D, the test results indicate that the tested soil is moderately corrosive to ferrous metals based on the resistivity test results and slightly corrosive based on the redox potential. Buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel should be appropriately protected against corrosion depending on the importance or expected service life of the element. Additional corrosivity testing may be needed, and a corrosion engineer may be consulted to provide recommendations to mitigate corrosion. Please refer to the CERCO Analytical report included in Appendix D for more information regarding their test results and brief evaluation.

Table 4 – Criteria for Deleterious Soil on Concrete		
Sulfate Content Percent by Weight	Sulfate Exposure	Exposure Class
0.0 to 0.1	Negligible	S0
0.1 to 0.2	Moderate	S1
0.2 to 2.0	Severe	S2
> 2.0	Very Severe	S3

Reference: American Concrete Institute (ACI) Committee 318 Table 19.3.1.1 (ACI, 2021)

6.9 Expansive Soil

Some clay minerals undergo volume changes upon wetting or drying. Unsaturated soils containing those minerals will shrink/swell with the removal/addition of water. The heaving pressures and differential movement associated with expansion and changes in soil moisture can damage structures and flatwork. Laboratory testing performed on selected soil samples indicated plasticity index values generally ranging between 4 and 13, indicating a low potential for swell/expansion. However, one laboratory result for a soil sample at location B-3 obtained at a depth of about 6 feet (within the Young Bay Mud) indicated a plasticity index of 54 which corresponds to critically high expansion potential. The Young Bay Mud is generally located below the groundwater where the risk of shrink/swell effects is low.

We would like to point out that, historically, Bay Mud was sometimes used in fill operations, and there is a possibility that expansive Bay Mud may be encountered within the artificial fill. Note that chemical treatment is often used to reduce expansive characteristics.

6.10 Excavation Characteristics

We anticipate that the project will involve excavations of up to approximately 10 feet for the proposed swimming pools. The geologic units encountered over this interval during the subsurface exploration included artificial fill and Young Bay Mud that generally consisted of loose to medium dense sandy soils and soft to stiff clays.

We anticipate that conventional earthmoving and foundation drilling equipment in good working condition should be able to make the proposed excavations; however, consideration should be given to using light-weight and/or track equipment when overexcavating existing fills as conventional rubber-tired equipment may induce pumping, rutting, or bearing failure. Excavations in fill, where present, may encounter obstructions consisting of debris, rubble, abandoned structures, or over-sized materials that may require special handling or demolition equipment for removal.

Near-vertical cuts in these deposits may not be stable particularly if the excavation is exposed to rainfall/runoff, encounters seepage, or extends below groundwater. Groundwater was measured at depths ranging from 3.5 to 5 feet below grade within monitoring wells installed as part of this investigation. Variations in groundwater levels within and outside this range should be anticipated. We anticipate dewatering measures will be needed to provide a dry excavation in which to work. Excavations that extend near or below the water table may experience “quick” conditions or bottom instability. The conditions at the bottom of the excavations can be improved by soil mitigation or ground improvement.

Excavated materials may be wet and need to be dried out before reuse as fill or off-hauled if there is not sufficient time for drying.

Any ground disturbance work performed as part of the Aquatic Center development shall be conducted in accordance with the Construction Soil and Groundwater Management Plan (SGMP) prepared for the project.

6.11 Construction Dewatering

Groundwater was encountered in our exploratory borings at depths as shallow as approximately 3.5 feet below the ground surface. Fluctuations in the groundwater level may occur as a result of variations in seasonal precipitation and other factors. Water intrusion into the excavations may occur as a result of groundwater intrusion or surface runoff. The contractor should be prepared to take appropriate dewatering measures in the event that water intrudes into the excavations. Considerations for construction dewatering should include anticipated drawdown, volume of pumping, potential for settlement, and groundwater discharge. Disposal of groundwater should be performed in accordance with the guidelines of the Regional Water Quality Control Board.

When excavating near or below groundwater, the dewatering system should depress the water level below the bottom of the cut to reduce the potential for subgrade instability and washout from behind sheeting or sloughing of exposed trench walls. The dewatering system should maintain the water level about 2 feet below the bottom of excavation to improve bottom stability when placing and compacting fill. Note that additional measures such as subgrade stabilizing and filtering geosynthetic materials and granular bridging layers may be needed depending upon the depth of excavation and strength of exposed subgrade soils. Sump pumps, well points, deep wells, geotextile-geonet composites, perforated underdrains, or stone blankets should be used, as appropriate, to drain water from below the bedding and foundation material and provide a stable working surface. Perforated underdrains and open-graded stone blankets should be wrapped in a suitable geotextile filter to reduce the potential for the removal of fines and subsequent creation of voids in the overlying and adjacent materials. The operation of the dewatering system should continue during and after construction of the below-grade structure until the potential uplift due to buoyancy has been mitigated. For deeper portions of the excavations, consideration should be given to installing a temporary cut-off wall (e.g. interlocking sheet piles, or similar) to help reduce the volume of water generated to maintain a workable area.

Consideration can be given to installing deep wells near the pools which can be used to draw down the groundwater level during construction. If dewatering wells will be left in place for post-construction use to draw down water level when the pool is empty for repairs or maintenance, the siting of the wells should be carefully considered along with their long-term performance capabilities and maintenance considerations.

6.12 Uplift Resistance

The groundwater levels at the project site are anticipated to range from about 3½ to 5 feet bgs based upon measurements from monitoring wells installed at the project site, summarized in Table 2.

Further, we anticipate that groundwater levels may rise in the future in response to sea-level rise. When the water level in the pools is lowered below the elevation of the adjacent groundwater, the pool will be subject to buoyant uplift forces that can be resisted by a combination of relief valves in the pool bottom, weight of the pool structure, pumping wells around the perimeter of the pool, or structural elements to tie down the pool. Temporary dewatering wells used during construction can be converted to permanent dewatering wells. Considering the size of the large pool, we anticipate that several pumping wells will be required around the perimeter of the pool as well as additional dewatering efforts in the interior of the pool to draw down the groundwater table and assist with dewatering during construction. While efforts to empty the pools for maintenance/repairs could be coordinated with low groundwater levels as measured in sounding wells around the perimeter of the pools, we judge that the efforts to achieve sufficiently low groundwater levels may be hindered by rising groundwater due to sea level rise or other unanticipated conditions during the life of the structure. Permanent dewatering wells around the pool perimeter typically have a limited perimeter of influence to draw down the groundwater table and the filtering materials included in the well construction can become clogged over time reducing their effectiveness. We judge that multiple relief valves would be required in the pool bottom to reduce hydrostatic loads. Further, the water pumped from dewatering wells and from the pool would need to be properly discharged which is anticipated to have an associated cost implication. Therefore, we recommend that the pools be designed to resist hydrostatic uplift using structural elements such as deep foundation, tiedown anchors, or uplift resistance incorporated into the ground improvement system. Recent groundwater measurements documented a high groundwater level of 3½ feet bgs. However, we recommend the design groundwater level take into consideration elevated groundwater levels over the life of the structure due to rising sea level over time.

Uplift resistance can be provided using a combination of the weight of the structure or structural tie-downs which can be included in the ground improvement elements or installed separately (e.g. deep foundations or tiedown anchors). Considering the combined goals of reducing total and differential settlement of the pools as well as the uplift resistance requirements, we suggest that a deep foundation system such as auger cast in place piles (ACIP) be considered below pools in combination with a mat or pile cap/grade beams connected to the pool shell. When deep foundations or ground improvement elements are installed from existing grade, portions of these elements remaining within the depth of the pool excavations would need to be carefully removed as the excavation proceeds to maintain structural integrity of the required portion of the elements below the pool structure.

7 CONCLUSIONS

Based on our review of the referenced background data, site field reconnaissance, subsurface evaluation, and laboratory testing, our opinion is that the proposed project is feasible from a geotechnical standpoint. Geotechnical considerations include the following:

- Our subsurface exploration encountered artificial fill, Young Bay Mud, Dune Sand, and Old Bay Mud. The artificial fill generally consisted of loose to dense silty and clayey sand and very stiff sandy lean clay and it varies in depth from the surface to depths between 3 to 9 feet below existing grade. The Young Bay Mud, as encountered below the artificial fill, generally consisted of soft to stiff clays and medium dense silty sand and varies in thickness between 8 to 13 feet with the bottom of the Young Bay Mud varying up to about 19 feet below existing grade. The Dune Sand, as encountered below the Young Bay Mud, generally consisted of loose to very dense silty and clayey sand and extended to a maximum depth of 39 feet below existing grade in the borings. Old Bay Mud was encountered below the Dune Sand in Boring B-1 at a depth of about 39 feet below existing grade. Based on CPT soundings, we anticipate that Old Bay Mud is present; however, it is difficult to distinguish between the Dune Sand and Old Bay Mud. In general, the old bay mud as encountered in the boring consisted of bluish-gray, wet, dense to very dense silty sand.

- Groundwater was encountered during our subsurface exploration at depths ranging between 5 feet and 13¾ feet below the ground surface in the CPT soundings, and approximately between 13 feet and 14 feet below the ground surface in the borings. Groundwater measurements in monitoring wells varied from 3½ to 5 feet bgs. Variation and fluctuation in groundwater levels should be anticipated as discussed in Section 5.4. The contractor should be prepared to take appropriate dewatering measures in the event that water intrudes into the excavations as discussed in Section 6.11.

- The site could experience a relatively large degree of ground shaking during a significant earthquake on a nearby fault. Seismic design criteria are presented in Section 8.1.

- The project site is located within a liquefaction hazard zone established by the California Geological Survey (CGS, 2003). The results of our analyses indicate that there is a high potential for liquefaction to occur at the site and that dynamic settlement will be up to about 3½ inches with about 2 inches of differential settlement over a distance of 50 feet. Based on our evaluation, we judge that surface disruption due to liquefaction is likely to impact development of the site. Since the anticipated settlement exceeds acceptable tolerances for structures supported on conventional shallow footings, foundation type selection and ground improvement considerations are provided in the recommendation section.

- The potential for surface manifestation of liquefaction or sand boils is moderate to high. Foundation type selection is provided in the recommendation section.

- The estimated lateral spread displacement of up to approximately 1 inch from our analysis conforms with the upper limit on lateral displacement for shallow foundations in Table 12.13-2 of ASCE 7-16 on projects with a Risk Category of I, II, or III. Foundation type selection is provided in the recommendation section to mitigate differential lateral spread displacement for structural elements.

- The site and vicinity are relatively flat with low topographic relief of about 1 foot on site and an average slope gradient of approximately 2.3 percent. No significant slopes are proposed for the project under consideration. The site is not located within a hazard zone for earthquake-induced landslides on the Seismic Hazard Zones Map prepared by the CGS (2003). Based on the existing topography and our review of existing maps and literature, we do not regard landslides or seismic slope stability as design considerations for the proposed project.

- Unsuitable materials are anticipated to be present at the site based on our subsurface exploration. Recommendations for remedial pad grading are provided to mitigate surficial concerns related to undocumented fill and weak or soft soils at shallow depth which may impact access for construction equipment.

- The subsurface conditions encountered during the exploration indicate that collapsible soil is not a consideration for the site. If present, foundation type selection and remedial pad grading are provided in the recommendation section to mitigate differential settlement resulting from collapsible soils.

- The site is not in an area known for regional land subsidence due to groundwater withdrawal, peat loss, or oil extraction (USGS, 2018). As such we do not regard land subsidence as a design consideration.

- Based on the subsurface materials encountered in our exploratory borings and CPT soundings, static and dynamic settlement as discussed in Sections 6.1.5 and 6.7 are a design consideration. Ground improvement and deep foundation systems, or a combination of these, should be considered to mitigate static and dynamic settlement.

- Limited laboratory testing of one soil sample collected during the subsurface exploration for this study indicates that the tested soil does not meet the definition of a corrosive environment for structures (Caltrans, 2021) and the sulfate exposure to concrete is negligible (Class S0). Based on electrical resistivity, the sample tested are considered to be moderately corrosive to ferrous metals and slightly corrosive based on redox potential, as noted in Appendix D. A corrosion engineer may be consulted to provide specific guidance on protective measures to mitigate corrosion.

- Laboratory testing performed on selected soil samples indicated plasticity index values generally ranging between 4 and 13, indicating a low potential for swell/expansion. However, one laboratory result for a soil sample at location B-3 obtained at a depth of about 6 feet (within the Young Bay Mud) indicated a plasticity index of 54 which corresponds to critically high expansion potential. The Young Bay Mud is generally located below the groundwater where the risk of shrink/swell effects is low. However, Bay Mud was often used as fill and may be encountered within the fill materials at the project site. Recommendations are provided to mitigate soil expansion.

- Excavations that remain unsupported and exposed to water, or encounter seepage, or granular soil may be unstable and prone to sloughing. Recommendations for excavation stabilization are provided.

- The earth materials underlying the site over the anticipated depth of excavation should be excavatable with conventional earth moving equipment in good working condition; however, there is a moderate to high risk of encountering weak surficial conditions which may impact accessibility for construction equipment, and recommendations are provided for ground improvement and chemical treatment.

- We anticipate that the pool will be subject to buoyant uplift forces given that the groundwater levels at the project site were measured to range from 3½ to 5 feet, and groundwater may rise in the future in response to sea-level rise. Uplift resistance is a design consideration for the project and can be resisted by a combination of relief valves in the pool bottom, weight of the pool structure, pumping wells around the perimeter of the pool, or structural elements to tie down the pool. Recommendations are provided in the proceeding sections below.

8 RECOMMENDATIONS

The following sections present our geotechnical recommendations for the design and construction of the proposed improvements. The project improvements should be designed and constructed in accordance with these recommendations, applicable codes, and appropriate construction practices.

8.1 Seismic Design Criteria

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Seismic Site Class D was selected based on the shear wave velocity measurements collected on CPT-2, regional mapped Vs30 values, and subsurface findings indicating soft clay soils are less than 10 feet thick. The spectral ordinates and seismic coefficients based on the mapped values of the risk-targeted spectral response acceleration, consistent with Section 11.4 of ASCE Standard 7-16, are presented in the Table 5 (SEAOC & OSHPD, 2023). In conformance with the 2022 California Building Code and the exception to item 1 in Section 11.4.8 of ASCE 7-16 Supplement 3, the spectral ordinates consistent with Section 11.4 of ASCE Standard 7-16 provided in Table 5 may be used for seismic design.

Table 5 – California Building Code Seismic Design Criteria	
Seismic Design Parameter **Evaluated for 37.779751º North latitude, 122.272906º West longitude**	**Value**
Site Class	D – Stiff Soil
Site Coefficient, F_a	1
Site Coefficient, F_v	1.7
Mapped Spectral Acceleration at 0.2-second Period, S_s	1.536 g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.6 g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.536 g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	1.530 g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.024 g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	1.020 g
Seismic Design Category for Risk Category I, II, or III	D
Note: *S_{M1} and S_{D1} for Section 11.4 parameters include 50% increase per ASCE 7-16 Section 11.4.8 Item 1	

8.2 Earthwork Recommendations

Earthwork should be performed in accordance with the requirements of applicable governing agencies and the recommendations presented below. Evaluations performed by the geotechnical consultant during the course of operations may result in new recommendations, which could supersede the recommendations in this section.

8.2.1 Pre-Construction Conference

We recommend that a pre-construction conference be held to discuss the recommendations presented in the report. Representatives of the City, the design engineer, Ninyo & Moore, and the contractor should be in attendance to discuss project schedule and earthwork requirements.

8.2.2 Site Preparation

Site preparation should begin with the demolition of the designated existing improvements and removal of vegetation, utility lines, surface obstructions (e.g., pavements, aggregate base, curb/gutter, foundations), rubble and debris, and other deleterious materials from areas to be graded. Vegetation should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside of the proposed excavation and fill areas. Rubble and excavated materials that do not meet criteria for use as fill should be removed from the site for disposal in an appropriate landfill. Soil containing roots or other organic matter may be stockpiled for later use as landscaping fill, as authorized by the owner's representative. Active utilities within the project limits, if any, should be re-routed or protected from damage by construction activities. Existing utilities to be abandoned should be removed, crushed in place, or backfilled with grout. Excavations resulting from removal of buried utilities, tree stumps, or obstructions should be backfilled with compacted fill in accordance with the recommendations in the following sections.

8.2.3 Treatment of Near-Surface Soils

In order to provide suitable support and reduce the potential for settlement of the proposed improvements, we recommend that the upper one to two feet of fill soils in the areas beneath the proposed new improvements be recompacted. Prior to compaction of surficial soils, a representative of our firm should observe proof rolling of the surface to check for pumping which would suggest weak surficial soils and possible need for alternative considerations (e.g. chemical treatment, geogrid, or other methods to improve subgrade strength). Where overexcavation is performed, suitable excavated soils should be replaced as engineered fill compacted to 90 percent relative compaction per ASTM D 1557. The lateral limits of surficial compaction for the building area should extend to approximately five (5) feet beyond the building perimeter or to a distance equal to the depth of overexcavation, whichever is greater, where overexcavation is performed. The lateral limits of surficial compaction for pavements should extend to approximately 2 feet or to a distance equal to the depth of overexcavation, whichever is greater, where overexcavation is performed.

Where excavation is performed, the bottom of the excavation may expose soft or weak soils. The excavation bottom should be evaluated by our representative during the excavation work. Prior to placing new compacted fill, the exposed subgrade should be observed by a representative of Ninyo & Moore to confirm the bottom is firm and unyielding.

8.2.4 Chemical Treatment

We would like to point out that, historically, Bay Mud was sometimes used in fill operations, and there is a possibility that expansive Bay Mud may be encountered within the artificial fill. Note that chemical treatment is often used to reduce expansion potential and strengthen soils, especially in fill over Bay Mud conditions, to improve access for grading equipment. The on-site soil may be chemically treated with high calcium quicklime to reduce the expansion characteristic of the soil as an alternative to importing select fill. The high calcium quicklime, for treatment of primarily clayey soils, should conform to ASTM Standard C977. Alternatively, cement treatment can be used to improve strength of sandy or combination (e.g. sand, silt, clay mixed soils). The chemical treatment should be performed by an experienced contractor that specializes in the chemical treatment of soil. The chemical agent should be proportioned and spread with a mechanical spreader and mixed into the soil on a mixing table or in place to produce consistent distribution of the agent within the treated layer.

The depth of mixing should not exceed 18 inches per lift or the capacity of the mixer if less. Precautions to reduce the potential for dusting of quicklime or cement, such as scheduling or suspending operations to avoid windy weather, should be taken. Casting or tailgating of the chemical agent should not be permitted. The mixer should be equipped with a rotary cutting/mixing assembly, and an automatic water distribution system. Mixing or spreading operations should not be performed during inclement weather or when the ambient temperature is less than 35 degrees Fahrenheit or during foggy or rainy weather. Adjacent passes of the mixer should overlap by 4 inches or more.

For preliminary cost evaluation a dosage of 5 percent by dry weight of soil should be assumed, with an assumed dry weight of soil of 110 pcf. The actual dosage should be determined through laboratory testing at the time of construction. Testing typically requires about 5 days from receipt of a fresh sample of lime. The contractor should provide a sample of the lime that will be used in construction to Ninyo & Moore about 2 weeks prior to the planned start of lime treatment.

Mixing and pulverizing should continue until the treated soil does not contain untreated soil clods larger than 1 inch and the quantity of untreated soil clods retained on the No. 4 sieve is less than 40 percent of the dry soil mass. Water should be added as-needed during the mixing process to achieve moisture content above the optimum, as evaluated by ASTM D1557, for the lime-soil mixture. The lime-soil mixture should be re-mixed following a 16-hour minimum mellowing period after the initial mixing. The lime-soil mixture should be compacted within 3 days after initial mixing to achieve 90 percent of the reference density as evaluated by ASTM D1557 on a dry density basis.

The grading contractor should provide assistance to Ninyo & Moore with grade checking to confirm surface elevations and depth of mixing as the lime treatment operation proceeds.

8.2.5 Observation and Removals

Prior to placement of fill, or the placement of forms or reinforcement for foundations, the client should request an evaluation of the exposed subgrade by Ninyo & Moore. Materials that are considered unsuitable shall be excavated under the observation of Ninyo & Moore in accordance with the recommendations in this section or supplemental recommendations by the geotechnical engineer.

Unsuitable materials include, but may not be limited to dry, loose, soft, wet, expansive, organic, or compressible natural soil, and undocumented or otherwise deleterious fill materials. Unsuitable materials should be removed from trench bottoms and below bearing surfaces to a depth at which suitable foundation subgrade is exposed, as evaluated in the field by Ninyo & Moore.

8.2.6 Material Recommendations

Materials used during earthwork, grading, and paving operations should comply with the requirements listed in Table 6. On-site materials that have been chemically treated may be used as fill outside areas where the chemical properties may impact planned vegetation. Materials should be evaluated by the geotechnical engineer for suitability prior to use. The contractor should notify the geotechnical consultant prior to import of materials or use of on-site materials to permit time for sampling, testing, and evaluation of the proposed materials. On-site materials may need to be dried out before re-use as fill. Also, as previously noted, expansive clay soils can be difficult to work with. The contractor should be responsible for the uniformity of import material brought to the site.

Table 6 – Recommended Material Requirements

Material and Use	Source	Requirements[1,2,3]
Select Fill	Import	Close-graded with 35 percent or more passing No. 4 sieve and either: Expansion Index of 50 or less, Plasticity Index of 12 or less, or less than 10 percent, by dry weight, passing No. 200 sieve
Pipe/Conduit Bedding and Pipe Zone Material -material below conduit invert to 12 inches above conduit	Import	90 to 100 percent (by mass) should pass No. 4 sieve, and 5 percent or less should pass No. 200 sieve
Trench Backfill - above bedding material	Import or On-site Borrow	As per general fill and excluding rock/lumps retained on 4-inch sieve or 2-inch sieve in top 12 inches
Aggregate Base	Import	Class 2, ¾ inch max. Should not contain recycled asphalt concrete if used below floor slabs, CSS[5] Section 26-1.02
Controlled Low Strength Material (CLSM)	Import	CSS5 Section 19-3.02G

Notes:
1 In general, fill should be free of rocks or lumps in excess of 6 inches in diameter, trash, debris, roots, vegetation or other deleterious material.
2 In general, import fill should be tested or documented to be non-corrosive[3] and free from hazardous materials in concentrations above levels of concern.
3 The specification of utility owner or local agency may supersede the indicated requirements in this table.
4 Non-corrosive as defined by the Corrosion Guidelines (Caltrans, 2021).
5 CSS is California Standard Specifications (Caltrans, 2022).

8.2.7 Subgrade Preparation

Where chemical treatment has not been performed, subgrade below slabs or fill should be prepared as per the recommendations in Table 7. Prepared subgrade should be maintained in a moist (but not saturated) condition by the periodic sprinkling of water prior to placement of additional overlying fill. Subgrade that has been permitted to dry out and loosen or develop desiccation cracking, should be scarified, moisture-conditioned, and recompacted as per the requirements above.

Table 7 – Subgrade Preparation Recommendations

Subgrade Location	Source
Below Slabs, Pavement, and General Fill	• After clearing per Section 8.2.2., check for unsuitable materials as per Section 8.2.5 • Scarify 8 inches then moisture condition and compact as per Section 8.2.8. • Keep in moist condition by sprinkling water.

8.2.8 Fill Placement and Compaction

Fill and backfill should be compacted in horizontal lifts in conformance with the recommendations presented in Table 8. The allowable uncompacted thickness of each lift of fill depends on the type of compaction equipment utilized, but generally should not exceed 8 inches in loose thickness.

Table 8 – Fill Placement and Compaction Recommendations

Fill Type	Location	Compacted Density[1]	Moisture Content[2]
Subgrade	Below pavement (within 12 inches of finished subgrade)	95 percent	+ 2 percent or above
	Below slabs or fill and in locations not already specified	90 percent	+ 2 percent or above
General Fill	Below pavement (within 12 inches of finished subgrade)	95 percent	+ 2 percent or above
	In locations not already specified	90 percent	+ 2 percent or above
Lime-Treated Soils	All locations	95%	+3%
Bedding and Pipe Zone Fill	Material below invert to 12 inches above pipe or conduit	90 percent	Within +/- 2 Optimum
Trench Backfill	Top 12 inches below finish subgrade for areas subject to vehicular loading	95 percent	+ 2 percent or above
	In locations not already specified	90 percent	+ 2 percent or above

Table 8 – Fill Placement and Compaction Recommendations			
Fill Type	Location	Compacted Density[1]	Moisture Content[2]
Aggregate Base	Below slabs or pavement	95 percent	Near Optimum

Notes:
[1] Expressed as percent relative compaction or ratio of field density to reference density (typically on a dry density basis for soil and aggregate). The reference density of soil and aggregate should be evaluated by ASTM D 1557.
[2] Target moisture content at compaction relative to the optimum as evaluated by ASTM D 1557

Compacted fill should be maintained in a moist (but not saturated) condition by the periodic sprinkling of water prior to placement of additional overlying fill. Fill that has been permitted to dry out and loosen or develop desiccation cracking, should be scarified, moisture-conditioned, and recompacted as per the requirements above.

8.2.9 Temporary Excavations and Shoring

We understand that the maximum depth of the pools will be on the order of 7 feet. Considering that up to 3 feet of fill may be placed in the pool areas, the depth of pools is estimated to be on the order of 10 feet below existing grade, and excavations up to about 12 feet below finished grade are anticipated. Given the historic and more recent observations of groundwater depth, groundwater is likely to be encountered during excavation for the pool(s). The construction contractor should be advised that installation of a robust groundwater control system will most likely be required. The design, installation, and operation of such a system, which is the responsibility of the contractor, should ensure that groundwater levels are maintained at least 2 feet below the deepest point of the excavation. Dewatering pits or sumps or wellpoints should be used to depress the groundwater level (if encountered) below the bottom of the excavation.

Excavations should be stabilized in accordance with the Excavation Rules and Regulations (29 Code of Federal Regulations [CFR], Part 1926) stipulated by the Occupational Safety and Health Administration (OSHA). Stabilization should consist of shoring sidewalls or laying slopes back.

Table 9 lists the OSHA material type classifications and corresponding allowable temporary slope layback inclinations for soil deposits that may be encountered on site. We encountered granular soils that consisted of loose to very dense, silty sand and sand during our subsurface investigation, which corresponds to OSHA Type C soil. If materials other than those anticipated are encountered, Ninyo & Moore should be provided an opportunity to review subsurface conditions.

Alternatively, an internally-braced shoring system or trench shield conforming to the OSHA Excavation Rules and Regulations (29 CFR, Part 1926) may be used to stabilize excavation sidewalls during construction. The lateral earth pressures listed in Table 9 may be used to design or select the internally-braced shoring system or trench shield. The recommendations listed in Table 9 are based upon the limited subsurface data provided by our subsurface exploration and reflect the influence of the environmental conditions that existed at the time of our exploration. Excavation stability, material classifications, allowable slopes, and shoring pressures should be re-evaluated and revised, as-needed, during construction. Excavations, shoring systems and the surrounding areas should be evaluated daily by a competent person for indications of possible instability or collapse.

If the contractor intends to use temporary shoring to support the excavation during construction, and does not have a fully redundant groundwater control system (meaning extra pumps and power sources available on site at all times), then the shoring should be designed to resist full hydrostatic water pressures on the shoring.

Table 9 – OSHA Material Classifications and Allowable Slopes

Material	OSHA Classification	Allowable Temporary Slope[1,2,3]	Lateral Earth Pressure on Shoring[4] (psf)
Cohesive Soils (above groundwater)	Type B	1h:1v (45°)	45×D + 72
Granular Soils (above groundwater)	Type C	1½ h:1v (34°)	80×D + 72

Notes:
1 Allowable slope for excavations less than 20 feet deep. Excavation sidewalls in cohesive soil may be benched to meet the allowable slope criteria (measured from the bottom edge of the excavation). The allowable bench height is 4 feet. The bench at the bottom of the excavation may protrude above the allowable slope criteria.
2 In layered soil, layers shall not be sloped steeper than the layer below.
3 Temporary excavations less than 5 feet deep may be made with vertical side slopes and remain unshored if judged to be stable by a competent person (29 CFR, Part 1926.650).
4 'D' is depth of excavation for excavations up to 20 feet deep. Includes a surface surcharge equivalent to two feet of soil.

The shoring system should be designed or selected by a suitably qualified individual or specialty subcontractor. The shoring parameters presented in this report are preliminary design criteria, and the designer should evaluate the adequacy of these parameters and make appropriate modifications for their design. We recommend that the contractor take appropriate measures to protect workers. OSHA requirements pertaining to worker safety should be observed.

Excavations made in close proximity to existing structures may undermine the foundation of those structures and/or cause soil movement related distress to the existing structures.

Stabilization techniques for excavations in close proximity to existing structures will need to account for the additional loads imposed on the shoring system and appropriate setback distances for temporary slopes. The contractor should be solely responsible for protection of existing site improvements and provide shoring and/or underpinning as needed.

The excavation bottoms may encounter wet, loose material which may be subject to pumping under heavy equipment loads. The contractor should be prepared to stabilize the bottom of the excavations. In general, unstable bottom conditions may be mitigated by using a stabilizing geogrid, overexcavating the excavation bottom to suitable depths and replacing with compacted fill, chemical treatment, ground improvement or other suitable method. Additionally, aeration of wet soils should be anticipated.

8.2.10 Utility Trenches

Trenches constructed for the installation of underground utilities should be stabilized in accordance with our recommendations in Section 8.2.9. Where groundwater is encountered within utility trench excavations, groundwater should remain drawn down until sufficient backfill has been placed to counteract uplift forces. Utility trenches should be backfilled with materials that conform to our recommendations in Section 8.2.6. Trench backfill, bedding, and pipe zone fill should be compacted in accordance with Section 8.2.8 of this report. Bedding and pipe zone fill should be shoveled under pipe haunches and compacted by manual or mechanical, hand-held tampers. Trench backfill should be compacted by mechanical means. Densification of trench backfill by flooding or jetting should not be permitted.

Trenches should not be excavated adjacent to footings. If trenches are to be excavated near a continuous footing, the bottom of the trench should be located above a 2:1 (horizontal to vertical) plane projected downward from the bottom of the footing. Utility lines that cross beneath footings should be encased in concrete or CLSM below the footing for a distance equivalent to the depth of the excavation.

8.3 Foundations

Foundations should be designed in accordance with structural considerations and our geotechnical recommendations. In addition, requirements of the governing jurisdictions, practices of the Structural Engineers Association of California, and applicable building codes should be considered in the design of the structures.

8.3.1 Foundation Type Selection

Settlement estimates based on static settlement due to sustained loads and seismically-induced settlement can be reduced if the ground is improved or deep foundations are used. At this site, given the significant level of seismic shaking, and the depth and thickness of the potentially liquefiable soil, mitigating this estimated settlement is likely to be very costly.

In order to reduce the effects of differential settlement including where underground utilities enter/exit structures supported on deep foundations or improved soil. Utilities should be either be supported on piles or improved soil. Alternatively, a utility vault could be constructed to house utilities and include flexible connections to reduce the impacts of differential settlement.

Minor improvements that have higher tolerance for total or differential settlement can be supported on rigid shallow foundations without soil improvement.

8.3.2 Ground Improvement

Ground improvement and deep foundation systems, or a combination of these, should be considered to mitigate the static and dynamic settlement considerations, as discussed in Sections 6.1.5 and 6.7 of the report. We recommend that displacement methods be considered for alternative foundations or soil improvement to reduce the amount of soil off-haul generated. Ground improvement should extend through the upper liquefiable soils to mitigate the seismically-induced settlement, and shallow foundations can then be supported on the improved soil. As previously indicated, we recommend that ground improvement could include rigid inclusion, deep soil mixing (DSM), drilled displacement columns (DDC), stone columns, aggregate piers, pressure grout, or other ground improvement methods that achieve the performance requirements of the project. In general, shallow foundations supported on improved soil have a significantly improved allowable bearing capacity compared to shallow foundations supported on un-improved site soils. The grid spacing of ground improvement should take into consideration the spanning capabilities of the supported slab-on-grade or pool shell.

We recommend that a specialty contractor be retained for design of ground improvement system(s) including element size, spacing, depth, and layout of the proposed ground improvement as a design-build component to the construction. The ground improvement should extend to a minimum depth of 20 feet below the existing ground surface to mitigate total static settlement to ½ inch. Seismic settlement below a depth of 20 feet is up to 2.6 inches with differential static plus seismic settlement up to about 2 inches over 50 feet. Ground improvement to a depth of 34 feet reduces total static plus seismic settlement to about 1 inch with differential settlement up to about ½ inches over 50 feet. The design of the ground improvement system should verify compliance with specified bearing capacity and settlement tolerance, and include consideration of downdrag loads from consolidation and seismic settlement. Typically, the spacing between individual ground improvement elements varies from about 5 to 15 feet on-center, depending upon the type of ground improvement. The ground improvement should be designed to achieve an allowable bearing capacity of at least 3500 psf for dead load plus live load and a 1/3 increase for all loads including wind or seismic. We understand from the design team that post-improvement combined static and seismic differential settlement which exceeds 2 inches across the length or width of the pool will result in unreasonably high post-earthquake repair costs, and is not acceptable.

To achieve the bearing capacity and design settlement tolerances, we recommend that soil improvement (e.g. deep foundations or ground improvement) be performed for the swimming pools and associated underground piping and surface chambers. For reduction of differential settlement between the pool decking and the pools, ground improvement should be considered below the pool deck as well. We recommend that soil improvement be performed to a minimum depth of 34 feet below existing grade. Where possible, the ground improvement should extend laterally at least 10 feet beyond the limits of improvements that have differential settlement tolerance concerns or one row of ground improvement members, whichever is less. If there are existing improvements within this recommended zone of ground improvement, they would need to be demolished and rebuilt or evaluated individually on a case-by-case basis. We recommend that consideration be given to performing the soil improvement from existing site grades to reduce difficulties associated with excavation and access in unimproved excavations, particularly for the deeper pool excavations; however, contractor means and methods may include ground improvement installation following required excavations.

The soil improvement should include CPT testing prior to improvement and post-improvement to check that the improvement has achieved the specified settlement tolerance. Depending upon the type of improvement, bearing capacity testing should be performed. Post-improvement performance testing that indicates settlement tolerance or specified bearing capacity has not been achieved would require additional improvement at intermediate spacing to the initial installation grid. The proposed ground improvement method and approach should be submitted to the design team and Ninyo & Moore for review.

8.3.3 Ground Improvement Contractor Requirements

The ground improvement contractor (Contractor) shall be responsible for design of a ground improvement system that meets the project densification, allowable bearing capacity, and settlement requirements. Industry recognized standards or design methods specific to the Contractor's equipment and construction methods should be used.

The contractor shall provide an improvement plan with shop drawings and design computations, using generally accepted design methodology in geotechnical and structural engineering that meets the performance requirements. These requirements include the factor of safety and the tolerable settlement amounts in the case of structural footings and pools. The following minimum performance requirements shall be used in the table and bullet points below:

Table 10 – Ground Improvement Specifications

Ground Improvement Options	Settlement Specification
Un-mitigated Soils	• Total Static & Dynamic: Estimated to be > 3 inches • Differential Static & Seismic: Estimated to be > 2 inches (over a distance of 50 feet)
Ground Improvement to 20 feet min.	• Total Static = ½ inch • Total Dynamic = 2.6 inches • Differential Static & Seismic (over a distance of 50 feet) = 2 inches
Ground Improvement to 34 feet min.	• Total Static & Seismic = 1 inch • Differential Static & Seismic (over a distance of 50 feet) = ½ inch

Notes:
The ground improvement should extend a minimum of 10 feet or one row of improvements beyond the area of improvement, whichever is less.

- Allowable Bearing Pressure (Min.) for Footings supported by Ground Improvement:
  - Dead and Live Load: 3,500 psf.
  - Dead and Live and Earthquake loads allowable bearing pressure with 1/3 increase of 4667 psf.
  - Ultimate Load bearing pressure of 10,500 psf.

The Contractor should make their own interpretation of strength parameters for the soil, obtained or derived from the soil boring logs, cone penetration tests, and any geotechnical laboratory testing data provided in the Geotechnical Report. Static settlement shall be assessed using appropriate soil parameters for an elastic settlement analysis based on an area replacement ratio considering the stiffness of the native soils, and the ground improvement system. Liquefaction and seismic settlement estimates shall be performed using methodology presented in the project geotechnical report, which followed the procedures in the Idriss and Boulanger, 2014. Liquefaction and settlement shall be evaluated for the upper 50-feet of the soil profile. Any additional subsurface information needed to design the ground improvement shall be the responsibility of the Contractor, and results of additional subsurface information should be provided to the Owner/Design Team.

8.3.4 Submittals Required

- The Contractor should submit detailed design calculations and construction drawings to the Owner for approval at least six (6) weeks prior to the start of construction. All plans should be signed and sealed by a Geotechnical Engineer (the Designer) registered in the State of California.
- The Contractor Experience Profile. The Contractor must submit documentation evidencing the experience requirements.
- Pre-Construction Test Data – The Contractor should furnish Owner a description of the installation equipment, installation records, complete test data and original digital files, analysis of the test data, compliance with acceptance criteria, and recommended design parameter values based on the pre-construction test program results. The report shall be prepared, signed and sealed by a Geotechnical Engineer registered in the State of California.
- Shop Drawings of the ground improvement plan signed and sealed by a California Licensed Geotechnical Engineer showing horizontal limits, locations, pattern, spacing, diameters, top and bottom elevations, and identification numbers, in addition to any other details needed to describe the work.
- Pre-construction test report should be submitted for review.
- Field Validation Program Plan: At least 30 days before the start of the field validation program, the Contractor should submit a field validation program plan which contains descriptions of the construction procedures, equipment and ancillary equipment to be used for ground improvement, operational and material parameters to be monitored during field validation, layout of the ground improvement elements to be constructed, and summary of QC/QA samples to be collected and tested, along with examples of the forms that will be used to document the work.
- Ground Improvement Work Plan.
- Detailed descriptions of sequence of construction and all construction procedures, equipment (catalog cut-sheets), and ancillary equipment to be used.
- Methods for controlling and recording the verticality and the top and bottom elevation of each element.

- When ground improvement elements are required to penetrate into a bearing layer, the necessary procedure and the measurement to confirm the end-bearing.

- Working drawings and calculations for the ground improvement elements, showing the site location of the project, and the dimensions, layout and locations of all elements. Drawings should indicate the identification number of every element. Calculations and drawings should demonstrate that the element layout, depth and quantity meet the specification requirements. The design calculations shall be performed by a Professional Engineer registered in state of California, who shall also prepare, stamp, and sign the drawings.

- Ground Improvement schedule information.

- Sample Daily Production Report.

- Details of all means and methods proposed for QC/QA activities including surveying, process monitoring, sampling, testing, documentation, and schedule milestones.

- Names of any subcontractors used for QC/QA activities. An independent laboratory should be used for QC/QA testing and should be approved by the City, SEOR and GEOR.

- Material Certifications: Certificates of compliance must be submitted as proof of conformance to materials standards and requirements.

- Production Records. By the end of the next business day following each shift, the Contractor shall submit a Daily Production Report in the approved format. The Daily Production Report shall be completed and signed by the Contractor's Project Superintendent. The report should contain at a minimum:

    - Project name.

    - Day, month, year, time of work shift (beginning and end).

    - Name of field superintendent in charge of work for the contractor.

    - Ground improvement equipment (rig number) in operation during the shift and specific activities conducted by said equipment.

    - Treatment zone and reference drawing number.

    - Elevation of top and bottom of treatment zone.

    - Element number, diameter, and location coordinates.

    - Date and time (start and finish) of element.

    - A record of the location of each completed column/element installed during the work shift and all zones completed to-date on a plan of suitable scale to clearly show the location of the elements.

    - Element verticality measurements.

    - A description of obstructions, interruptions, or other difficulties during installation and their resolution.

- Other pertinent observations including, but not limited to: ground settlement and/or heave, collapses of the treatment zone, and any unusual behavior of any equipment during the process.
- For QA/QC testing, provide collection date, time, plan location, elevation, and identification numbers of all samples.
- Quantities of all materials delivered to site, plus a reconciliation with amounts used for the ground improvement operation.
- Summary of any downtime or other unproductive time, including time, duration and reason.
- Detailed results of all testing.
- Quality Control/Quality Assurance Records. Calibration data must be submitted for all measurement devices. Within three business days of completing any QC/QA testing, the Contractor shall submit the test results, including original data sheets from the laboratory and an evaluation of the compliance of the test results with project acceptance criteria. Equipment should be calibrated prior to initial use and repeated every 3 months.
- As-Built Field Measurement Data. After completion of the project, the Contractor must submit as-built field measurement data indicating surveyed as-built plan locations of each CDSM element including: the element center (per site specific coordinates), the element dimension, the column verticality, and the top and bottom elevations of each element to the accuracy required by the project Specifications.
- Verification testing for the ground improvement should include pre improvement and post improvement testing to verify compliance.

8.3.5 Deep Foundations

Deep foundations consisting of driven or drilled displacement piles are an option for settlement sensitive structures to mitigate static settlement. However, deep displacement foundations may encounter refusal at depths shallower that 34 feet which is the anticipated maximum depth of liquefaction and associated settlement. We understand that deep foundations are not currently under consideration for the project; however, we can provide recommendations for the deep foundation, if requested.

8.3.6 Shallow Foundations (Improved Soil)

Allowable bearing capacity of shallow foundations supported by improved soil is 3,500 psf for dead plus live loads. The allowable bearing capacity can be increased by one-third for all loads including wind or seismic.

Resistance to laterals loads for shallow foundations bearing on granular fill may be designed using a coefficient of friction of 0.45 (total frictional resistance equals coefficient of friction times the dead load). Foundations may be designed using a passive resistance value of 350 pounds per square foot per foot of depth. The upper foot should be ignored for passive resistance unless confined by a slab or pavement. Where waterproofing is placed below footings, the coefficient of friction is significantly reduced and is dependent upon the material used for waterproofing. If waterproofing is placed below the footing, we recommend ignoring the coefficient of friction below footings.

8.3.7 Rigid Shallow Foundations (Unimproved Soil)

Rigid shallow foundations such as footings interconnected with grade beams or reinforced thickened concrete mat foundations are effective for reducing differential settlement, and can be used over unimproved soil for structures or planned improvements which are more tolerant of settlement. The allowable capacities of shallow foundations will be governed primarily by settlement. The existing fill at the project site is not considered engineered fill, and the upper one to two feet of existing fill should be recompacted per Section 8.2.3 to improve foundation supporting conditions.

Lime treatment of existing clayey soils is another option for improving supporting conditions for shallow foundations. For granular fill soils, cement treatment is more effective. Both sandy and clayey fill soils were encountered in our subsurface exploration. Therefore, limits of surficial treatment should be further evaluated for appropriate treatment option with lime, cement, or a combination of both.

Allowable bearing capacity of shallow foundations (unmitigated for liquefaction) would be on the order of about 1000 psf to 2000 psf for dead load plus live load, depending upon the settlement tolerance and subgrade preparation. The allowable bearing capacity can be increased by one-third for all loads including wind or seismic.

Resistance to laterals loads for shallow foundations bearing on fill may be designed using a coefficient of friction of 0.30 (total frictional resistance equals coefficient of friction times the dead load). Foundations may be designed using a passive resistance value of 300 pounds per square foot per foot of depth. The upper foot should be ignored for passive resistance unless confined by a slab or pavement. Where waterproofing is placed below footings, the coefficient of friction is significantly reduced and is dependent upon the material used for waterproofing. If waterproofing is placed below the footing, we recommend ignoring the coefficient of friction below footings.

Mat foundations supported on engineered fill overlying Young Bay Mud may be designed using a coefficient of subgrade reaction, $\mathbf{K_{v1}}$, of 100 kips per cubic foot (kcf). The coefficient of subgrade reaction **K_b** for a mat of a specific width, may be evaluated using the following equation where **b** is the width of the foundation:

$$K_b = K_{v1}[(b+1)/2b]^2$$

8.3.8 Slabs-on-Grade (Improved Soil)

Building floor slabs and the pool bottom slabs underlain by improved soil should be designed by the project structural engineer based on the anticipated loading conditions. The subgrade should be prepared in accordance with Section 8.2.7.

Beneath the pools' bottoms and the pools' decks, an 8-inch thick layer of Caltrans Class 2 aggregate base should be used, placed in accordance with Section 8.2.8. A modulus of subgrade reaction of 300 pounds per square inch per inch of deflection (psi/in) for the improved soil can be used for initial modeling of the slab. The actual subgrade reaction below the mat will result in a variable subgrade modulus that matches the tendency for dishing settlement between soil improvement columns.

For other slabs where a vapor retarding system is not used, slabs should be constructed on 6 inches, or more, of Caltrans Class 2 aggregate base conforming to Section 8.2.6 and placed in accordance with Section 8.2.8.

Slabs should be reinforced with deformed steel bars. We recommend that masonry briquettes or plastic chairs be used to aid in the correct placement of slab reinforcement in the upper half of the slab. Refer to Section 8.8 for the recommended concrete cover over reinforcing steel. A vapor retarder is recommended in areas where moisture-sensitive floor coverings or conditioned environments are anticipated. Joints consistent with ACI guidelines (ACI, 2020) may be constructed at periodic intervals to reduce the potential for random cracking of the slab.

8.3.9 Slab-on-Grade (Unimproved Soil)

Slab-on-grade floors for buildings on unimproved soil should be designed by the project structural engineer as structural slabs based on the anticipated loading conditions. A vapor retarder is recommended in areas where moisture-sensitive floor coverings or conditioned environments are anticipated. Where a vapor retarding system is not used, slabs should be constructed on 4 inches of compacted aggregate base. The slab should be reinforced with deformed steel bars with a nominal diameter of ⅜-inch or more as designed by the project structural engineer. Masonry briquettes or plastic chairs should be used to maintain the position of slab reinforcement, during concrete placement, in the upper half of the slab with appropriate concrete cover over the reinforcing steel. Refer to Section 8.8 for the recommended concrete cover over reinforcing steel. Joints consistent with the guidelines of ACI Committee 302 may be constructed at periodic intervals to reduce the potential for random cracking of the slab.

8.3.10 Moisture Vapor Retarder

The migration of moisture through slabs underlying enclosed spaces or overlain by moisture sensitive floor coverings should be discouraged by providing a moisture vapor retarding system between the subgrade soil and the bottom of slabs. We recommend that the moisture vapor retarding system consist of a 4-inch-thick capillary break, overlain by a 15-mil-thick plastic membrane. Sand should not be placed over the vapor retarder. The capillary break should be constructed of clean, compacted, open-graded crushed rock or angular gravel of ¾-inch nominal size. The crushed rock or angular gravel should be compacted with a vibratory plate compactor or roller to reduce the potential for damage to the vapor retarder by rock puncture during placement of reinforcement and concrete. The plastic membrane should conform to the requirements in the latest version of ASTM Standard E 1745 for a Class A membrane. To reduce the potential for slab curling and cracking, an appropriate concrete mix with low shrinkage characteristics and a low water-to-cementitious-materials ratio should be specified. In addition, the concrete should be delivered and placed in accordance with ASTM C94 with attention to concrete temperature and elapsed time from batching to placement, and the slab should be cured in accordance with the guidelines of ACI Committee 302.

Where the exterior grade is at a higher elevation than the moisture vapor retarding system (including the capillary break layer), consideration should be given to constructing a subdrain around the foundation perimeter. The subdrain should consist of ¾-inch crushed rock wrapped in filter fabric (Mirafi 140N, or equivalent). The subdrain should be 12-inches wide capped by a pavement or 12 inches of native soil and drained by a 4-inch perforated pipe (Schedule 40 polyvinyl chloride pipe, or similar) with the perforations facing down. The pipe should be sloped at 1 percent or more to discharge at an appropriate outlet away from the foundation. The pipe should be located below the bottom elevation of the moisture vapor retarding system but above a plane extending down and away from the bottom edge of the foundation at a 2:1 (horizontal to vertical) gradient. A sump may be used where gravity drainage is not feasible. The effectiveness of a perimeter subdrain pipe should be considered relative to the invert elevation of the subdrain and anticipated depth of groundwater.

8.4 Retaining Walls Including Pool Side Walls

Walls backfilled with imported select fill and retaining up to 10 feet of soil above the wall footing with level backfill may be designed for active or at-rest equivalent fluid earth pressures of 82 or 91 psf per foot depth (below design groundwater level) and 40 or 60 pcf EFW for drained conditions or above design groundwater level. Walls that yield or deflect may be designed for active earth pressures. Wall deflection equivalent to about 1 percent of wall height may be needed to reduce at-rest earth pressures to active earth pressures. Walls that are restrained by abutting walls should be designed to resist at-rest earth pressures. An additional equivalent fluid pressure of 14 psf per foot depth may be used to evaluate seismic earth pressure on retaining walls, as appropriate, for consideration with active earth pressures.

Walls retaining level ground should be designed to resist construction or live load surcharges on the backfill. The lateral earth pressure due to a backfill surcharge of 240 psf should be a uniform horizontal surcharge of 80 psf for yielding conditions and 120 psf for at-rest conditions.

Pool walls should be designed for at-rest earth pressures, using the recommended values above.

Wall height should be evaluated as the vertical distance above the wall footing to the ground surface at the heel of the wall.

8.5 Hydrostatic Uplift

The design of the swimming pool and underground utilities should consider hydrostatic uplift and include appropriate design measures to resist hydrostatic uplift. We recommend a design groundwater level of 3 feet or higher below existing grade and take into consideration potentially higher groundwater levels due to sea level rise over the life of the project. Hydrostatic uplift can be resisted by a combination of dead weight, tie-down elements, and lowered groundwater level. The pools should be designed to resist hydrostatic uplift when the pool is empty for repair or maintenance as discussed in Section 6.12. The improved soil below the pool and underground utilities may be constructed with tie-down features that can be structurally connected to the pool shell and underground utilities. Alternatively, the pool shell and/or structurally incorporated pool base mat can be thickened to increase the dead weight to restrict hydrostatic uplift. Extending the limits of the pool base mat beyond the pool limits and utilizing the effective weight of the overlying soils is another way providing additional resistance to uplift loads. Deep wells can be installed near the pool to lower the groundwater level and reduce hydrostatic uplift, and hydrostatic relief valves should be installed in the pools to discharge groundwater into the pools, if needed.

8.6 Tiedown Anchors (Preliminary Recommendations)

Tiedown anchors consisting of a bar tendon installed in a drilled hole backfilled with grout, may be used to provide tensile resistance to uplift. A smooth plastic sleeve should be provided over the tendon to create an unbonded zone that extends no less than __ feet for bar tendons, and no less than __ feet below the ground surface. Gravity-grouted tiedown anchors embedded ___ feet below grade with a bonded length of no less than __ feet may be designed for an allowable grout-to-ground bond strength of _____ psf with a safety factor of 2. Pressure grouting during initial grout placement or during one or more post-grouting operations, may be performed below a depth of __ feet from the ground surface to enhance pullout resistance. The allowable grout-to-ground bond strength may be increased to ______ psf, with a safety factor of 2, where pressure grouting is performed to enhance pullout resistance with injection pressures of 150 pounds per square inch (psi) or more.

Tensile static load testing should be performed to check the design assumptions and installation methods. Verification testing to 200 percent of the design load (DL) should be performed on pre-production tiedown anchors. Verification testing should be performed on each anchor type (combination of bonded length, nominal diameter, tendon size, or installation method) specified with no less than two verification tests. Verification testing above 200 percent DL, but not more than 80 percent of the specified minimum tensile strength for the tendon, may be performed to justify an increase in the assumed grout-to-ground bond strength. Five percent of the production anchors should be proof tested to 160 percent DL. The foundation contractor's testing equipment should include dial gages capable of measuring to 0.001 inches with sufficient range for the anticipated movement, dial gage supports, jack with pressure gage, electronic load cell for verification creep testing, and a reaction frame. The hydraulic jack and pressure gage should have a range not exceeding twice the maximum test pressure and the gage should be graduated in increments of 100 psi or less.

The load testing should conform with Federal Highway Administration (FHWA) guidelines. The verification testing should consist of no less than three, progressively increasing load cycles from the alignment load (AL) or 5 percent DL to 130 percent DL in loading increments of not more than 15 percent DL with a hold time of 2.5 minutes at each load and a 60-minute creep test at 130 percent DL. Anchor head movement should be measured and recorded following the 2.5-minute hold at each load and at 1, 2, 3, 4, 5, 6, 10, 20, 30, 40, 50, and 60 minutes during the creep test. Following the creep test, the test load should be increased to 145 percent DL before beginning a final load cycle from the alignment load to 200 percent DL in increments of not more than 15 percent DL, with a 10-minute hold time at 200 percent. After the 10-minute hold, the test load should be reduced in increments of not more the 50 percent DL with a 5-minute hold at each load increment.

The proof testing should consist of one load cycle from AL up to 130 percent DL with load increments of not more than 15 percent DL. Anchor head movement should be measured and recorded following a 2.5-minute hold at each intermediate load with a 10- or 60-minute creep test at 130 percent DL. Anchor head movement should be measured and recorded at 1, 2, 3, 4, 5, 6, and 10 minutes during the creep test with additional measurements at 20, 30, 40, 50, and 60 minutes if the test is extended. Following the creep test, the test load should be increased to 160 percent DL in increments of not more than 15 percent DL with a 2.5-minute hold at each increment before reducing the test load in increments of not more the 30 percent DL with a 4-minute hold at each load increment.

The acceptance criteria for verification and proof testing should consist of (1) the apparent free length at 130 percent DL exceeds the jack length plus 80 percent of the unbonded length, (2) anchor movement of not more than 0.04 inches during the creep test between the 1-minute and 10-minute readings or not more than 0.08 inches between the 6- and 60-minute reading, and (3) the maximum test load can be held for 10 minutes during the verification test or 2.5 minutes during the proof test without continued anchor movement.

Class I Corrosion Protection with encapsulated tendons over the unbonded length and encapsulation or fusion bonded epoxy coatings over the bonded length should be provided for tiedown anchors to mitigate against potentially aggressive ground conditions.

Tiedown ground anchors may be prestressed with a nominal lock off load of up to 10 kips. If a greater lock-off load is needed, the impact on footing settlement should be re-evaluated.

Drilling to install the tiedown anchors will likely encounter cohesionless soil and groundwater. Overburden drilling techniques that advance casing, such as rotary duplex or sonic drilling, may be needed to mitigate unstable conditions. Hollow stem auger or open-hole rotary drilling stabilized with a polymer-water slurry may also be considered.

The nominal diameter of the drilled hole should be about 6 inches or more. The center-to-center spacing between adjacent tiedown anchors should be at least 30 inches and not less than three times the nominal diameter of the drilled hole. For groups of three or more tiedown anchors, the allowable group uplift resistance should not be more than the sum of the allowable uplift resistance for each anchor in the group, and not more than two-thirds of the sum of the effective weight of the soil block defined by the group plus the ultimate shear resistance around the perimeter of the block. An effective average soil unit weight of ___ pounds per cubic foot and an ultimate shear resistance of ___ pounds per square foot per foot below finish grade may be used for this calculation.

The foundation contractor should submit detailed plans for the anchor load testing, calibration reports for the test equipment, and a grouting plan. The grouting plan should include the grout mix design; methods and equipment for monitoring grout depth, volume, and pressure during placement; grouting rate calculations; estimated grout curing time; and procedures for contractor monitoring of grout quality.

The foundation contractor should select the drilling method, grouting procedure, and grouting pressure used for installation of the anchors. The foundation contractor is responsible for estimating grout take. Drilled holes that encounter groundwater or cohesionless soil may be unstable and may need to be stabilized by temporary casing or use of drilling mud. Addition of bentonite to the drilling mud or slurry is not recommended. The foundation contractor should monitor the conditions in the vicinity of the installation on a daily basis for signs of ground heave or subsidence.

Anchors should be primary grouted on the day the bonded length is drilled. The contractor should monitor and measure grout quantity and pressure during grouting operations. Gages for monitoring grout pressures should be capable of measuring not less than 150 psi or twice the actual grout pressures used. Each anchor should be primary grouted in one operation within one hour of grout mixing. The primary grout should be injected from the lowest point of the drilled holes until uncontaminated grout flows from the top of the hole while the tremie pipe extends below the level of the grout in the hole. Temporary casing, if used, should be extracted in stages so that after each length of casing is removed, the grout level is brought back up to the top of the hole. The grout pressures and grout takes should be controlled to reduce the potential for heave or fracturing of rock or soil.

The geotechnical engineer of record should be retained to observe anchor drilling, tendon installation, grouting, load testing, and prestressing.

8.7 Pavements and Flatwork

Recommendations for pavement (rigid and flexible) and exterior flatwork are presented in the following sections. A design R-value of 12 was selected based on the results of the R-Value testing performed for this study (Appendix C). The pavement subgrade should be evaluated by the geotechnical engineer during grading to check the finish subgrade for consistency with the assumed condition. Finished grades should be reviewed relative to thickness of existing fill and depth to weak subgrade soils which may require mitigation for construction equipment access. Recommendations for preparation of subgrade are presented in Section 8.2.7.

8.7.1 Asphalt Concrete Pavement

Recommended asphalt pavement sections based on the empirical procedure in the Highway Design Manual (Caltrans, 2020) are presented in Table 11 for a range of traffic indexes. Alternative sections are provided for consideration. The designer may interpolate between the values provided once a traffic index has been selected. The pavement sections were designed for a 20-year service life presuming that periodic maintenance, including crack sealing and resurfacing will be performed during the service life of the pavement. Premature deterioration may occur without periodic maintenance.

Laboratory testing for this evaluation indicated that some of the near-surface site soil is highly expansive. Seasonal variations in soil moisture, particularly near the edge of pavement, may result in differential vertical and lateral movement with seasonal shrinkage and swelling of the expansive soil. The potential degree of differential movement from shrinkage/swelling of expansive subgrade soil can be reduced, where desirable, by chemically treating the subgrade with quicklime. Where the expansion characteristic of the pavement subgrade is not mitigated by chemical treatment and the pavement is not laterally restrained by curbs, the potential for longitudinal cracking from differential lateral movement can be mitigated by placing a layer of geotextile (Mirafi 600X or equivalent) below the aggregate base layer.

Table 11 – Asphalt Concrete Pavement Structural Sections

Design R-Value	Traffic Index	Option 1	Option 2	Option 3
12	3	4 inches AC[1]	2 inches AC 4 inches AB[2]	4 inches AC 18 inches TS[3]
12	4	5¾ inches AC	2½ inches AC 6 inches AB	5 inches AC 18 inches TS
12	5	7½ inches AC	3 inches AC 8½ inches AB	5½ inches AC 18 inches TS
12	6	9 inches of AC	4 inches AC 10 inches AB	7 inches AC 18 inches TS
12	7	10½ inches AC	4½ inches AC 13 inches AB	8½ inches AC 18 inches TS

Notes:
1 AC is Type A, Dense-Graded Hot Mix Asphalt complying with Caltrans Standard Specification 39-2 (2022).
2 AB is Class II Aggregate Base complying with Caltrans Standard Specification 26-1.02 (2022).
3 TS is subgrade chemically treated with quicklime.

Asphalt concrete should be placed and compacted per in lifts not more than 4 inches thick to 91 percent of the reference density as evaluated by ASTM D2041 on a wet density basis. Pavements should be sloped so that runoff is diverted to an appropriate collector (concrete gutter, swale, or area drain) to reduce the potential for ponding of water on the pavement.

Concentration of runoff over asphalt pavement should be discouraged. Cracks that form in the asphalt concrete surface should be periodically sealed to reduce moisture intrusion into the aggregate base section. Deep curbs that extend 6 inches below the aggregate base section may be used to reduce the potential moisture intrusion into the aggregate base section adjacent to landscaped areas or the bottom of slopes. Subdrains may be considered as a supplement or alternative means of the mitigating moisture in the aggregate base section. Root barriers adjacent to trees may be considered to reduce the potential for pavement heave from root growth.

8.7.2 Exterior Flatwork

Walkways and other exterior flatwork not subject to vehicular loading should be 4 inches thick (or more) over 4 inches of aggregate base that conforms to the criteria for Class 2 aggregate base in Section 26-1.02 of the California Standard Specifications (Caltrans, 2023). Concrete and aggregate base thickness should be increased to 8 inches or more for flatwork subject to vehicular traffic up to periodic garbage trucks and emergency vehicles.

Appropriate jointing of concrete flatwork can encourage cracks to form at joints, reducing the potential for crack development between joints. Joints should be laid out in a square pattern at consistent intervals. Contraction and construction should be detailed and constructed in accordance with the guidelines of ACI Committee 302. The ratio of lateral spacing between contraction joints to the nominal thickness of the slab should not exceed 24 for jointed plain concrete. Contraction joints formed by premolded inserts, grooving plastic concrete, or saw-cutting at initial hardening, should extend to a depth equivalent to 25 percent of the slab thickness and 1 inch or more for thin slabs.

Flatwork may be reinforced with distributed steel to reduce potential for differential slab movement where cracking occurs. The distributed reinforcing steel should be terminated about 3 inches from contraction joints and should consist of No. 3 deformed bars at 18 inches on center, both ways, or with 6x6-D4/D4 welded wire fabric supplied as sheets (not rolls). Slabs reinforced with distributed steel should be 6 inches thick (or more) for No. 3 bar reinforcement and 5 inches thick (or more) for 6x6-D4/D4 reinforcement to provide adequate concrete cover for the steel. The lateral spacing between contraction joints should be 10 feet or less for a 5-inch-thick slab, and 12 feet or less for a 6-inch-thick slab. To reduce the potential for differential slab movement across joints, the distributed steel may be extended through the joints. This improvement will be balanced by a reduction in the functionality of the contraction joint to encourage crack formation at joints. Masonry briquettes or plastic chairs should be used to maintain the position of the reinforcement in the upper half of the slab with 1½ inches of cover over the steel. Root barriers adjacent to trees may be considered to reduce the potential for flatwork heave from root growth.

8.8 Concrete

Laboratory testing indicated that the concentration of sulfate and corresponding potential for sulfate attack on concrete is negligible for the soil tested. However, due to the potential variability in the on-site soil and the potential future use of reclaimed water at the site, we recommend that Type II/V or Type V cement be used for concrete structures in contact with soil. In addition, we recommend a water-to-cement ratio of no more than 0.45. A 3-inch thick, or thicker, concrete cover should be maintained over reinforcing steel where concrete is in contact with soil in accordance with recommendations of ACI Committee 318.

To reduce the potential for shrinkage cracks in the concrete during curing, concrete for slabs and flatwork should not contain large quantities of water or accelerating admixtures containing calcium chloride. Higher compressive strengths may be achieved by using larger aggregates in lieu of increasing the cement content and corresponding water demand. Additional workability, if desired, may be obtained by including water-reducing or air-entraining admixtures. Concrete should be placed in accordance with the guidelines of ACI Committee 302 and project specifications. Particular attention should be given to curing techniques and curing duration. Slabs that do not receive adequate curing have a more pronounced tendency to curl upwards at edges and corners, and to develop random shrinkage cracks and other defects.

In the event that contraction joints are used to influence the location of crack development in slabs and the joints are to be constructed by saw cutting of the slabs, saw cuts should be made by soff-cut sawing within 4 to 12 hours after the initial hardening (not curing) of the concrete, as required by atmospheric conditions. The contractor should be responsible for monitoring of the concrete during initial set or hardening and selecting the appropriate time for cutting the slabs.

8.9 Surface Drainage and Site Maintenance

Surface drainage on the site should generally be provided so that water is diverted away from structures and is not permitted to pond. Positive drainage should be established adjacent to structures to divert surface water to an appropriate collector (graded swale, v-ditch, or area drain) with a suitable outlet. Drainage gradients should be 2 percent or more for a distance of 5 feet or more from the structure for impervious surfaces and 5 percent or more for a distance of 10 feet or more from the structure for pervious surfaces. Slope, pad, and roof drainage (from adjacent structures) should be collected and diverted to suitable discharge areas away from structures or other slopes by non-erodible devices (e.g., gutters, downspouts, concrete swales, etc.). Graded swales, v ditches, or curb and gutter should be provided at the site perimeter to restrict flow of surface water onto and off of the site. Slopes should be vegetated or otherwise armored to reduce potential for erosion of soil. Drainage structures should be periodically cleaned out and repaired, as-needed, to maintain appropriate site drainage patterns.

Landscaping adjacent to foundations should include vegetation with low-water demands and irrigation should be limited to that which is needed to sustain the plants. Trees should be restricted from the areas adjacent to foundations a distance equivalent to the canopy radius of the mature tree. Infiltration basins, dry wells, and other stormwater management measures that rely on infiltration without a liner and subdrain should not be located within 20 feet of structure foundations. Bioretention planters located within 10 feet of structure foundations should be lined with concrete or a plastic membrane and include a subdrain.

Care should be taken by the contractor during grading to preserve any berms, drainage terraces, interceptor swales or other drainage devices on or adjacent to the project area. Drainage patterns established at the time of grading should be maintained for the life of the project. Future alteration of the established drainage patterns may impact the constructed improvements.

8.10 Geotechnical Engineer of Record

The recommendations provided in this report are based on preliminary design information for the proposed construction. The Geotechnical Engineer-of-Record (GEOR) should review the plans that are developed by the design team before construction bidding, to check that the scope of the project as designed is consistent with the assumed basis of this report and evaluate conformance with the geotechnical recommendations.

During construction, the GEOR should evaluate the exposed subsurface conditions for consistency with the conditions encountered in the discrete borings performed for this study, and to check that the work conforms with the geotechnical recommendations. Specifically, the geotechnical engineer should be retained to:

- Observe installation of ground improvement and tie-downs.
- Observe preparation and compaction of subgrade.
- Check and test imported materials prior to use as fill.
- Observe placement and compaction of fill and aggregate base.
- Perform field density tests to evaluate fill and subgrade compaction.
- Check foundation excavations for bearing materials and cleaning prior to placement of reinforcing steel and concrete.

The recommendations provided in this report assume that Ninyo & Moore will be retained as the geotechnical consultant during the construction phase of the project. If another geotechnical consultant is selected, we request that the selected consultant provide a letter to the architect and the owner (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the recommendations contained in this report.

9 LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical evaluation report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations will be provided, as appropriate. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur because of government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

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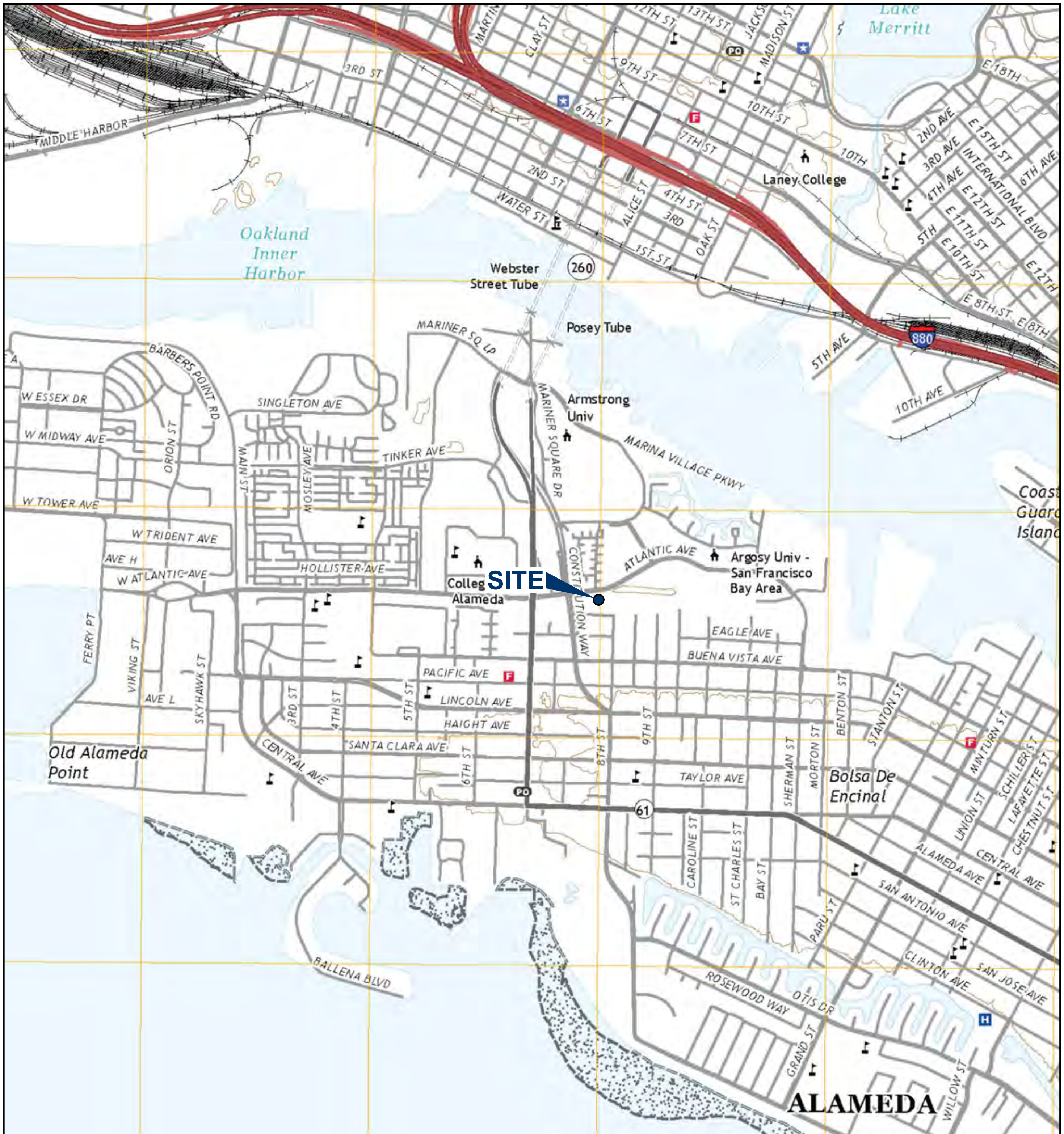
FIGURES



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NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE | REFERENCE: USGS, 2018

N

SCALE (FEET)

0 2,000 4,000

FIGURE 1

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SITE LOCATION

CITY OF ALAMEDA NEW AQUATICS CENTER
1100 ATLANTIC AVENUE
ALAMEDA, CALIFORNIA
403773009 | 5/25

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE | REFERENCE: GOOGLE EARTH, 2024

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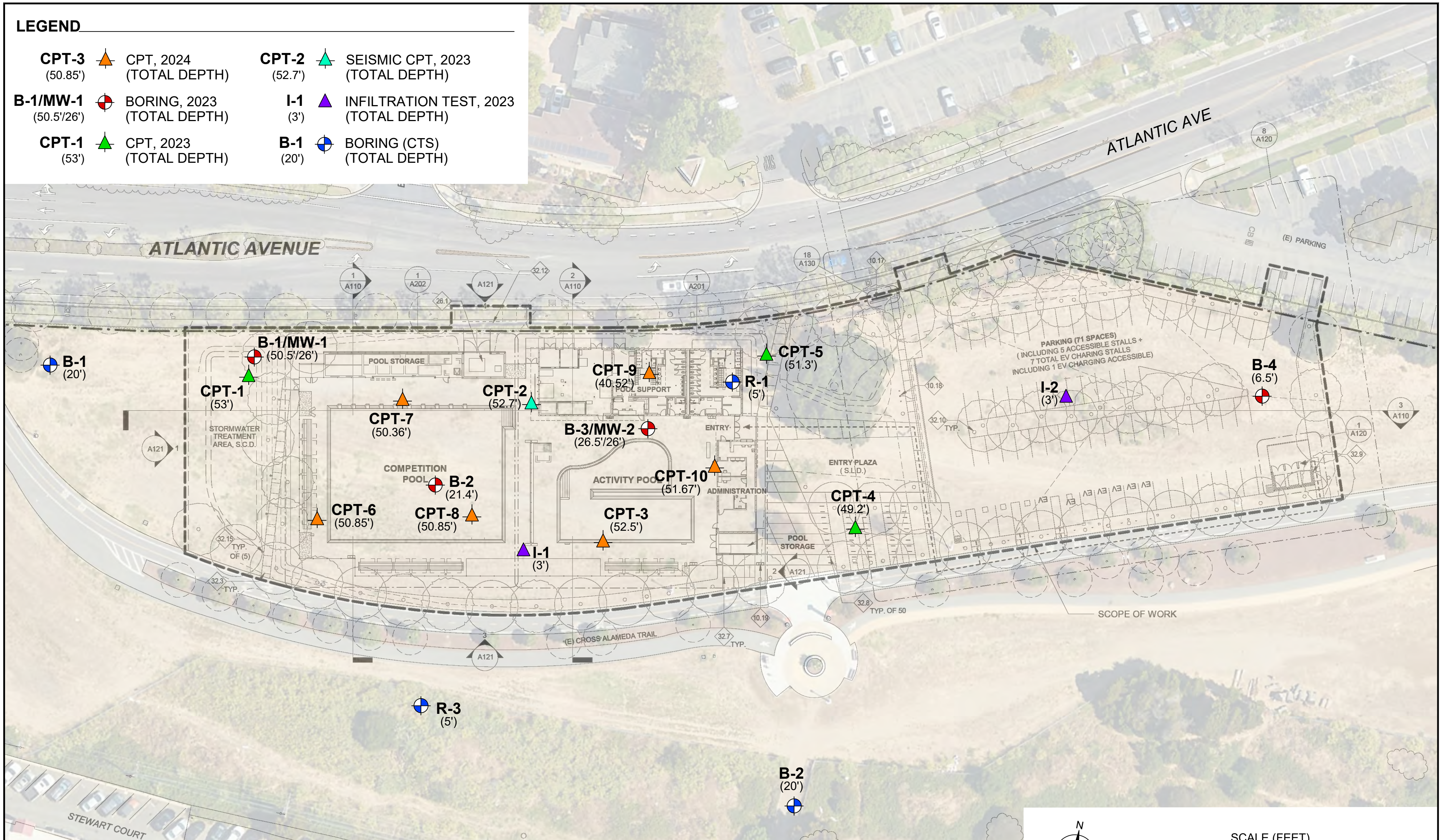
FIGURE 2

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

EXPLORATION LOCATIONS

CITY OF ALAMEDA NEW AQUATICS CENTER
1100 ATLANTIC AVENUE
ALAMEDA, CALIFORNIA
403773009 | 5/25



NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE I REFERENCES: ELS ARCHITECTURE AND URBAN DESIGN, 2025; GOOGLE EARTH, 2024

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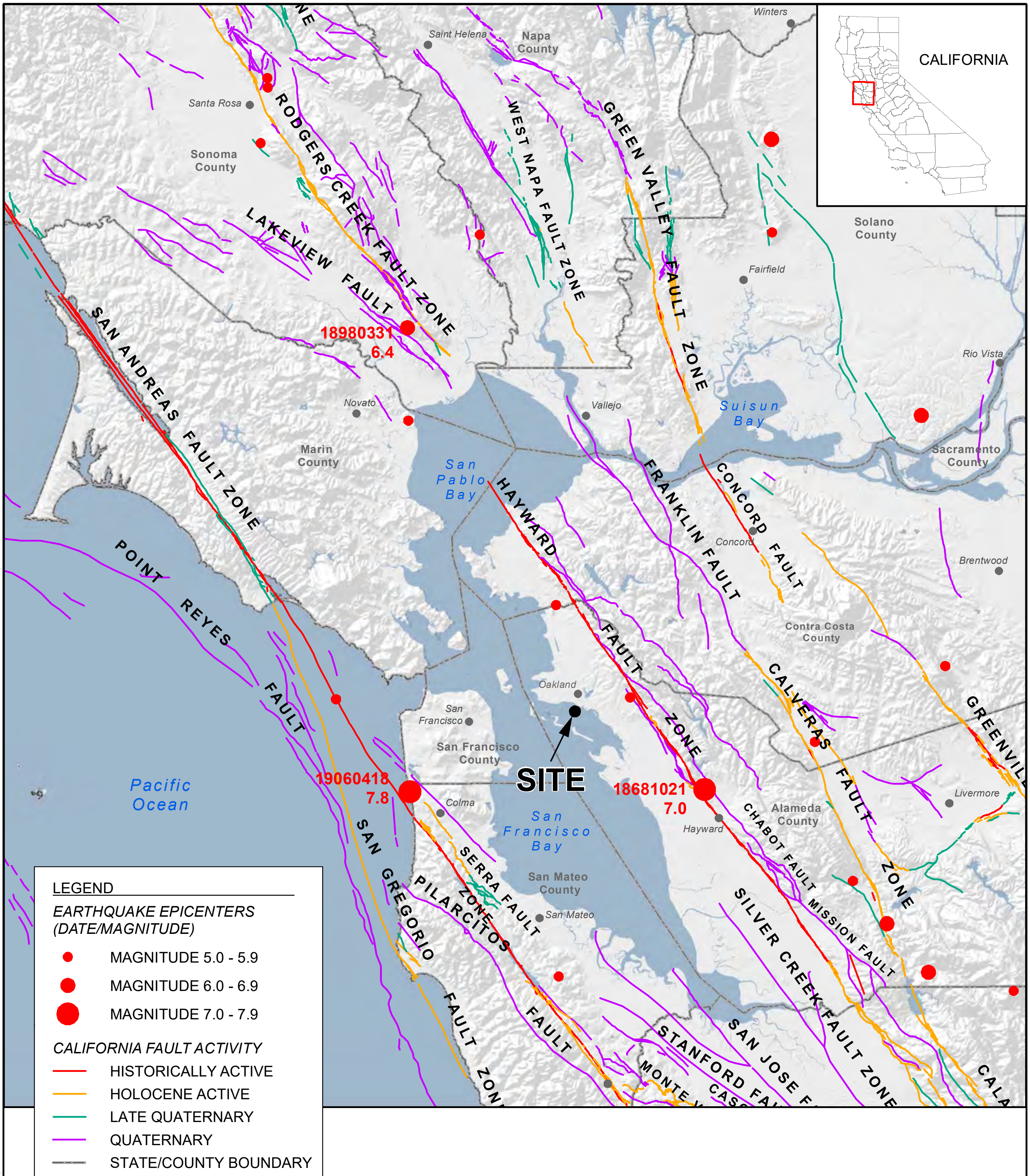
FIGURE 3

SITE PLAN WITH EXPLORATION LOCATIONS

CITY OF ALAMEDA NEW AQUATICS CENTER
1100 ATLANTIC AVENUE
ALAMEDA, CALIFORNIA
403773009 I 05/25

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



NOTE: DIRECTIONS, DIMENSIONS, AND LOCATIONS ARE APPROXIMATE

SOURCES: CALIFORNIA GEOLOGICAL SURVEY, 2010, FAULT ACTIVITY MAP OF CALIFORNIA;
CALIFORNIA GEOLOGICAL SURVEY, 2000, MAP SHEET MS 49

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FIGURE 4

FAULT LOCATIONS AND EARTHQUAKE EPICENTERS

CITY OF ALAMEDA NEW AQUATIC CENTER
1100 ATLANTIC AVENUE
ALAMEDA, CALIFORNIA

403773009 | 5/25

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Geotechnical & Environmental Sciences Consultants

LEGEND

af	ARTIFICIAL FILL (HISTORIC)	Qpaf	ALLUVIAL FAN & FLUVIAL DEPOSITS (PLEISTOCENE)
Qds	DUNE SAND (HOLOCENE/ PLEISTOCENE)	Qmt	MARINE TERRACE DEPOSITS (PLEISTOCENE)
Qms	MERRIT SAND (HOLOCENE/ PLEISTOCENE)		

THRUST FAULT

FAULT

GEOLOGIC CONTACT

STRIKE AND DIP OF BEDDING

N

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NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE I REFERENCE: USGS, GRAYMER, 2000

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FIGURE 5

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Geotechnical & Environmental Sciences Consultants

REGIONAL GEOLOGY

CITY OF ALAMEDA NEW AQUATICS CENTER
1100 ATLANTIC AVENUE
ALAMEDA, CALIFORNIA

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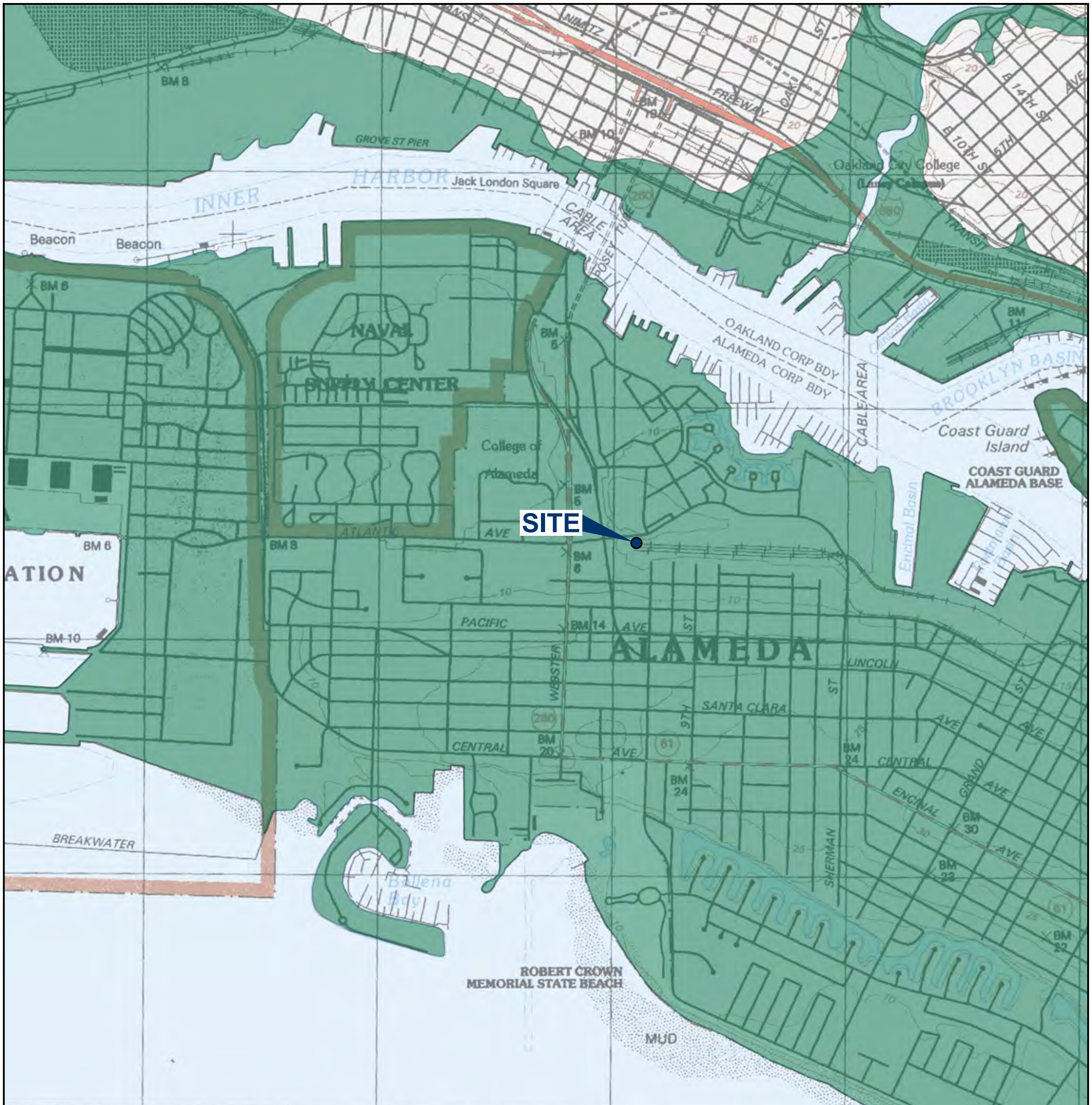

LEGEND

LIQUEFACTION ZONES:
Areas where historic occurrence of liquefaction, or local geological, geotechnical, and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE | REFERENCE: CGS; 1982, 2003

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FIGURE 6

SEISMIC HAZARD ZONES

CITY OF ALAMEDA NEW AQUATICS CENTER
1100 ATLANTIC AVENUE
ALAMEDA, CALIFORNIA
403773009 | 5/25

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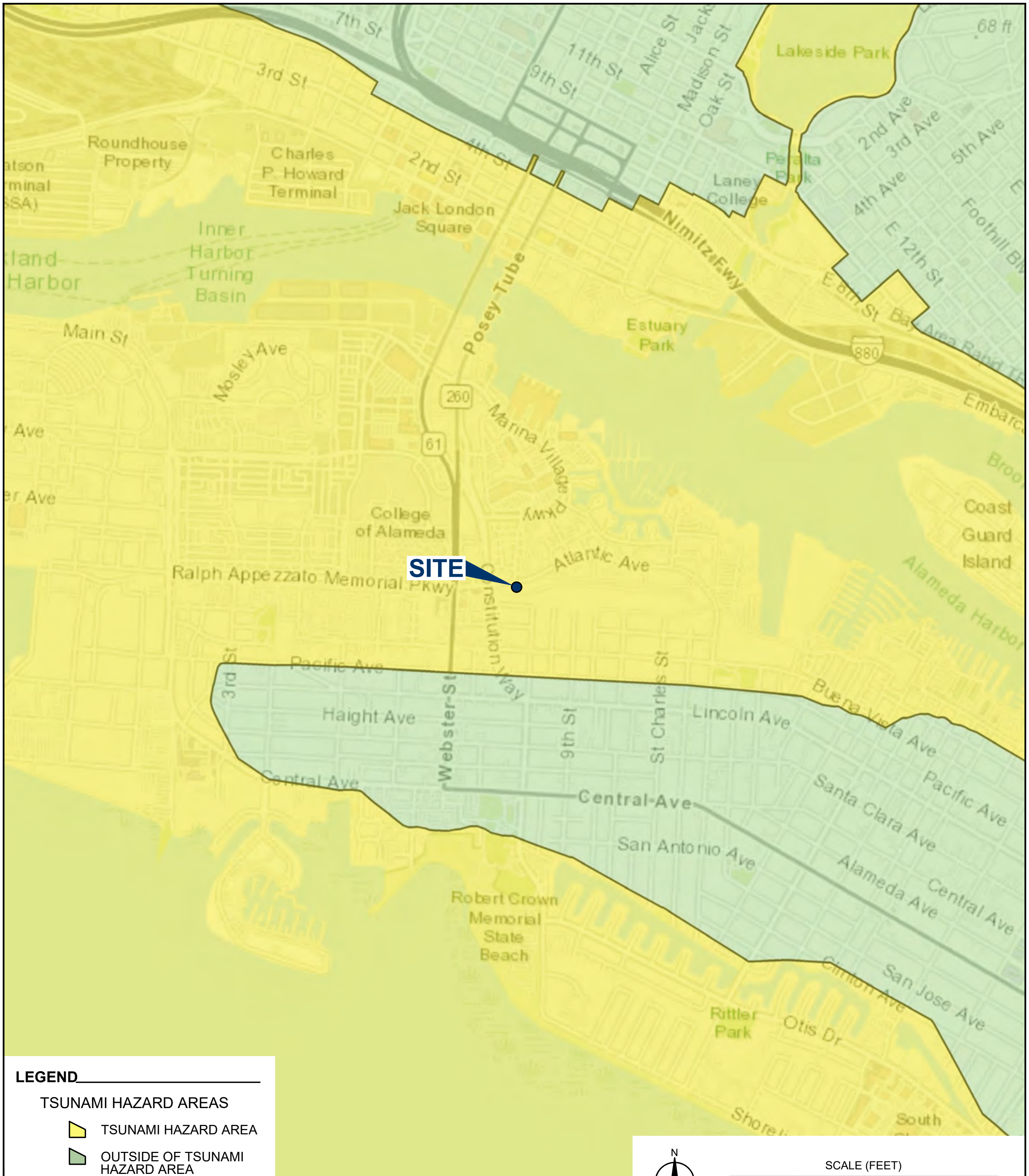
Geotechnical & Environmental Sciences Consultants



LEGEND

TSUNAMI HAZARD AREAS

- TSUNAMI HAZARD AREA
- OUTSIDE OF TSUNAMI HAZARD AREA

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SCALE (FEET)

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE I REFERENCE: CGS; 2021

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FIGURE 7

TSUNAMI INUNDATION

CITY OF ALAMEDA NEW AQUATICS CENTER
1100 ATLANTIC AVENUE
ALAMEDA, CALIFORNIA
403773009 I 5/25

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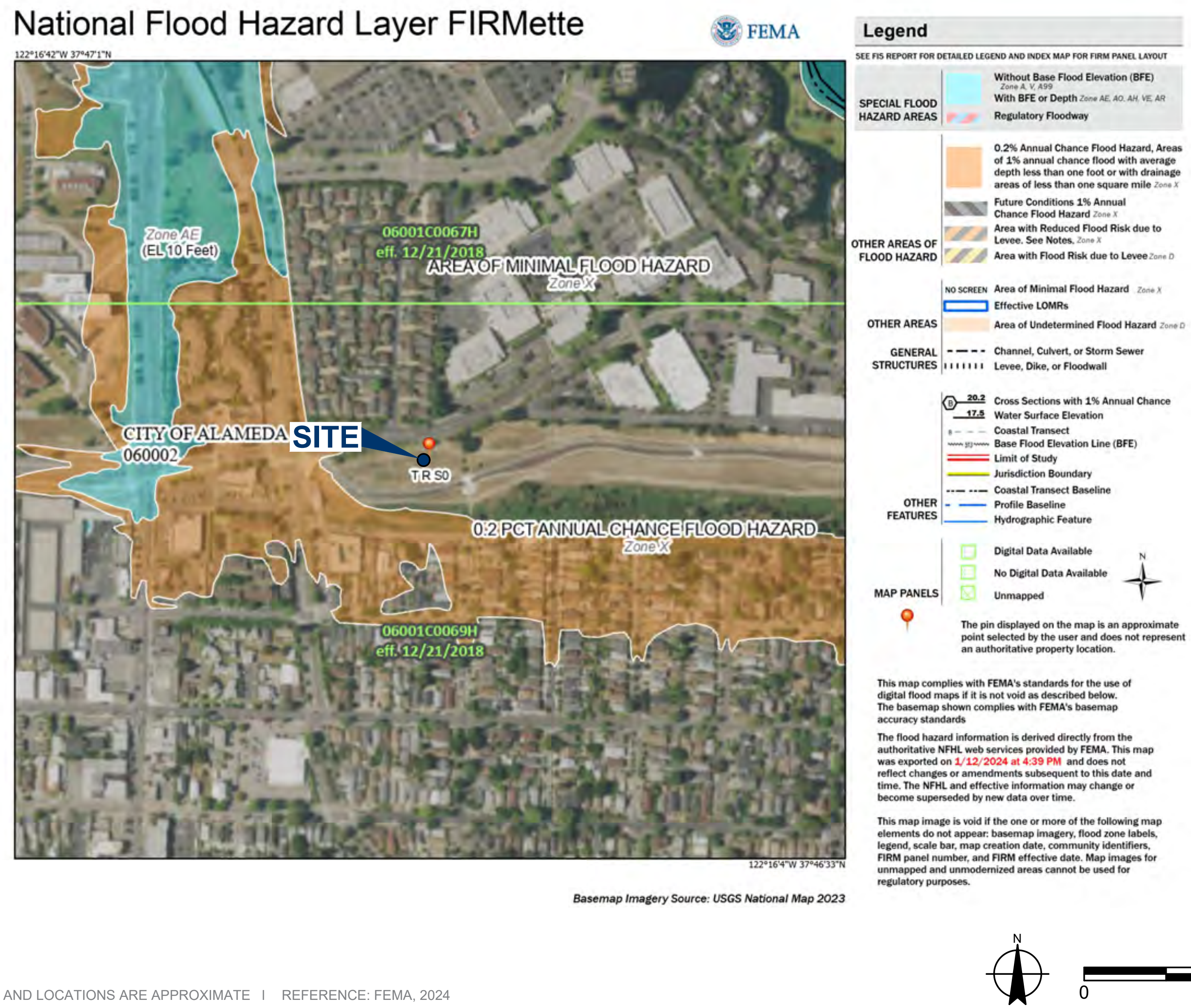

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NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE | REFERENCE: FEMA, 2024

FIGURE 8

Ninyo & Moore
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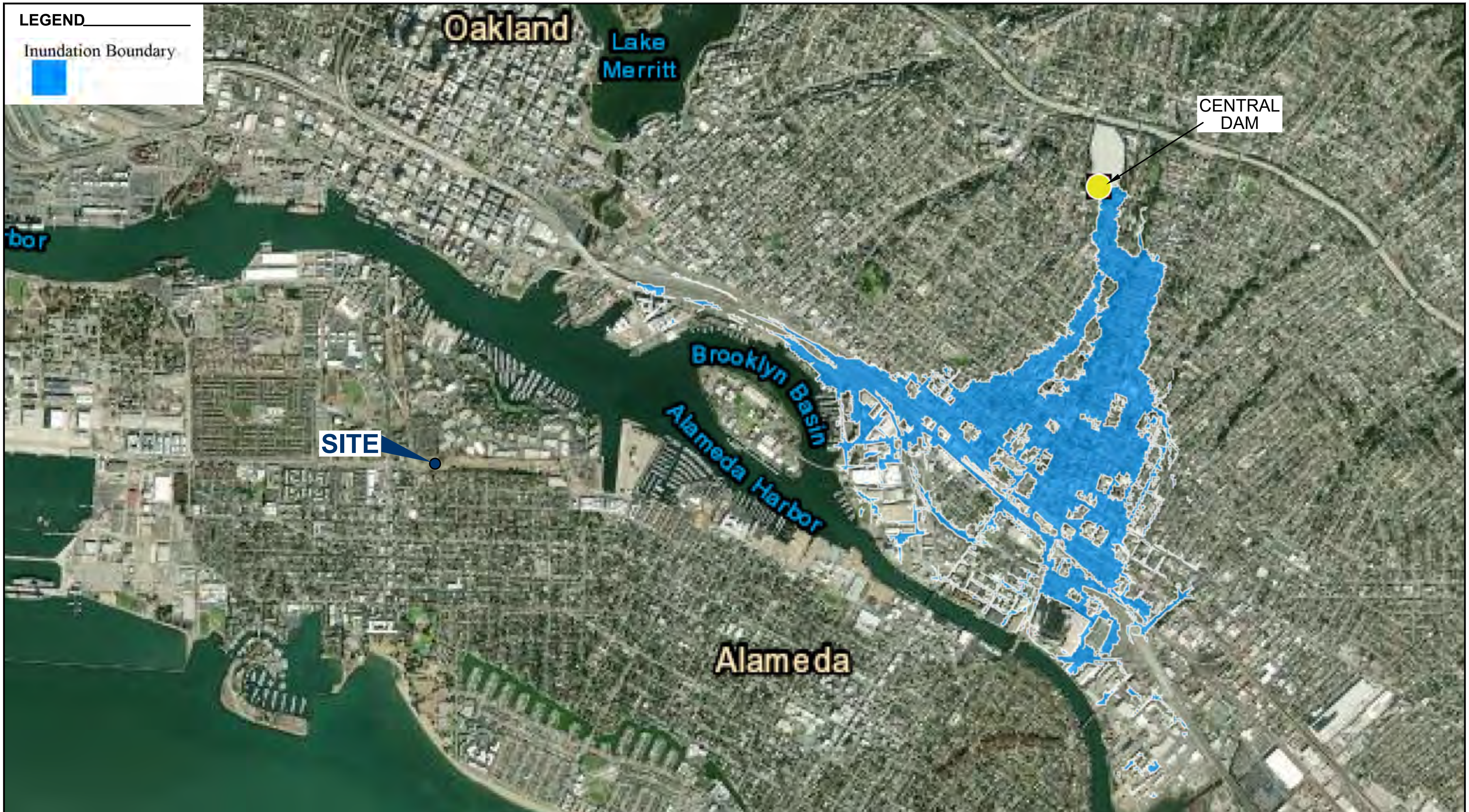
FEMA FLOOD HAZARD ZONES

CITY OF ALAMEDA NEW AQUATICS CENTER
1100 ATLANTIC AVENUE
ALAMEDA, CALIFORNIA
403773009 | 525



NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE | REFERENCES: BLM, ESRI, 2024

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SCALE (FEET)
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FIGURE 9

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DAM INUNDATION MAP

CITY OF ALAMEDA NEW AQUATICS CENTER
1100 ATLANTIC AVENUE
ALAMEDA, CALIFORNIA
403773009 | 525

APPENDIX A

Cone Penetrometer Test (CPT) Logs

Ninyo & Moore

https://ninyoandmoore.com/

CPT: CPT-01

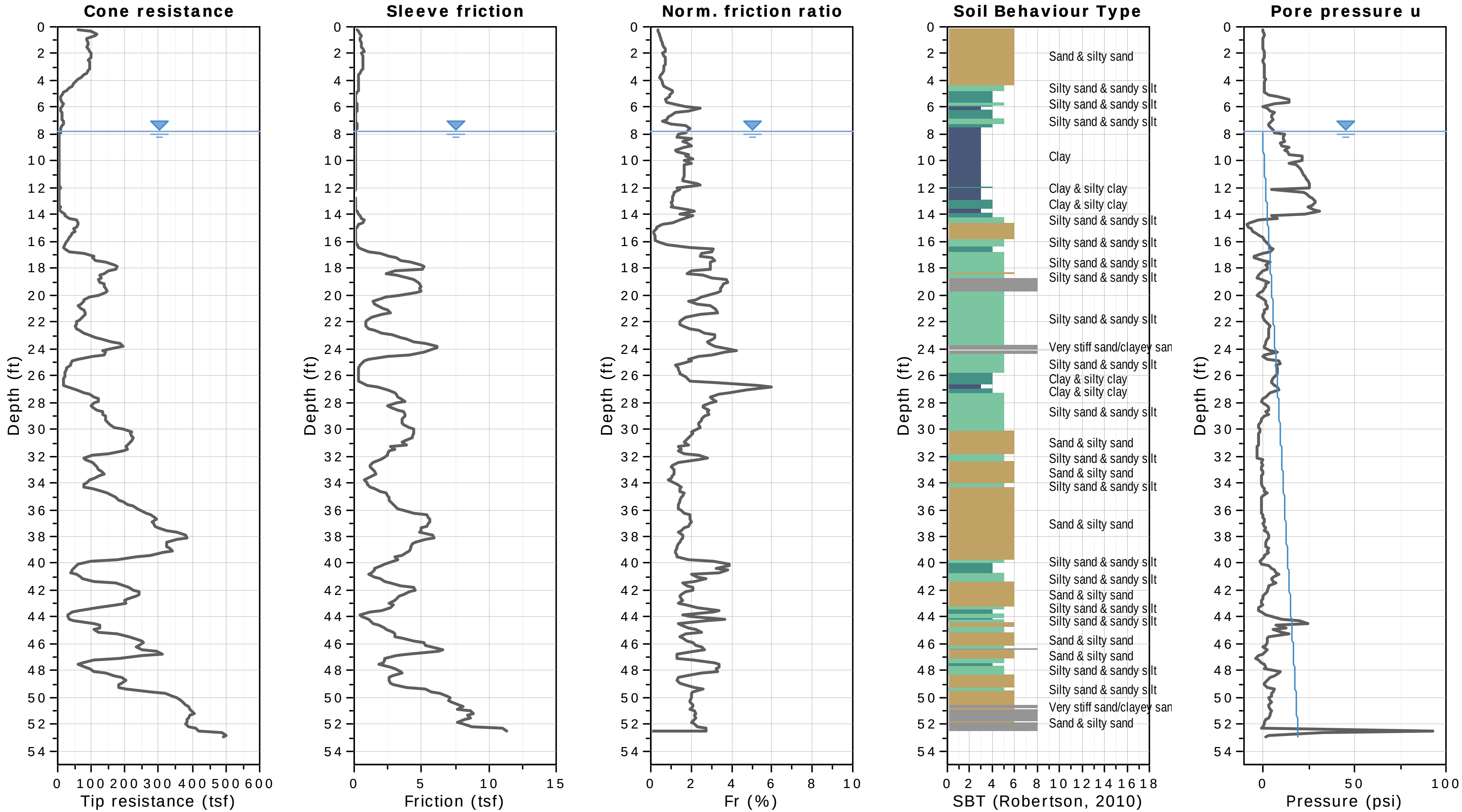
Total depth: 52.99 ft, Date: 10/23/2023

Cone Type: 15cm²

Cone Operator: AJ-ER

Project: Alameda Aquatic Center

Location: 1100 Atlantic Avenue, Alameda, CA 94501



Ninyo & Moore

https://ninyoandmoore.com/

CPT: CPT-02

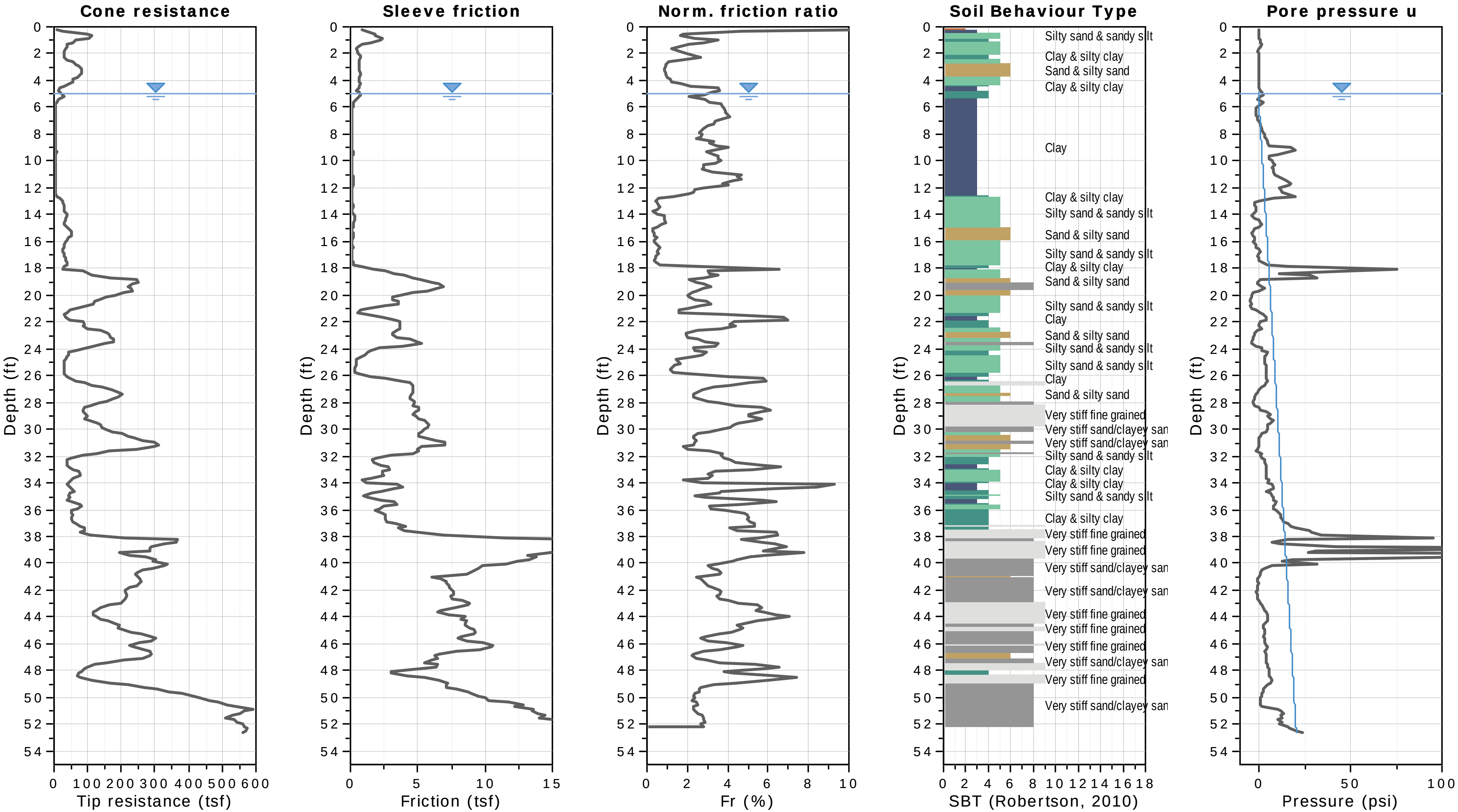
Total depth: 52.66 ft, Date: 10/23/2023

Cone Type: 15cm²

Cone Operator: AJ-ER

Project: Alameda Aquatic Center

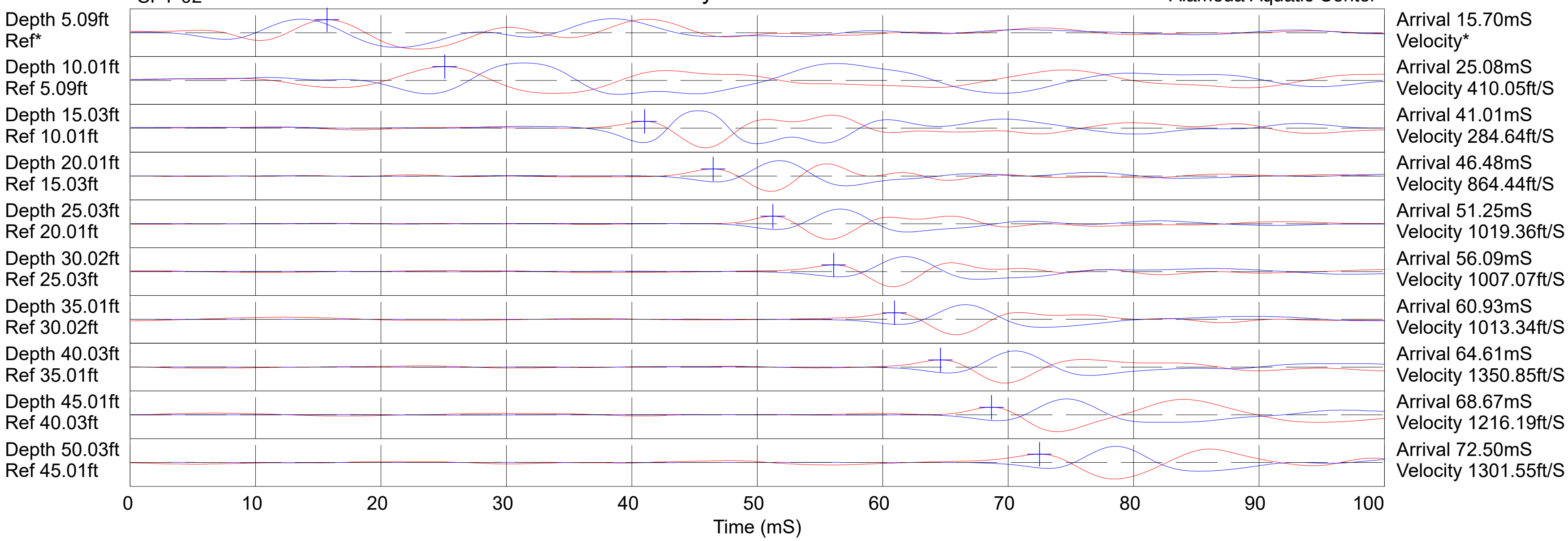
Location: 1100 Atlantic Avenue, Alameda, CA 94501



CPT-02

Ninyo and Moore

Alameda Aquatic Center



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

Project: Alameda Aquatic Center

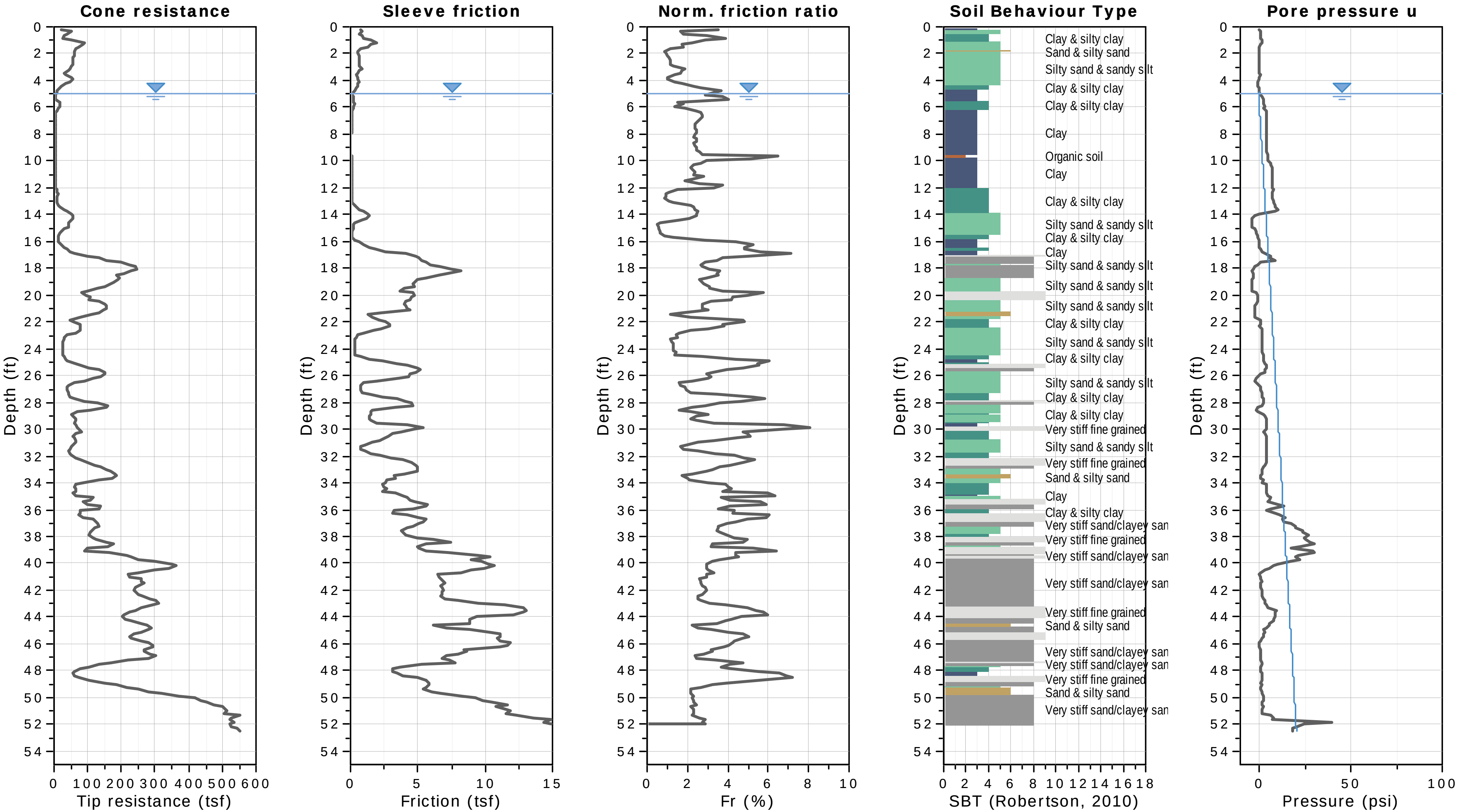
Location: 1100 Atlantic Avenue, Alameda, CA 94501

CPT: CPT-03

Total depth: 52.49 ft, Date: 10/23/2023

Cone Type: 15cm²

Cone Operator: AJ-ER



Ninyo & Moore

https://ninyoandmoore.com/

CPT: CPT-04

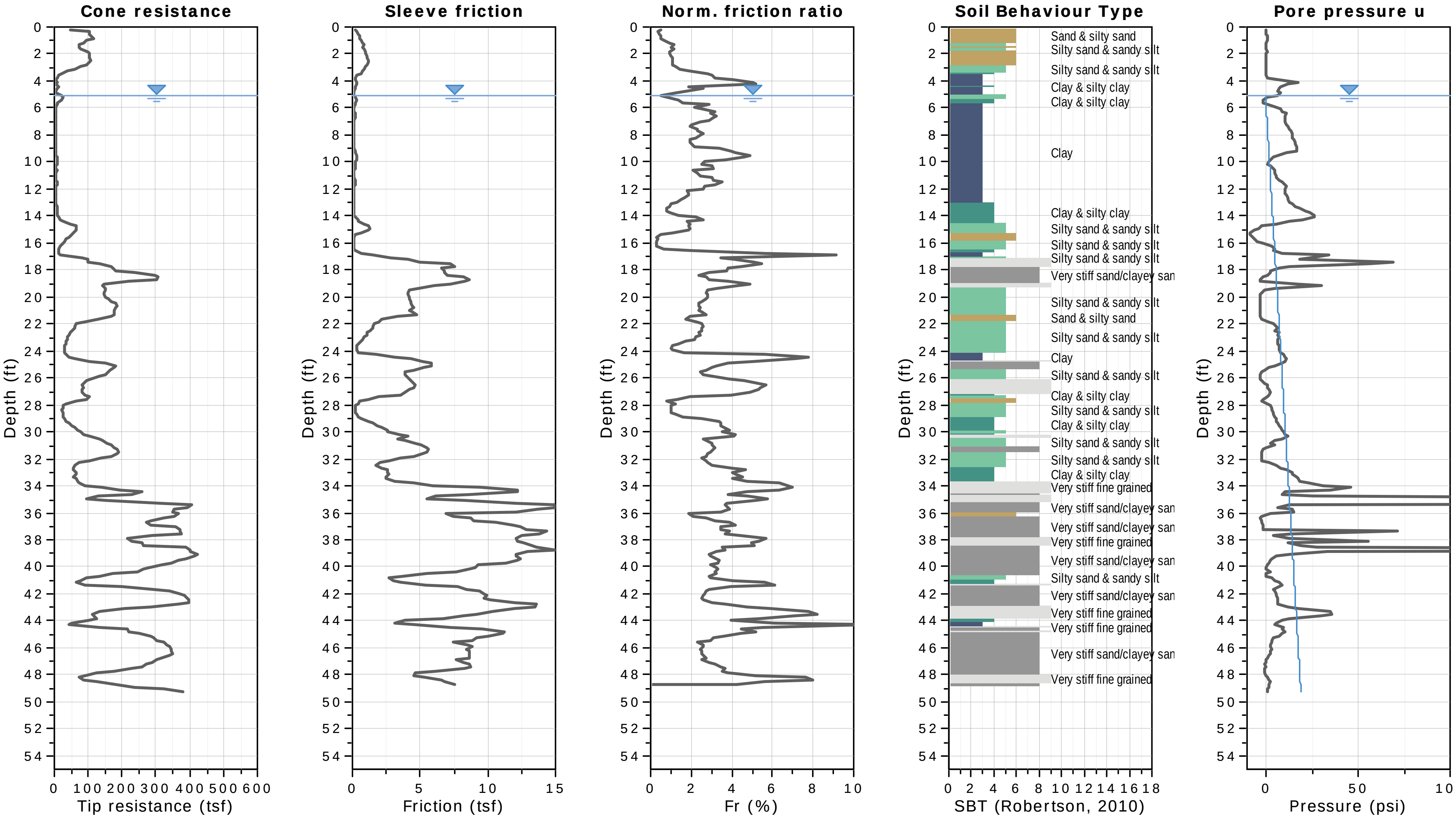
Total depth: 49.21 ft, Date: 10/23/2023

Cone Type: 15cm²

Cone Operator: AJ-ER

Project: Alameda Aquatic Center

Location: 1100 Atlantic Avenue, Alameda, CA 94501



Ninyo & Moore

https://ninyoandmoore.com/

CPT: CPT-05

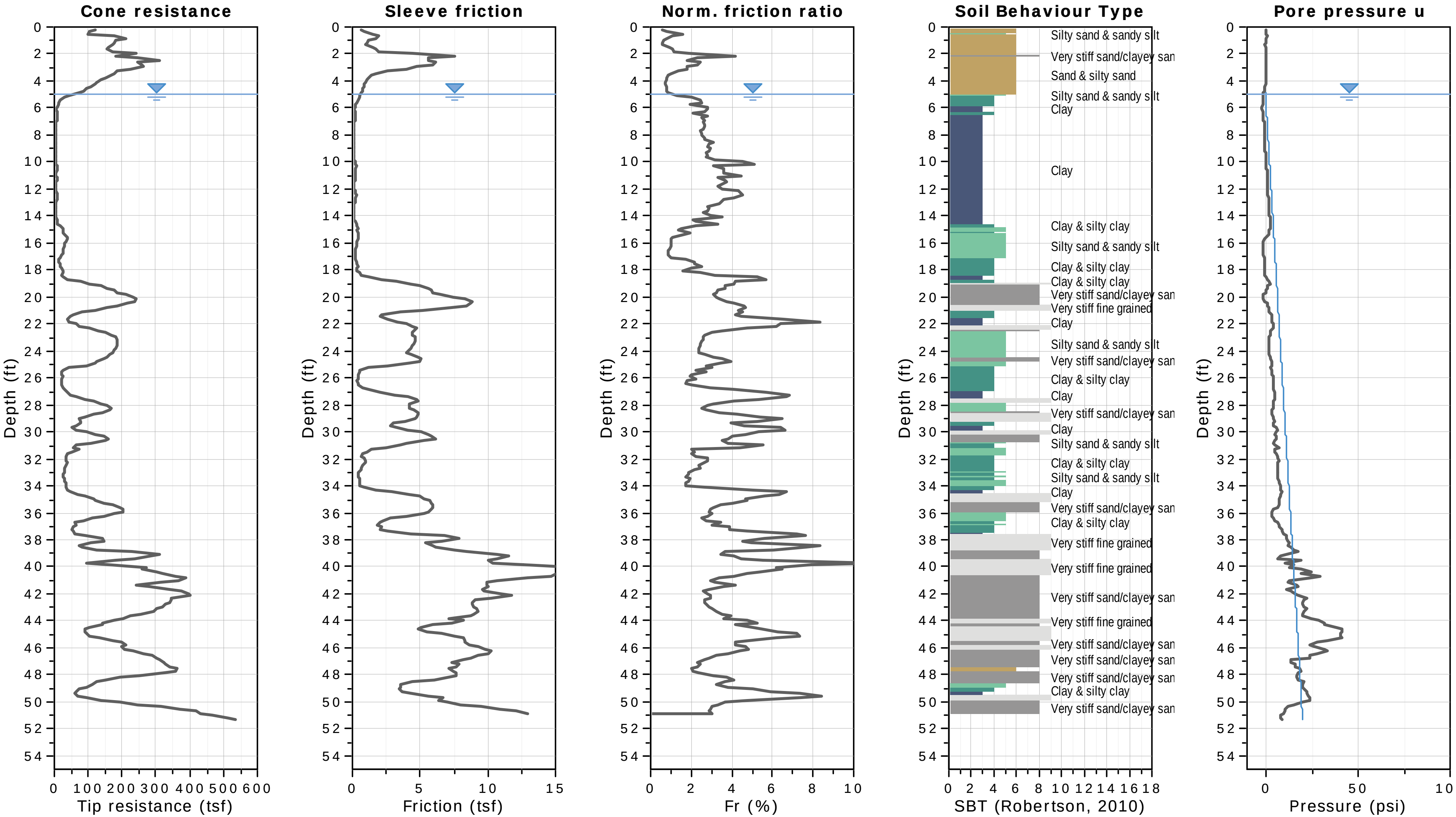
Total depth: 51.35 ft, Date: 10/23/2023

Cone Type: 15cm²

Cone Operator: AJ-ER

Project: Alameda Aquatic Center

Location: 1100 Atlantic Avenue, Alameda, CA 94501



Ninyo & Moore

https://ninyoandmoore.com/

CPT: CPT-06

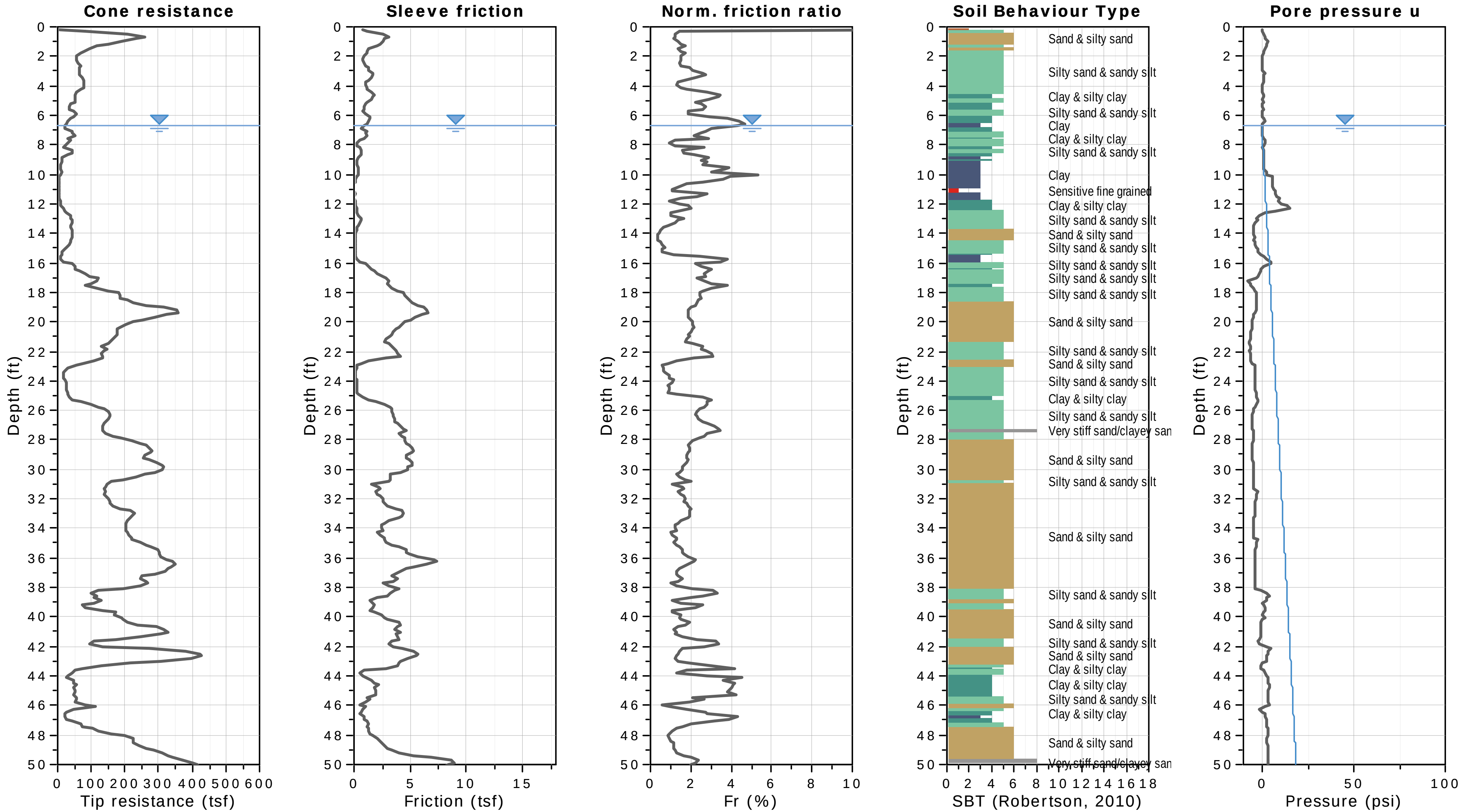
Total depth: 50.85 ft, Date: 10/23/2024

Cone Type: 15cm²

Cone Operator: JM-IY

Project: Alameda Aquatic Center

Location: 1100 Atlantic Avenue, Alameda, CA 94501



Ninyo & Moore

https://ninyoandmoore.com/

Project: Alameda Aquatic Center

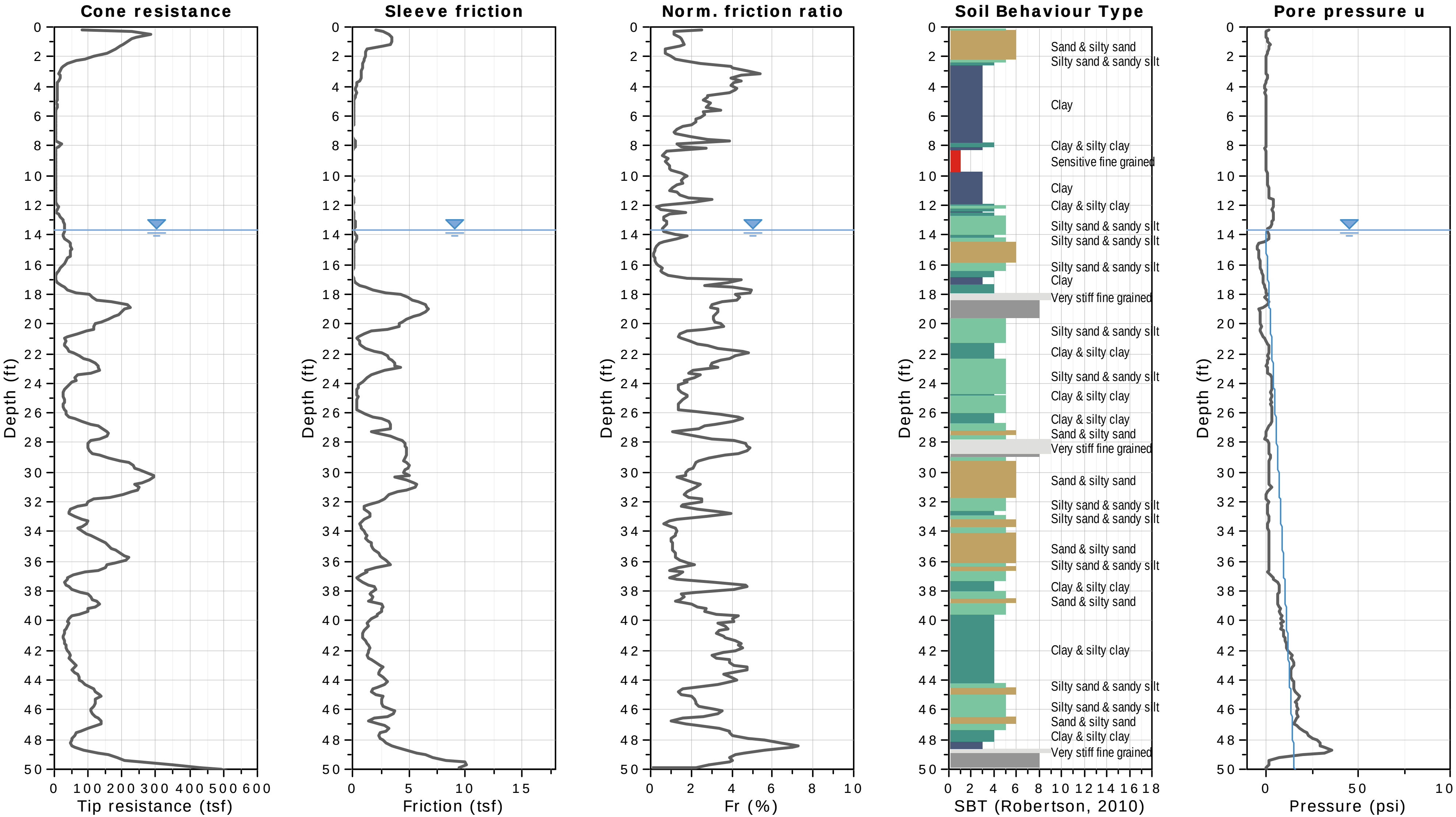
Location: 1100 Atlantic Avenue, Alameda, CA 94501

CPT: CPT-07

Total depth: 50.36 ft, Date: 10/23/2024

Cone Type: 15cm²

Cone Operator: JM-IY



Ninyo & Moore

https://ninyoandmoore.com/

CPT: CPT-08

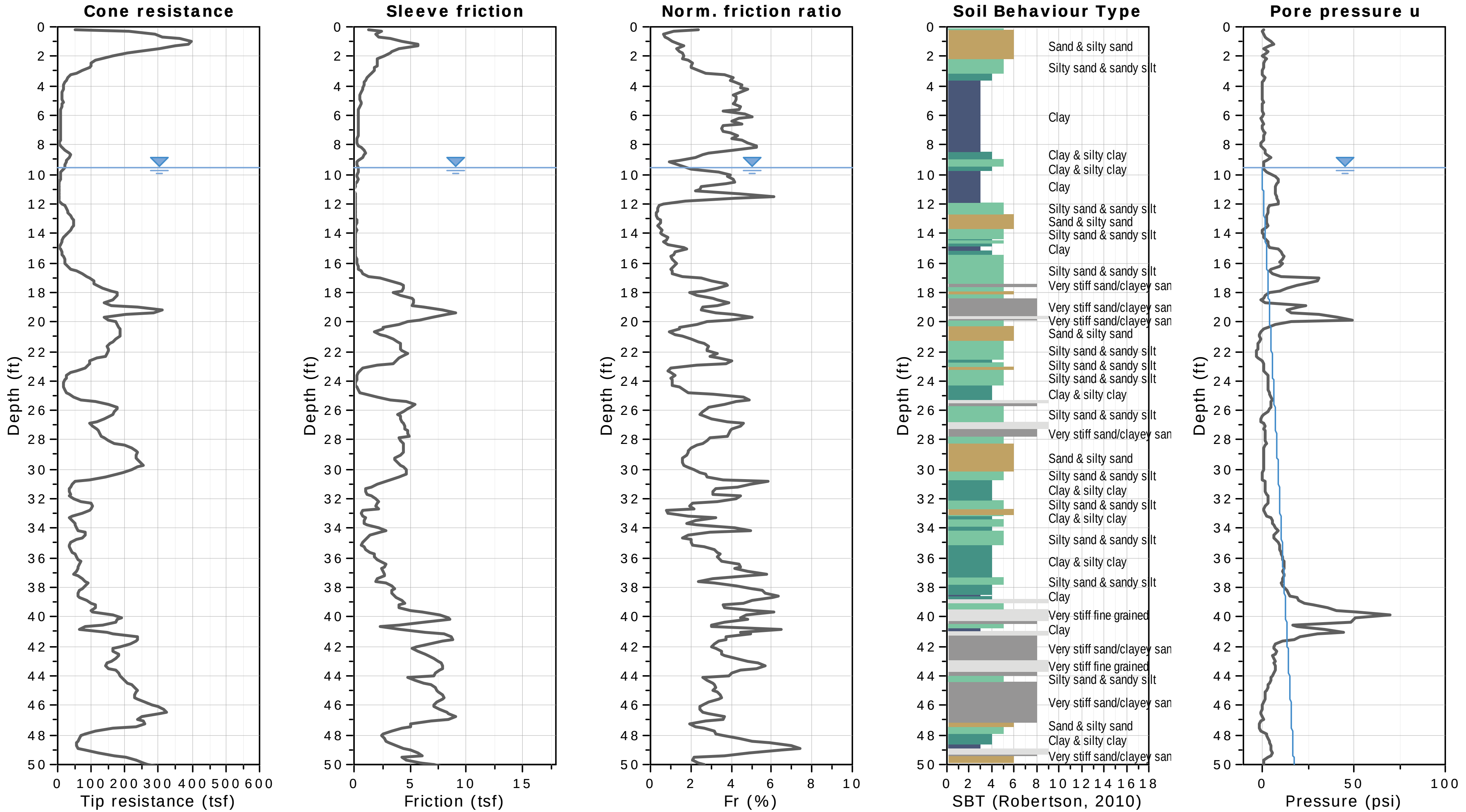
Total depth: 50.85 ft, Date: 10/23/2024

Cone Type: 15cm²

Cone Operator: JM-IY

Project: Alameda Aquatic Center

Location: 1100 Atlantic Avenue, Alameda, CA 94501



Ninyo & Moore

https://ninyoandmoore.com/

Project: Alameda Aquatic Center

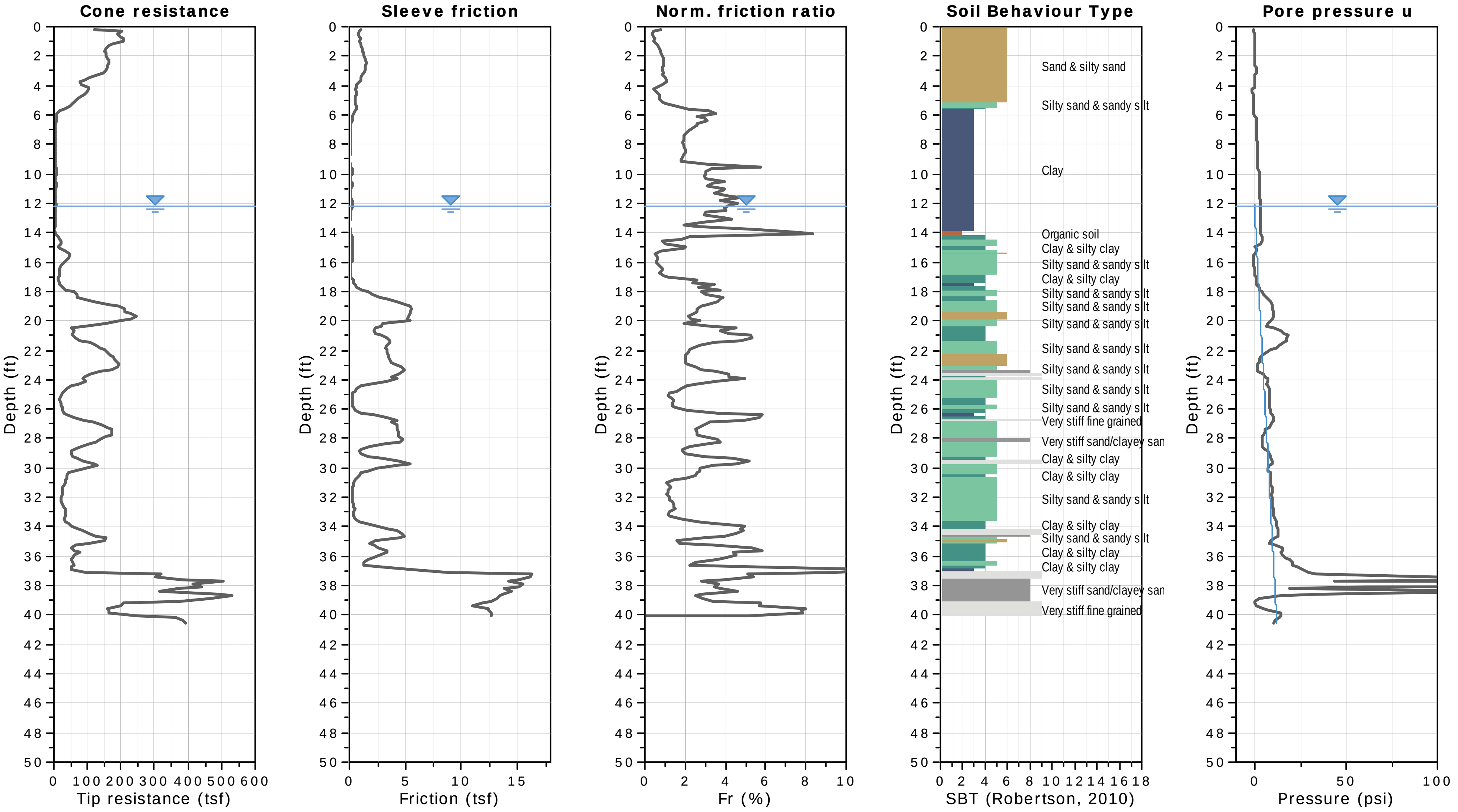
Location: 1100 Atlantic Avenue, Alameda, CA 94501

CPT: CPT-09

Total depth: 40.52 ft, Date: 10/23/2024

Cone Type: 15cm²

Cone Operator: JM-IY



Ninyo & Moore

https://ninyoandmoore.com/

CPT: CPT-10

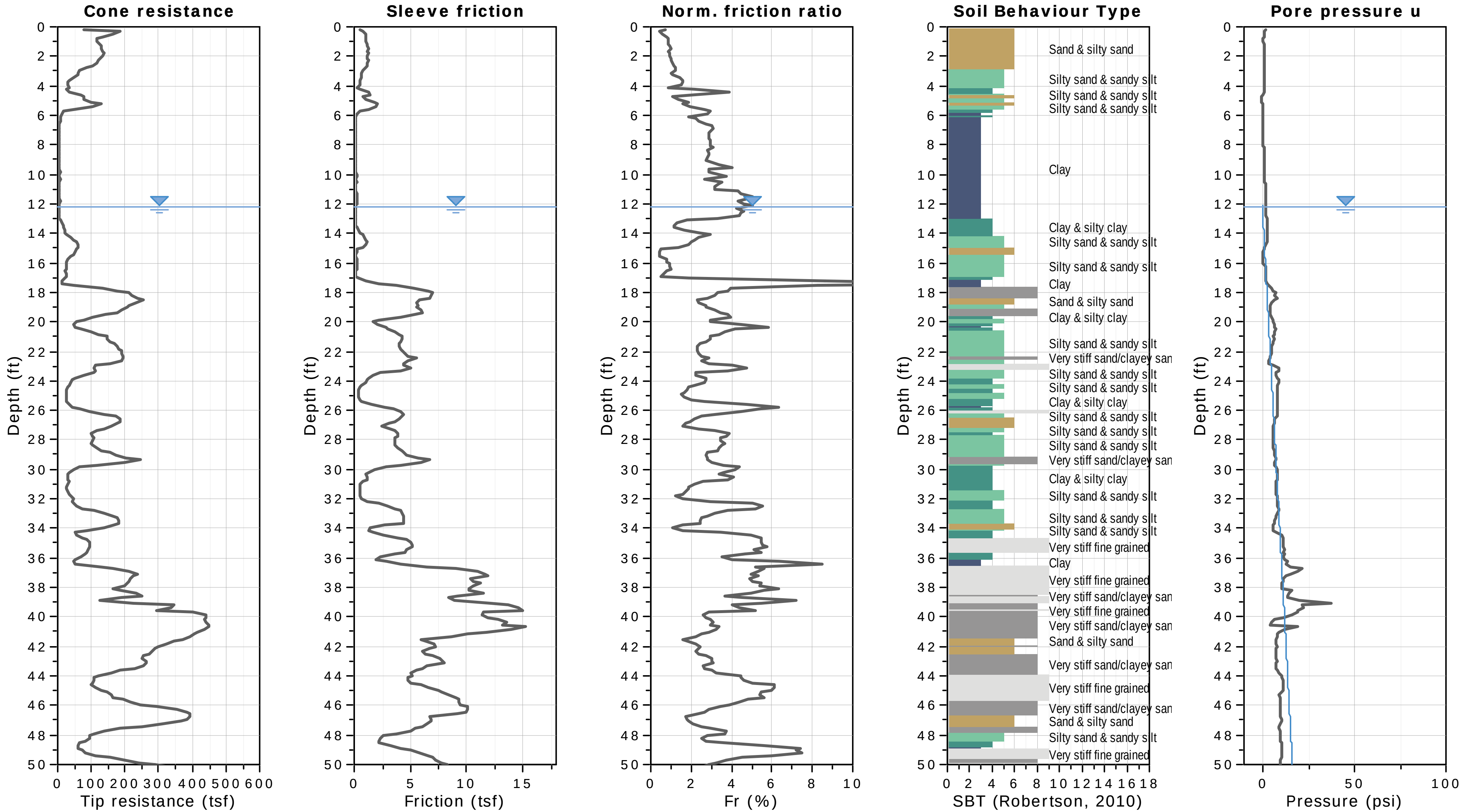
Total depth: 51.67 ft, Date: 10/23/2024

Cone Type: 15cm²

Cone Operator: JM-IY

Project: Alameda Aquatic Center

Location: 1100 Atlantic Avenue, Alameda, CA 94501



APPENDIX B

Boring Logs

APPENDIX B

BORING LOGS

Field Procedure for the Collection of Disturbed Samples
Disturbed soil samples were obtained in the field using the following method.

Bulk Samples
Bulk samples of representative earth materials were obtained from the exploratory boring. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler
Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 12 to 18 inches with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples
Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler
The sampler, with an external diameter of 3.0 inches, was lined with 6-inch long, thin brass liners with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass liners, sealed, and transported to the laboratory for testing.

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
							CL	Dashed line denotes material change. Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
15								
								The total depth line is a solid line that is drawn at the bottom of the boring.
20								

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BORING LOG

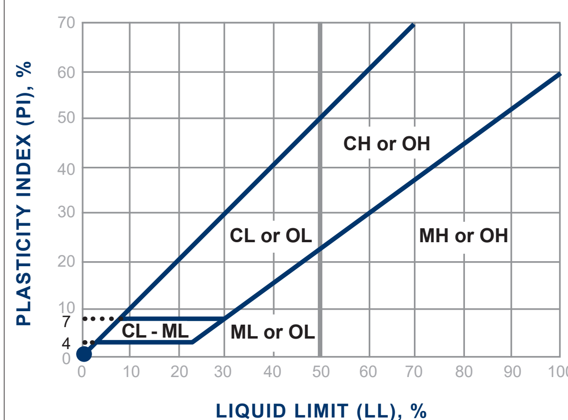
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions	
			Group Symbol	Group Name
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	GRAVEL more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	SILT and CLAY liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

Apparent Density	Spooling Cable or Cathead		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	Modified Split Barrel (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

Consistency - Fine-Grained Soil

Consis-tency	Spooling Cable or Cathead		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	Modified Split Barrel (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

DATE DRILLED	12/6/2023	BORING NO.	B-1
GROUND ELEVATION	8.6' NAVD88	SHEET 1 OF 2	
METHOD OF DRILLING	8" HSA, B-53R Truck Mounted (Exploration Geo.)		
DRIVE WEIGHT	140 lbs (wireline)	DROP	30 inches
SAMPLED BY	SSA	LOGGED BY SSA	REVIEWED BY RPM/MKW

DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SM	ARTIFICIAL FILL: Brown, moist, loose to medium dense, silty SAND.
		19	3.0				
		15					Moist to wet.
						CL	YOUNG BAY MUD: Gray, moist, firm, sandy lean CLAY.
10		6	30.3				
			▼				Wet.
						CL-ML	Blackish gray, wet, firm to stiff, silty CLAY.
		24	16.0	118.7			
						SM	DUNE SAND: Olive brown, wet, very dense, silty SAND.
20		82/12"					
		52					Brown.
30		40					Dense.
						SC	Olive brown, wet, medium dense, clayey SAND.
		48	18.7				
						SM	OLD BAY MUD: Bluish gray, wet, dense, silty SAND.
40							

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION

DATE DRILLED 12/6/2023 BORING NO. B-1

GROUND ELEVATION 8.6' NAVD88 SHEET 2 OF 2

METHOD OF DRILLING 8" HSA, B-53R Truck Mounted (Exploration Geo.)

DRIVE WEIGHT 140 lbs (wireline) DROP 30 inches

SAMPLED BY SSA LOGGED BY SSA REVIEWED BY RPM/MKW

Depth (feet)	Blows/Foot	U.S.C.S.	Description/Interpretation
40	42	SM	OLD BAY MUD (Continued): Bluish gray, wet, dense, silty SAND.
	31		
50	50/6"		Very dense.
			Total depth = 50.5 feet.
60			
70			
80			

Total depth = 50.5 feet.

Backfilled with neat cement shortly after drilling.

Notes:

Groundwater was measured at a depth of approximately 13.0 feet in the borehole shortly after completion of drilling.

Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents (ELS Architecture + Urban Design, 2025).

DATE DRILLED	12/5/2023	BORING NO.	B-2
GROUND ELEVATION	8.9' NAVD88	SHEET 1 OF 1	
METHOD OF DRILLING	8" HSA, B-53R Truck Mounted (Exploration Geo.)		
DRIVE WEIGHT	140 lbs (wireline)	DROP	30 inches
SAMPLED BY	SSA	LOGGED BY SSA	REVIEWED BY RPM/MKW

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	X					SC	ARTIFICIAL FILL: Gray and grayish brown, moist, medium dense, clayey SAND.
	X	X	34	8.2			
		X	27				Dark brown.
10		X	13	23.8	105.3	CL-ML	YOUNG BAY MUD: Black, moist, stiff, silty CLAY.
		X	14			SC	DUNE SAND: Grayish brown and olive brown, wet, loose to medium dense, clayey SAND.
20		X	92/11"			SM	Olive brown, wet, very dense, silty SAND.
30							
40							

Total depth = 21.4 feet.

Backfilled with neat cement shortly after drilling.

Notes:
Groundwater was measured at a depth of approximately 14.0 feet in the borehole shortly after completion of drilling.

Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents (ELS Architecture + Urban Design, 2025).

DATE DRILLED 12/6/2023 BORING NO. B-3

GROUND ELEVATION 9.4' NAVD88 SHEET 1 OF 1

METHOD OF DRILLING 8" HSA, B-53R Truck Mounted (Exploration Geo.)

DRIVE WEIGHT 140 lbs (wireline) DROP 30 inches

SAMPLED BY SSA LOGGED BY SSA REVIEWED BY RPM/MKW

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SM	ARTIFICIAL FILL: Light brown and bluish gray, moist, medium dense, silty SAND.
		26	17.7				
		40	67.5	58.2			
						CH	YOUNG BAY MUD: Bluish gray, moist, hard, fat CLAY (possible fill in the upper few feet).
10		10	78.4	53.3			Soft.
			▼				Wet.
						SM	Bluish gray, wet, medium dense, silty SAND.
		20					
						SC	DUNE SAND: Bluish gray and brown, wet, dense, clayey SAND.
20		42					
		35					
30							Total depth = 26.5 feet. Backfilled with neat cement shortly after drilling. Notes: Groundwater was measured at a depth of approximately 13.0 feet in the borehole shortly after completion of drilling. Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents (ELS Architecture + Urban Design, 2025).
40							

DATE DRILLED	12/5/2023	BORING NO.	B-4
GROUND ELEVATION	14.0' NAVD88	SHEET 1 OF 1	
METHOD OF DRILLING	8" HSA, B-53R Truck Mounted (Exploration Geo.)		
DRIVE WEIGHT	140 lbs (wireline)	DROP	30 inches
SAMPLED BY	SSA	LOGGED BY SSA	REVIEWED BY RPM/MKW

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	Bulk					SC	ARTIFICIAL FILL: Grayish brown, moist, medium dense to dense, clayey SAND; trace gravel.
	Driven	39	9.9	118.9			
						CL	Grayish brown, moist, very stiff, sandy lean CLAY.
	Driven	29	44.8	74.5			
							Total depth = 6.5 feet. Backfilled with neat cement shortly after drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents (ELS Architecture + Urban Design, 2025).
10							
20							
30							
40							

APPENDIX C

Laboratory Testing

APPENDIX C

LABORATORY TESTING

Classification
Soil was classified using visual-manual procedures (ASTM D 2488). Soil classifications were updated in accordance with the Unified Soil Classification System (USCS) and ASTM D 2487 based on the results of laboratory tests to evaluate particle size characteristics and Atterberg Limits. Soil classifications are indicated on the log of the exploratory boring in Appendix B.

In-Place Moisture and Density Tests
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix B.

200 Wash Analysis
An evaluation of the percentage of minus-200 sieve material in a selected soil sample was performed in accordance with ASTM D 1140. The results of the test are presented on Figure C-1.

Atterberg Limits
Tests were performed on selected soil samples to evaluate the liquid limit, plastic limit, and plasticity index in accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the USCS. The test results and classifications are shown on Figure C-2.

Consolidation Test
Consolidation test was performed on a selected relatively undisturbed soil sample in accordance with ASTM D 2435. The sample was inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The consolidation test results are summarized graphically on Figure C-3.

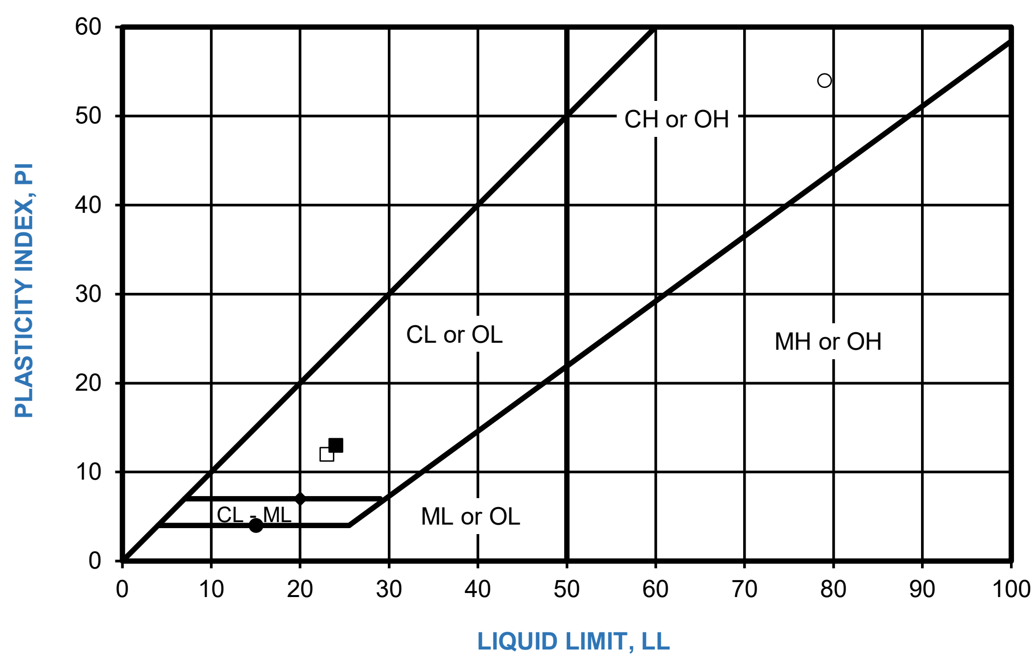
Unconsolidated-Undrained (UU) Triaxial Tests
Unconsolidated-Undrained Triaxial tests were performed on undisturbed samples in accordance with ASTM D 2850 to evaluate the undrained shear strength of selected materials. The results are shown on Figure C-4.

R-Value Test
The resistance value, or R-value, for site soils were evaluated in general accordance with CT 301. A sample was prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are shown on Figure C-5

SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	6.0 - 6.5	Silty SAND	100	16	SM
B-1	45.0 - 46.5	Silty SAND	100	33	SM
B-2	3.0 - 3.5	Clayey SAND	100	32	SC
B-2	16.0 - 16.5	Clayey SAND	100	20	SC
B-3	15.5 - 16.0	Silty SAND	100	24	SM
B-3	25.0 - 26.5	Clayey SAND	100	44	SC
B-4	0.0 - 5.0	Clayey SAND; trace gravel	91	39	SC

PERFORMED IN ACCORDANCE WITH 1140

SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-1	15.5 - 16.0	15	11	4	CL-ML	CL-ML
■	B-2	0.0 - 5.0	24	11	13	CL	SC
◆	B-2	10.5 - 11.0	20	13	7	CL-ML	CL-ML
○	B-3	6.0 - 6.5	79	25	54	CH	CH
□	B-4	3.0 - 3.5	23	11	12	CL	CL



PERFORMED IN ACCORDANCE WITH ASTM D 4318

FIGURE C-2

ATTERBERG LIMITS TEST RESULTS

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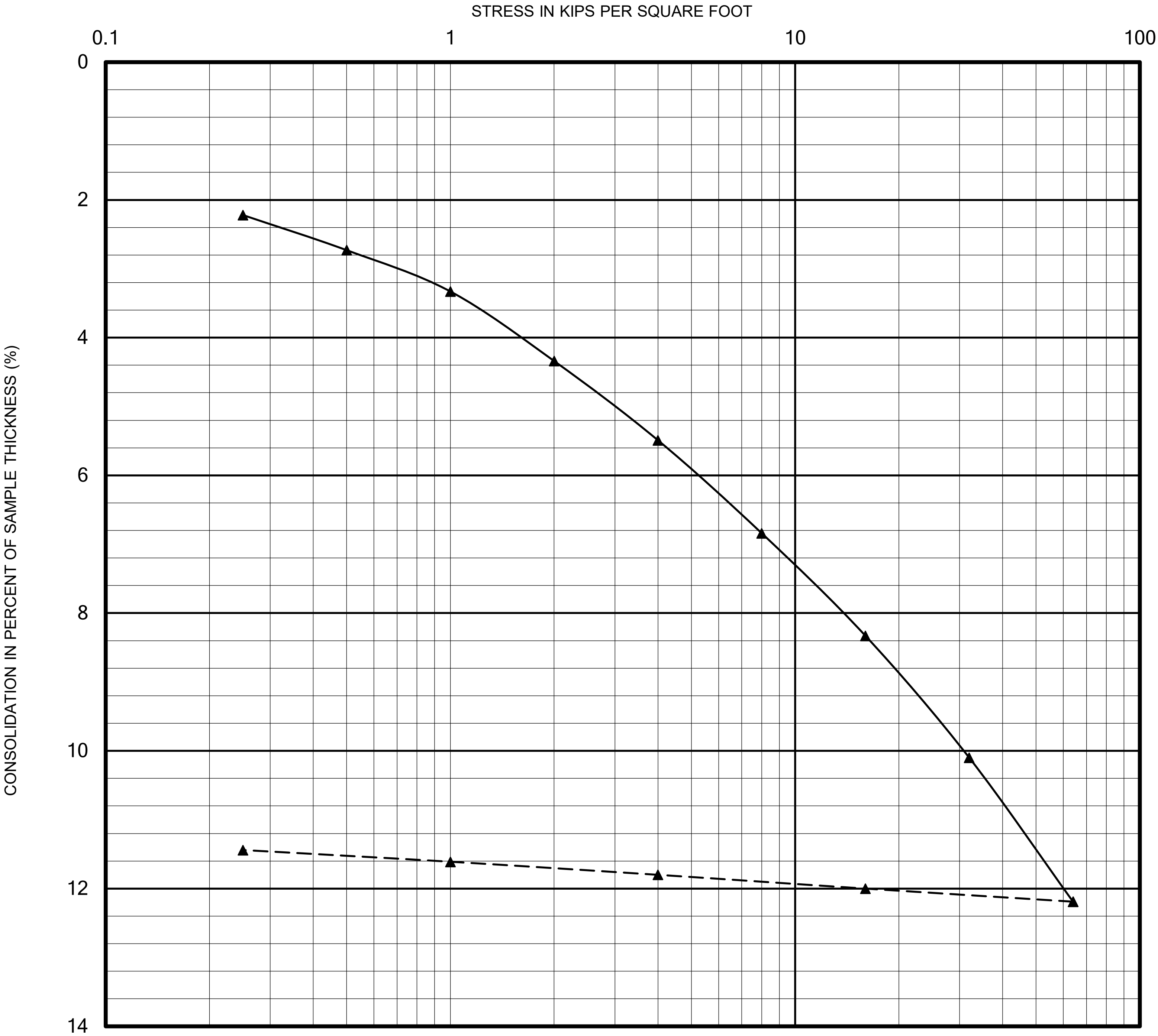
1100 ATLANTIC AVENUE, ALAMEDA, CALIFORNIA

403773009 | 05/25



Loading After Inundation
Rebound Cycle

Sample Location	B-2
Depth (ft)	11.0 - 11.5
Soil Type	CL-ML

PERFORMED IN ACCORDANCE WITH ASTM D 2435

FIGURE C-3

CONSOLIDATION TEST RESULTS

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Geotechnical & Environmental Sciences Consultants

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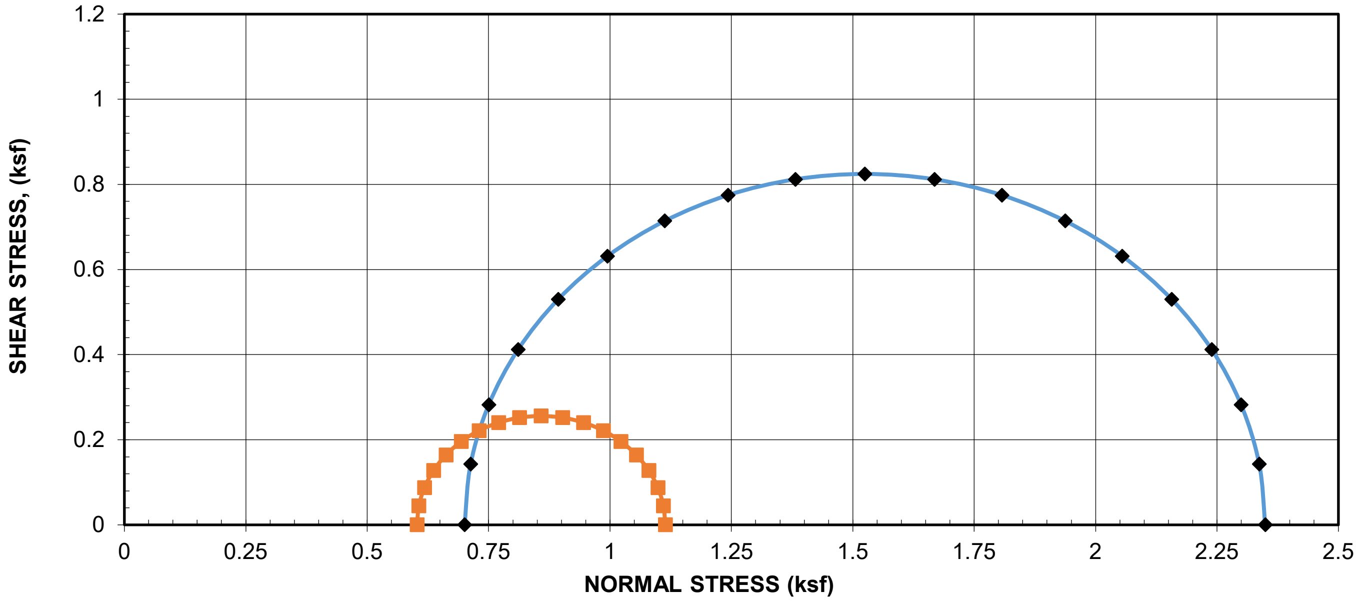
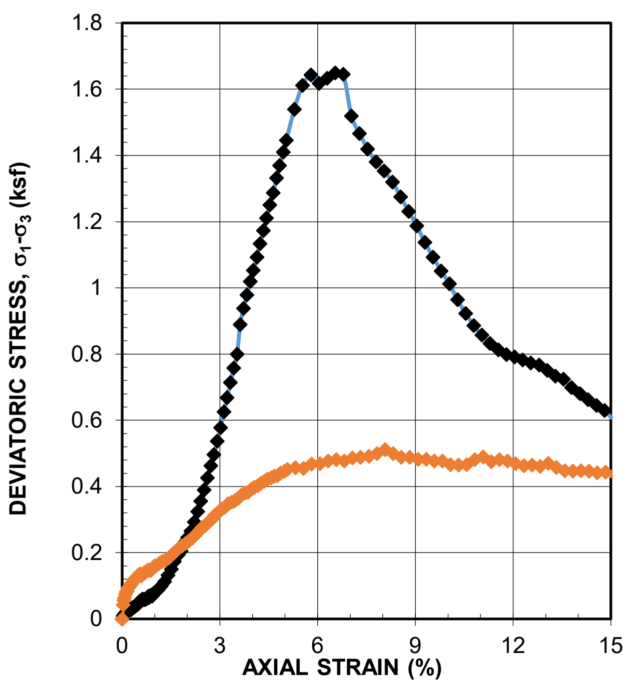
1100 ATLANTIC AVENUE, ALAMEDA, CALIFORNIA

403773009 | 05/25





SYMBOL	◆ (black)	◆ (orange)		
SAMPLE LENGTH, (in)	5.28	5.90		
SAMPLE DIAMETER, (in)	2.39	2.40		
SPECIFIC GRAVITY, ( )	2.65	2.65		
INITIAL MOISTURE, (%)	16.0	78.4		
INITIAL DRY DENSITY, (pcf)	118.7	53.3		
INITIAL VOID RATIO, ( )	0.3931	2.1024		
INITIAL SATURATION, (%)	107.9	98.8		
CELL PRESSURE, (ksf)	0.7	0.6		
BACK PRESSURE, (ksf)	0.0	0.0		
STRAIN RATE, (%/minute)	1.0	1.0		
AT FAILURE ELAPSED TIME, tf (min)	6.5	8.1		
AT FAILURE AXIAL STRAIN, εf (%)	6.5	8.1		
AT FAILURE DEVIATOR STRESS (ksf)	1.65	0.51		
AT FAILURE MAJOR STRESS, σ1f (ksf)	2.35	1.11		
AT FAILURE MINOR STRESS, σ3f (ksf)	0.70	0.60		
MEMBRANE CORRECTION USED	FALSE	TRUE		

	DESCRIPTION (USCS SOIL TYPE)	SAMPLE LOCATION	SAMPLE DEPTH (feet)	COMPRESSIVE STRENGTH (ksf)	UU SHEAR STRENGTH s_u, (ksf)	REMARKS
◆ (black)	Silty CLAY (CL-ML)	B-1	16.0-16.5	1.65	0.82	
◆ (orange)	Fat CLAY (CH)	B-3	10.5-11.0	0.51	0.26	

PERFORMED IN ACCORDANCE WITH ASTM D 2850 ON INTACT SPECIMENS

MOISTURE CONTENT & DENSITY EVALUATED BY ASTM D 2216 & ASTM D 7263, SPECIFIC GRAVITY ASSUMED

SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R-VALUE
B-4	0.0-5.0	SC	12.0

PERFORMED IN ACCORDANCE WITH ASTM D 2844/CT 301

APPENDIX D

Corrosivity Testing (CERCO Analytical)

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

2 January, 2024

Job No. 2312042
Cust. No.13270

Mr. Rathna Mothkuri
Ninyo & Moore
2149 O'Toole Avenue, Suite 30
San Jose, CA 95131

Subject: Project No.: 403773004
Project Name: City of Alameda New Aquatic Center, 1100 Atlantic Ave., Alameda, CA
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Mothkuri:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on December 22, 2023. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurement, this sample is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration is 28 mg/kg and is determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration is 78 mg/kg and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at this location.

The pH of the soil is 7.81, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 260-mV and is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Ninyo & Moore
Client's Project No.: 403773004
Client's Project Name: City of Alameda New Aquatic Center, 1100 Atlantic Avenue, Alameda, CA
Date Sampled: 5-Dec-23
Date Received: 22-Dec-23
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 2-Jan-2024

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2312042-001	B-2/0.0-5.0'	260	7.81	-	2,200	-	28	78

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	22-Dec-2023	28-Dec-2023	-	22-Dec-2023	-	28-Dec-2023	28-Dec-2023

* Results Reported on "As Received" Basis

N.D. - None Detected

Julia Clauson
Chemist

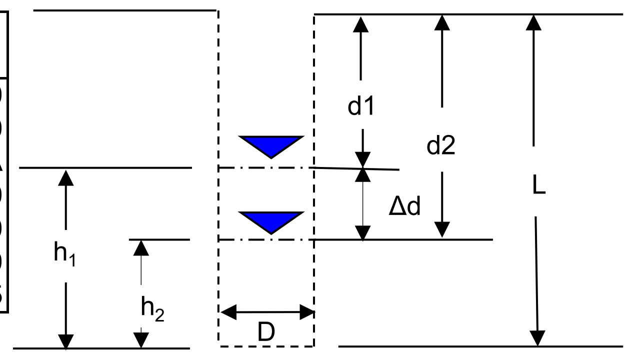
Quality Control Summary - All laboratory quality control parameters were found to be within established limits

APPENDIX E

Percolation Tests Results

Project =	City of Alameda New Aquatic Center	
Project No. =	403773009	
Depth of Boring, L (ft) =		3.0
Diameter of Boring, D (in) =		8.0
Diameter of Pipe (in) =		N/A
Initial Depth to Water, d1 (in), (Final Period) =		30.0
Initial Height of Water, h1 (in), (Final Period) =		6.0
Water Level Drop, Δd (in), (Final Period) =		0.0
Reduction factor, Rf =		2.5

h1 = L - d1 (in inches)
Rf = ((2h1 - Δd)/DIA) +1



Test No. (Hole No.)	Time (hr:min)	Elapsed Time (min)	Depth to Water, d (in)	Water Level, h (in)	Change in Water Level, Δd (in)	Time Interval (hour)	Percolation Rate (inch/hour)	Adjusted Percolation Rate (inch/hour)
IT-1	9:00 9:15	15	30.00	6.00	0.0	0.25	0.0	0.0
	9:15 9:30	15	30.00	6.00	0.0	0.25	0.0	0.0
	9:30 9:45	15	30.00	6.00	0.0	0.25	0.0	0.0
	9:45 10:00	15	30.00	6.00	0.0	0.25	0.0	0.0
	10:00 10:15	15	30.00	6.00	0.0	0.25	0.0	0.0
	10:15 10:30	15	30.00	6.00	0.0	0.25	0.0	0.0
	10:30 10:45	15	30.00	6.00	0.0	0.25	0.0	0.0
	10:45 11:00	15	30.00	6.00	0.0	0.25	0.0	0.0

FIGURE E-1

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

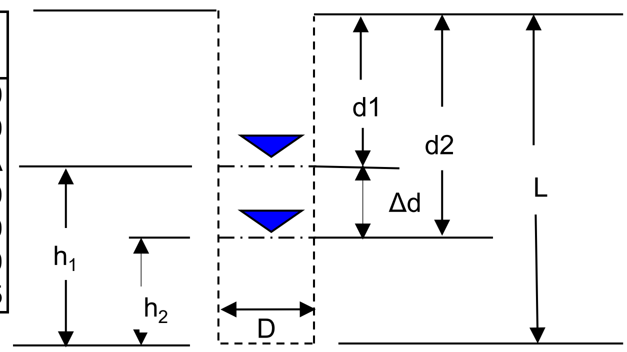
PERCOLATION TEST RESULTS

CITY OF ALAMEDA NEW AQUATIC CENTER
1100 ATLANTIC AVENUE, ALAMEDA, CALIFORNIA

Project =	City of Alameda New Aquatic Center
Project No. =	403773009
Depth of Boring, L (ft) =	3.0
Diameter of Boring, D (in) =	8.0
Diameter of Pipe (in) =	N/A
Initial Depth to Water, d1 (in), (Final Period) =	30.0
Initial Height of Water, h1 (in), (Final Period) =	6.0
Water Level Drop, Δd (in), (Final Period) =	0.0
Reduction factor, Rf =	2.5

h1 = L - d1 (in inches)

Rf = ((2h1 - Δd)/DIA) +1



Test No. (Hole No.)	Time (hr:min)	Elapsed Time (min)	Depth to Water, d (in)	Water Level, h (in)	Change in Water Level, Δd (in)	Time Interval (hour)	Percolation Rate (inch/hour)	Adjusted Percolation Rate (inch/hour)
IT-2	9:10 9:25	15	30.00	6.00	0.0	0.25	0.0	0.0
	9:25 9:40	15	30.00	6.00	0.0	0.25	0.0	0.0
	9:40 9:55	15	30.00	6.00	0.0	0.25	0.0	0.0
	9:55 10:10	15	30.00	6.00	0.0	0.25	0.0	0.0
	10:10 10:25	15	30.00	6.00	0.0	0.25	0.0	0.0
	10:25 10:40	15	30.00	6.00	0.0	0.25	0.0	0.0
	10:40 10:55	15	30.00	6.00	0.0	0.25	0.0	0.0
	10:55 11:10	15	30.00	6.00	0.0	0.25	0.0	0.0

REMARKS: At 9:55 AM, all the water was absorbed by the soil - active sign of caving/soil collapse at the bottom making the depth of the percolation hole to 2.5 feet. Added 6 inches of water to the hole and continued the test.

FIGURE E-2

PERCOLATION TEST RESULTS

CITY OF ALAMEDA NEW AQUATIC CENTER
1100 ATLANTIC AVENUE, ALAMEDA, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

APPENDIX F

Soil Disposal Certificate

Manifest

SOIL SAFE OF CA – TPST

Non-Hazardous Soils

Manifest #

Date of Shipment:	Responsible for Payment:	Transport Truck #:	Facility #:	Approval Number:	Load #
12/29/23		Lariat	A07	A5-8205	661

Generator / Consultant

Generator's Name and Billing Address:
CITY OF ALAMEDA
950 WEST MALL SQUARE
ALAMEDA, CA 94501

Generator's Phone #: 510-747-7948
Person to Contact:
FAX#:
Customer Account Number

Consultant's Name and Billing Address:

Consultant's Phone #:
Person to Contact:
FAX#:
Customer Account Number

Generation Site (Transport from): *(name & address)*
JEAN SWEENEY OPEN SPACE PARK
8TH STREET AND STEWART COURT
ALAMEDA, CA 94501

Site Phone #:
Person to Contact:
FAX#:

Designated Facility (Transport to): *(name & address)*
SOIL SAFE
12328 HIBISCUS AVENUE
ADELANTO, CA 92301

Facility Phone #: (800) 862-8001
Person to Contact: JOE PROVANSAL
FAX#: (760) 246-8004

Transporter Name and Mailing Address:
BELSHIRE
25971 TOWNE CENTRE DRIVE
FOOTHILL RANCH, CA 92610
3x55
BESI: 361569

Transporter's Phone #: 949-460-5200
CAR000183913
Person to Contact: LARRY MOOTHART
1629169
Customer Account Number
FAX#: 949-460-5210

Description of Soil	Moisture Content	Contaminated by:	Approx. Qty:	Description of Delivery	Gross Weight	Tare Weight	Net Weight
Sand ☐ Organic ☐ Clay ☐ Other ☐	0 - 10% ☐ 10 - 20% ☐ 20% - over ☐	Gas ☐ Diesel ☐ Other ☐	3 DM	Soil	39840	38100	1740
Sand ☐ Organic ☐ Clay ☐ Other ☐	0 - 10% ☐ 10 - 20% ☐ 20% - over ☐	Gas ☐ Diesel ☐ Other ☐					.87

List any exception to items listed above: Scale Ticket # 178976

Generator's and/or consultant's certification: I/We certify that the soil referenced herein is taken entirely from those soils descried in the Soil Data Sheet completed and certified by me/us for the Generation Site shown above and nothing has been added or done to such soil that would alter it in any way.

Print or Type Name: Generator ☐ Consultant ☐
Larry Moothart of BESI on behalf of generator
Signature and date: Month 12 Day 29 Year 23

Transporter

Transporter's certification: I/We acknowledge receipt of the soil referenced above and certify that such soil is being delivered in exactly the same condition as when received. I/We further certify that the soil is being directly transported from the Generation Site to the Designated Facility without off-loading, adding to, subtracting from or in any way delaying delivery to such site.

Print or Type Name: Paul Freeman
Signature and date: Month 12 Day 29 Year 23

Recycling Facility

Discrepancies:

Recycling Facility certifies the receipt of the soil covered by this manifest except as noted above:

Print or Type Name: Joe Provansal / Barry Meek / Bill Bishop
Signature and date: 1·26·04

Please print or type.

8THSTREE/3427030

TRANSPORTER COPY

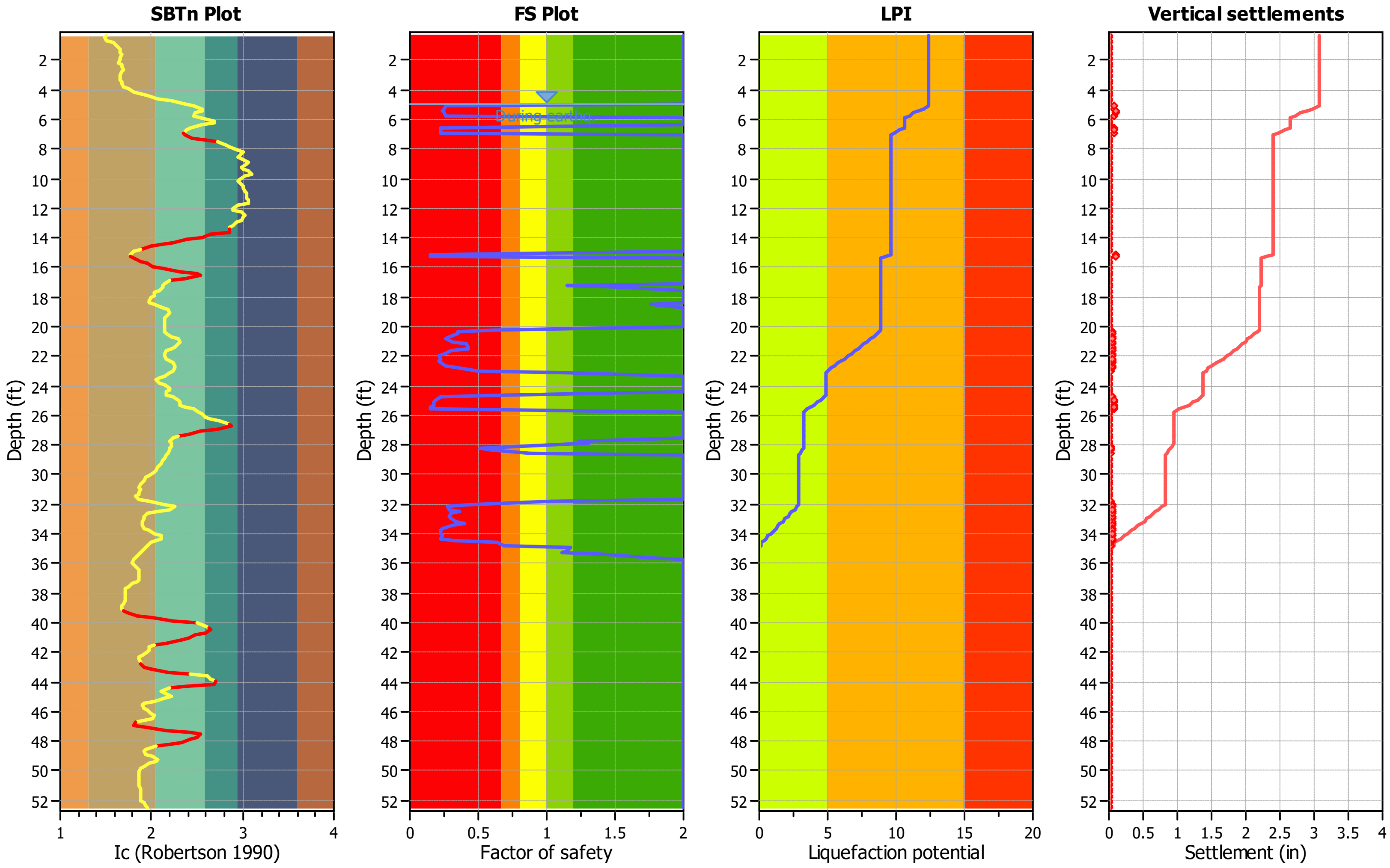
APPENDIX G

Calculation

Ninyo & Moore
2149 O'Toole Avenue, Suite 30
San Jose, CA 95131
https://ninyoandmoore.com

Project: 403773009 - City of Alameda New Aquatic Center
Location: 1100 Atlantic Avenue, Alameda, California 94501

CPT: CPT-01
Total depth: 52.50 ft

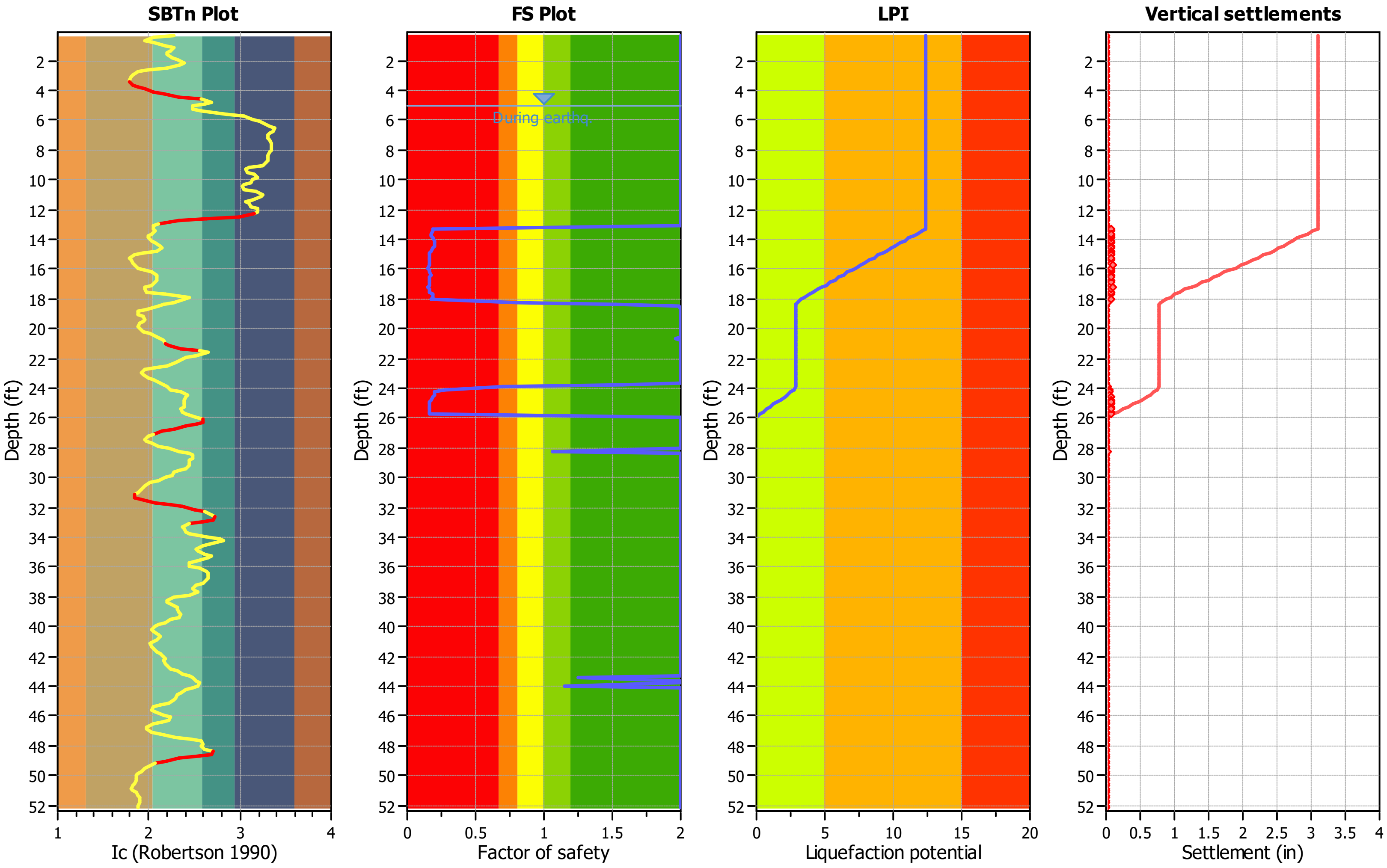


Analysis method:	B&I (2014)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.51	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.72	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Ninyo & Moore
2149 O'Toole Avenue, Suite 30
San Jose, CA 95131
https://ninyoandmoore.com

Project: 403773009 - City of Alameda New Aquatic Center
Location: 1100 Atlantic Avenue, Alameda, California 94501

CPT: CPT-02
Total depth: 52.17 ft

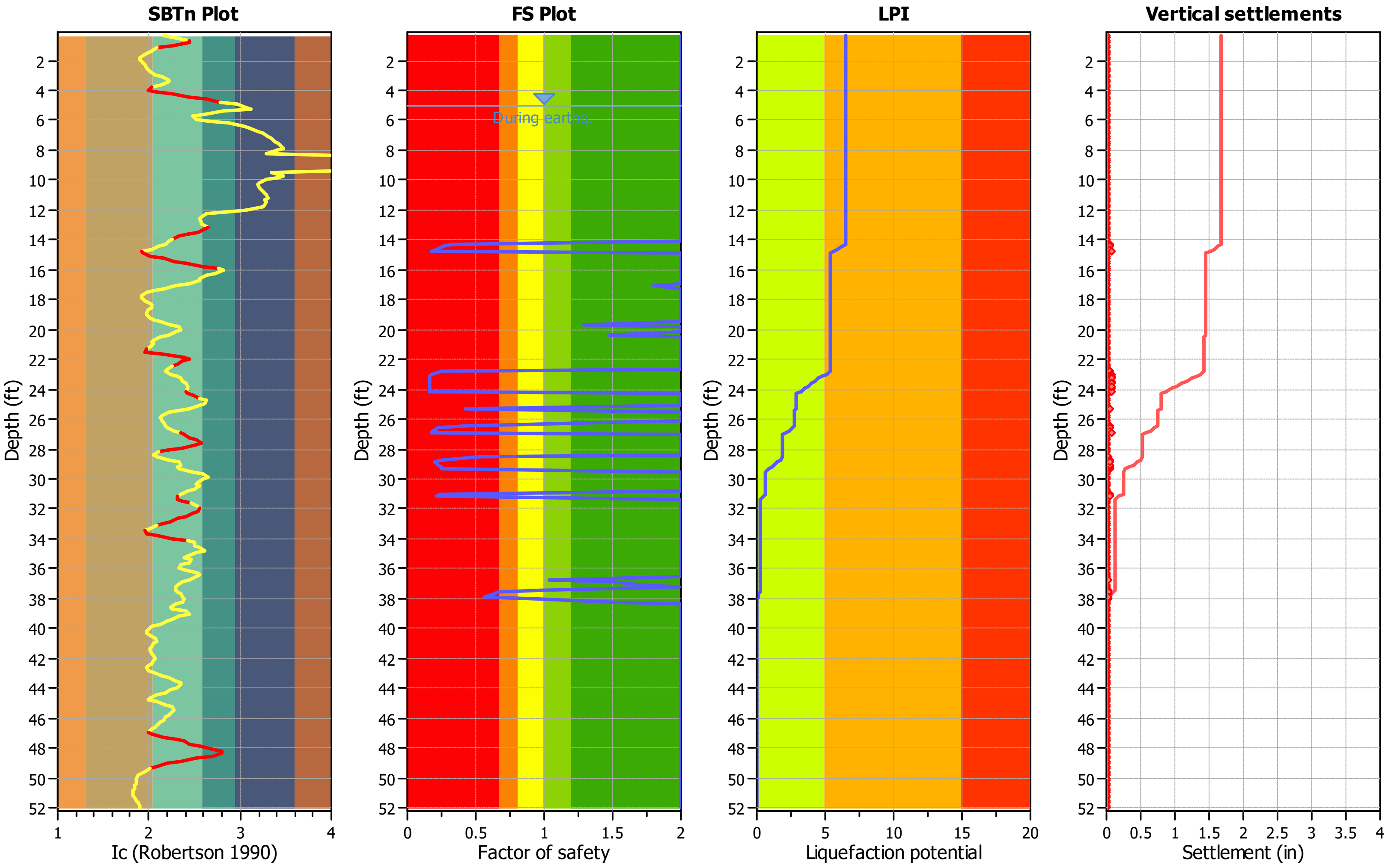


Analysis method:	B&I (2014)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	Yes
Earthquake magnitude M_w:	7.51	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	50.00 ft
Peak ground acceleration:	0.72	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

Ninyo & Moore
2149 O'Toole Avenue, Suite 30
San Jose, CA 95131
https://ninyoandmoore.com

Project: 403773009 - City of Alameda New Aquatic Center
Location: 1100 Atlantic Avenue, Alameda, California 94501

CPT: CPT-03
Total depth: 52.00 ft

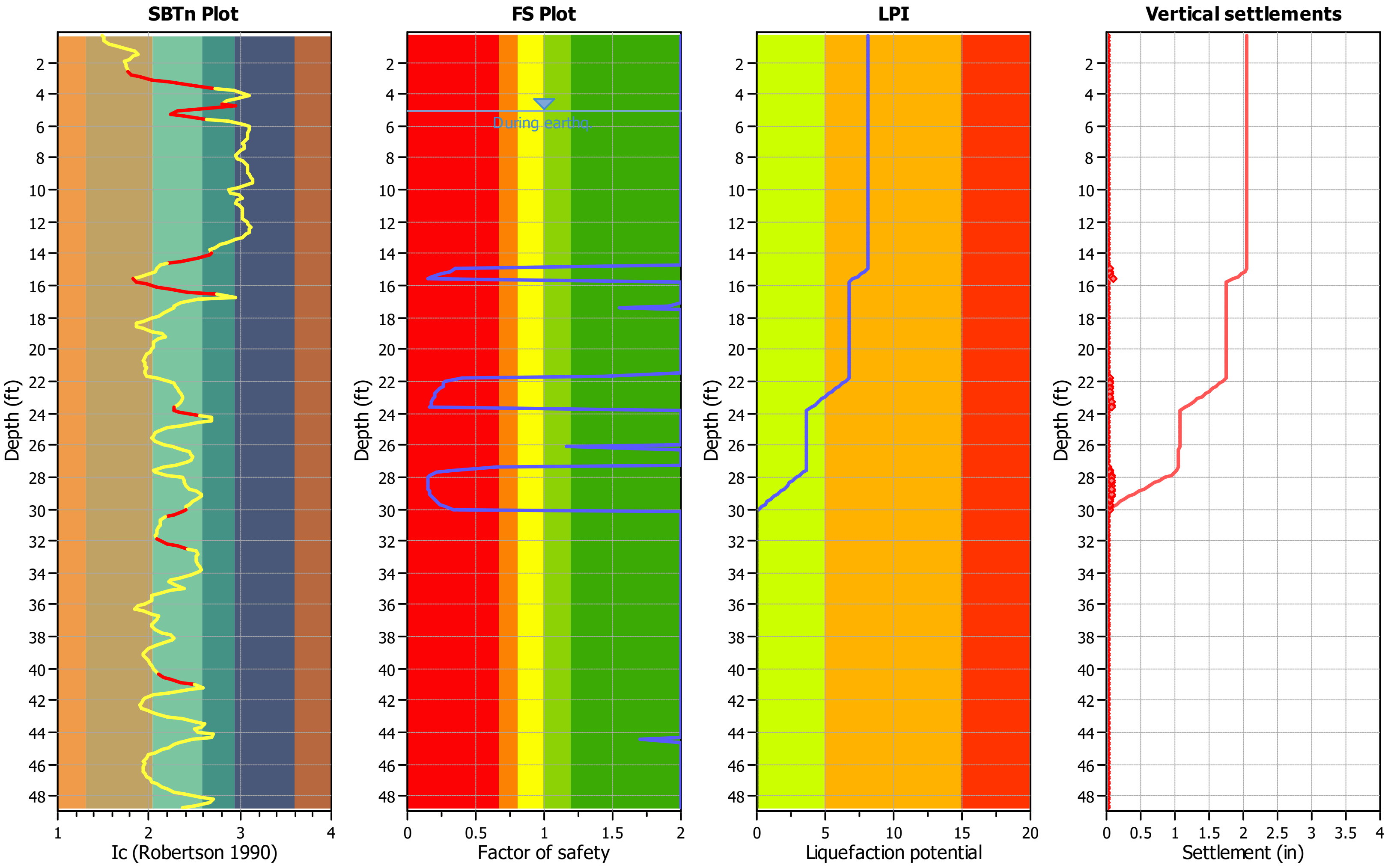


Analysis method:	B&I (2014)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	Yes
Earthquake magnitude M_w:	7.51	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	50.00 ft
Peak ground acceleration:	0.72	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

Ninyo & Moore
2149 O'Toole Avenue, Suite 30
San Jose, CA 95131
https://ninyoandmoore.com

Project: 403773009 - City of Alameda New Aquatic Center
Location: 1100 Atlantic Avenue, Alameda, California 94501

CPT: CPT-04
Total depth: 48.72 ft

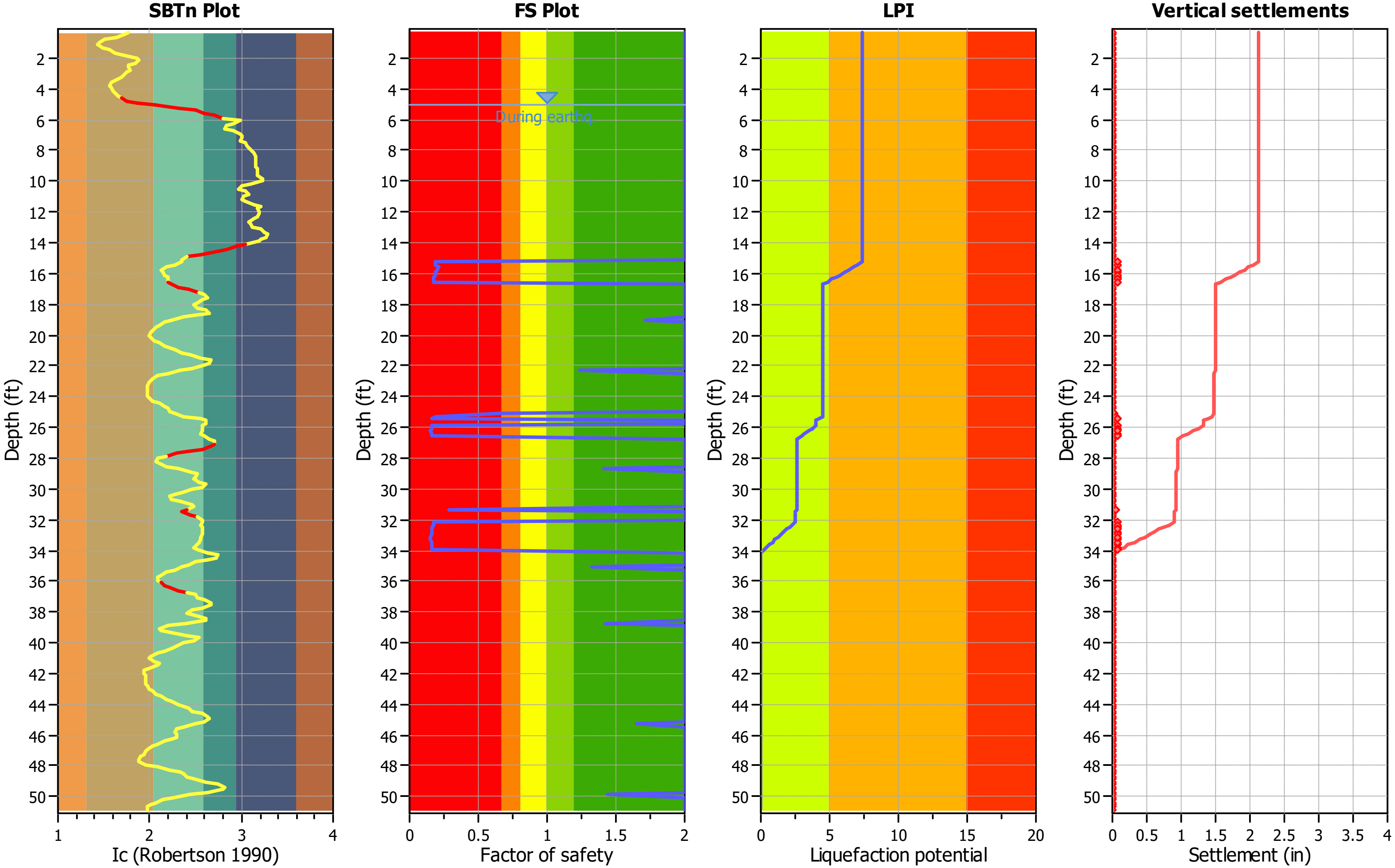


Analysis method:	B&I (2014)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.51	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.72	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Ninyo & Moore
2149 O'Toole Avenue, Suite 30
San Jose, CA 95131
https://ninyoandmoore.com

Project: **403773009 - City of Alameda New Aquatic Center**
Location: **1100 Atlantic Avenue, Alameda, California 94501**

CPT: CPT-05
Total depth: 50.86 ft

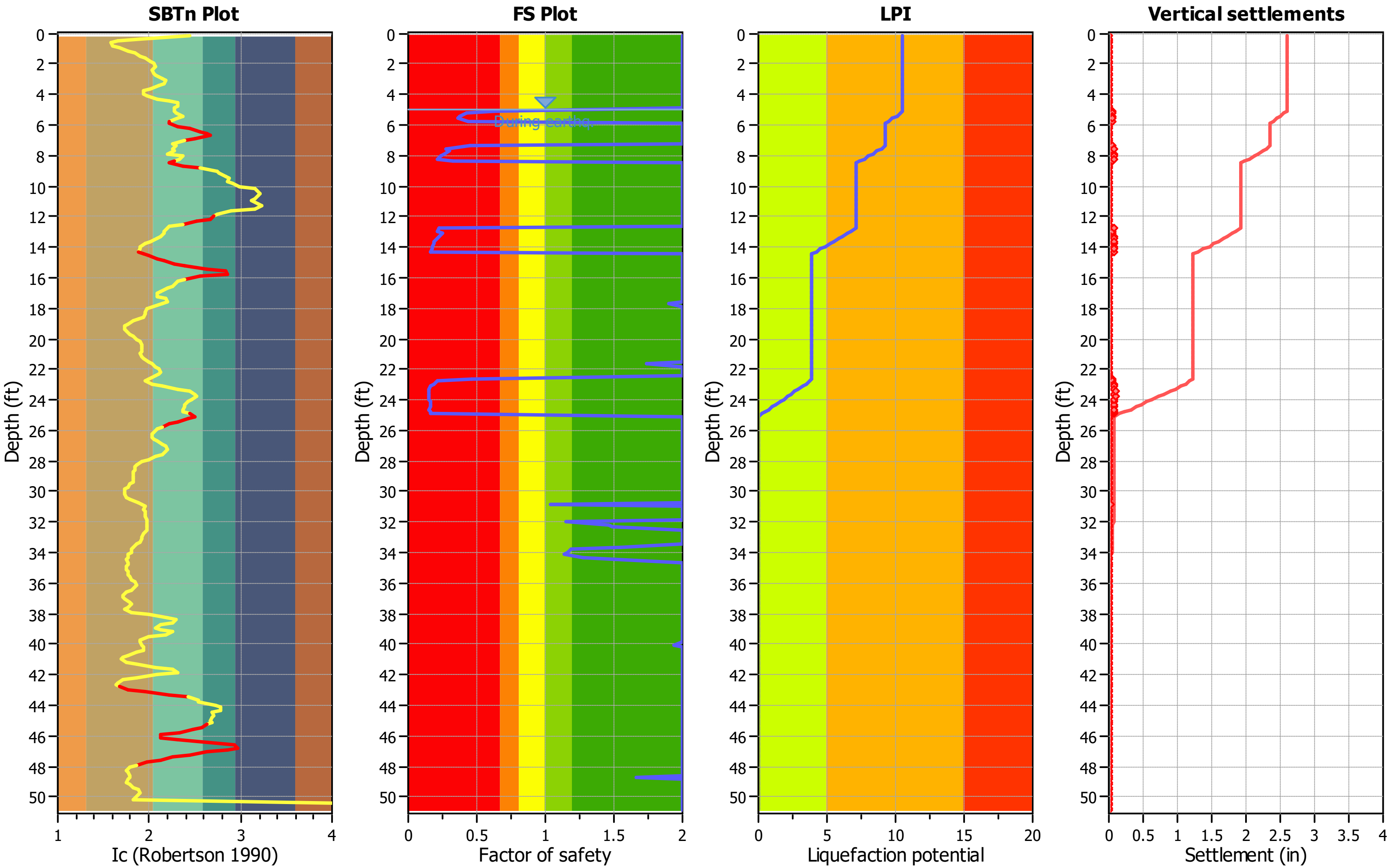


Analysis method:	B&I (2014)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	Yes
Earthquake magnitude M_w:	7.51	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	50.00 ft
Peak ground acceleration:	0.72	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

Ninyo & Moore
2149 O'Toole Avenue, Suite 30
San Jose, CA 95131
https://ninyoandmoore.com

Project: 403773009 - City of Alameda New Aquatic Center
Location: 1100 Atlantic Avenue, Alameda, California 94501

CPT: CPT-06
Total depth: 50.85 ft

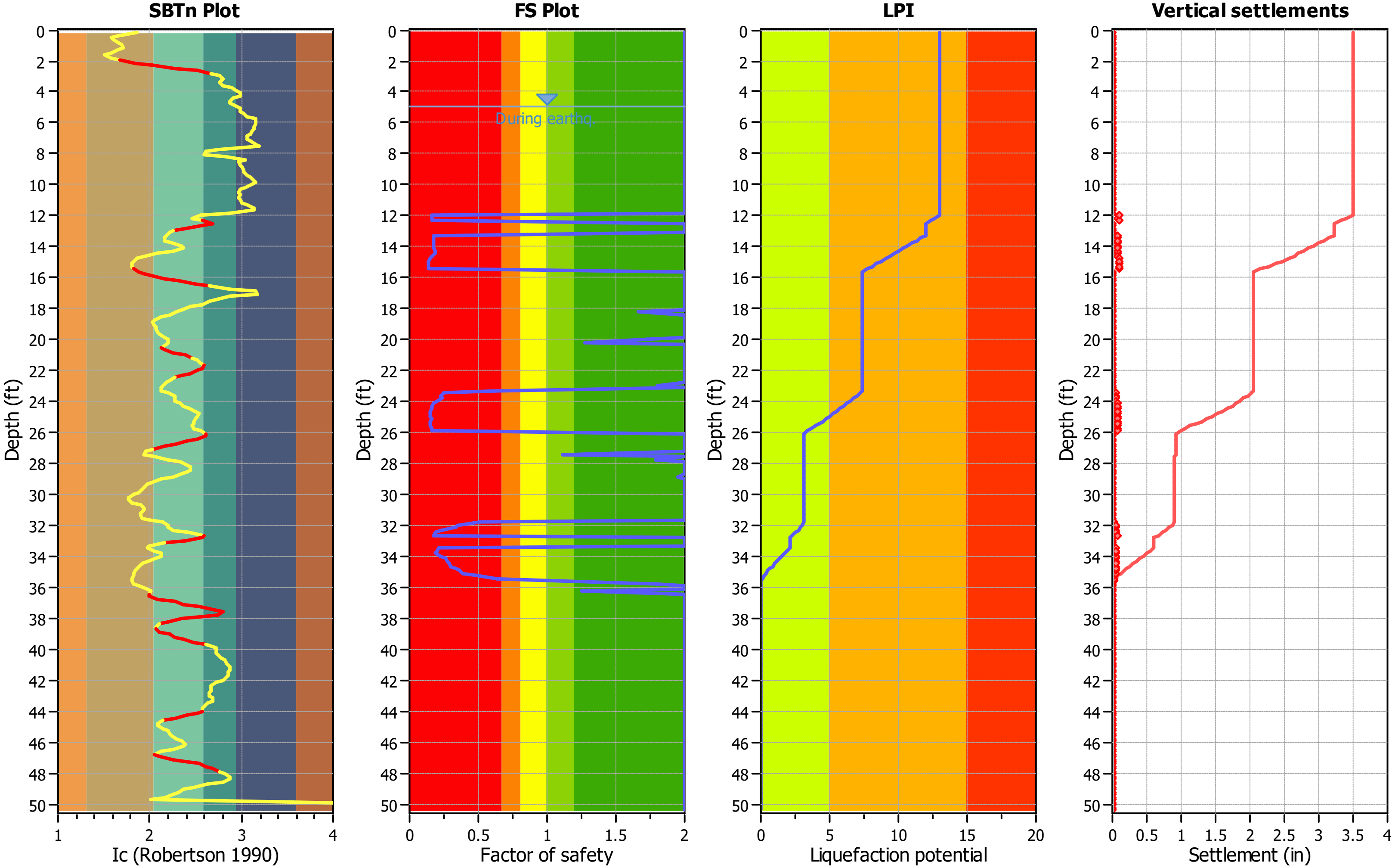


Analysis method:	B&I (2014)	G.W.T. (in-situ):	6.70 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.51	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.72	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Ninyo & Moore
2149 O'Toole Avenue, Suite 30
San Jose, CA 95131
https://ninyoandmoore.com

Project: 403773009 - City of Alameda New Aquatic Center
Location: 1100 Atlantic Avenue, Alameda, California 94501

CPT: CPT-07
Total depth: 50.36 ft

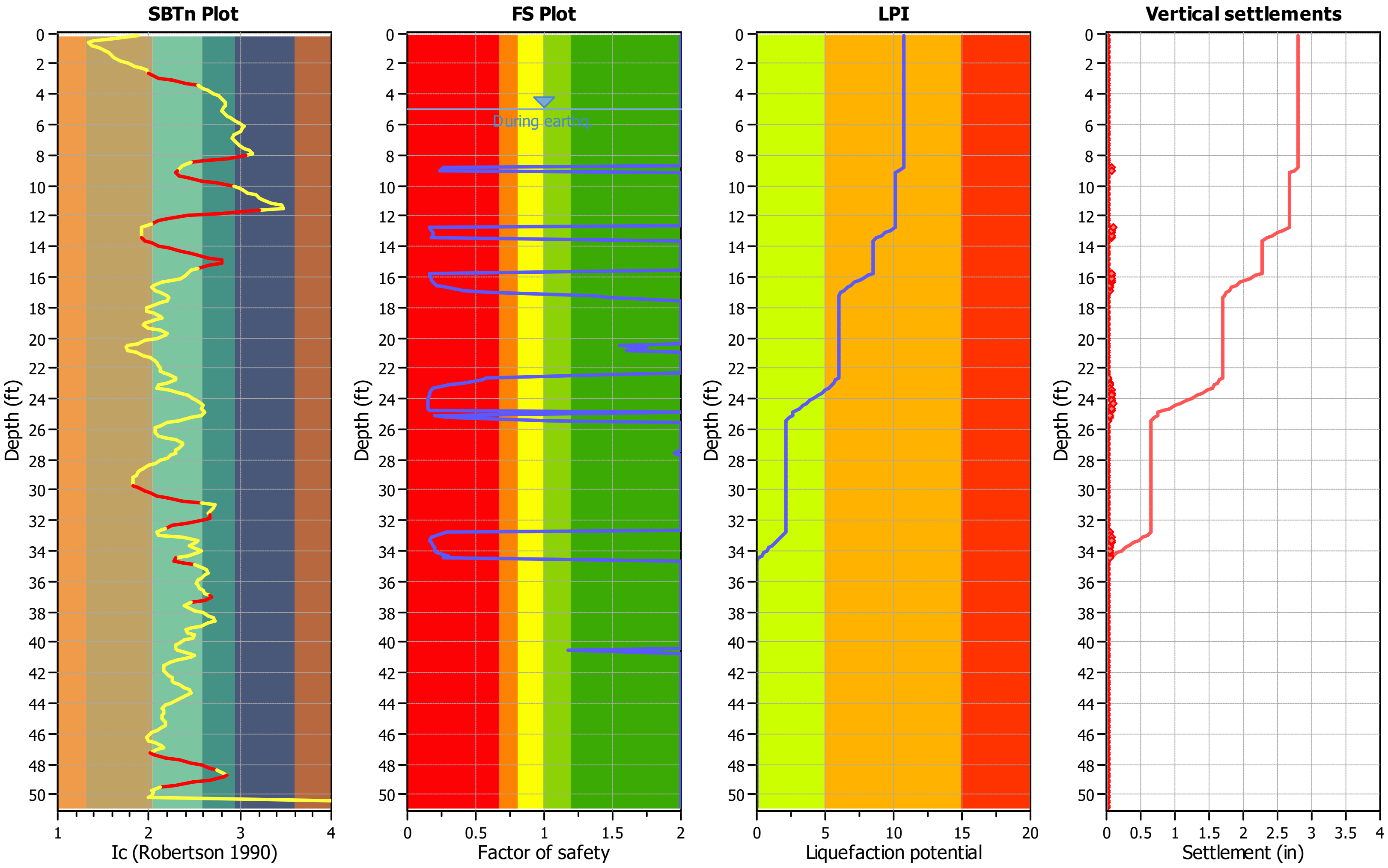


Analysis method:	B&I (2014)	G.W.T. (in-situ):	13.70 ft	Use fill:	No	Clay like behavior applied:	. Yes
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.51	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.72	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Ninyo & Moore
2149 O'Toole Avenue, Suite 30
San Jose, CA 95131
https://ninyoandmoore.com

Project: 403773009 - City of Alameda New Aquatic Center
Location: 1100 Atlantic Avenue, Alameda, California 94501

CPT: CPT-08
Total depth: 50.85 ft

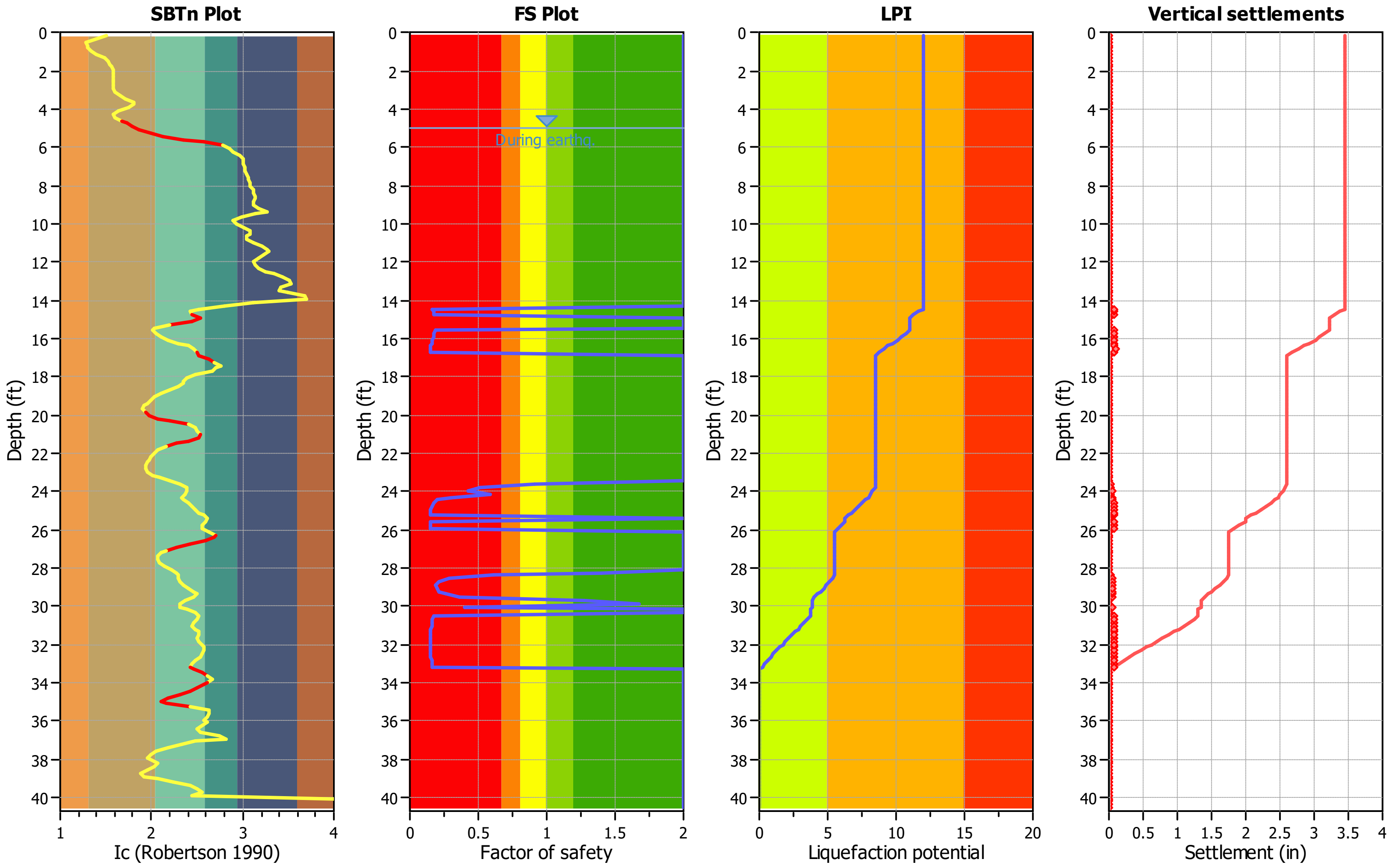


Analysis method:	B&I (2014)	G.W.T. (in-situ):	9.50 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.51	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.72	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Ninyo & Moore
2149 O'Toole Avenue, Suite 30
San Jose, CA 95131
https://ninyoandmoore.com

Project: **403773009 - City of Alameda New Aquatic Center**
Location: **1100 Atlantic Avenue, Alameda, California 94501**

CPT: CPT-09
Total depth: 40.52 ft

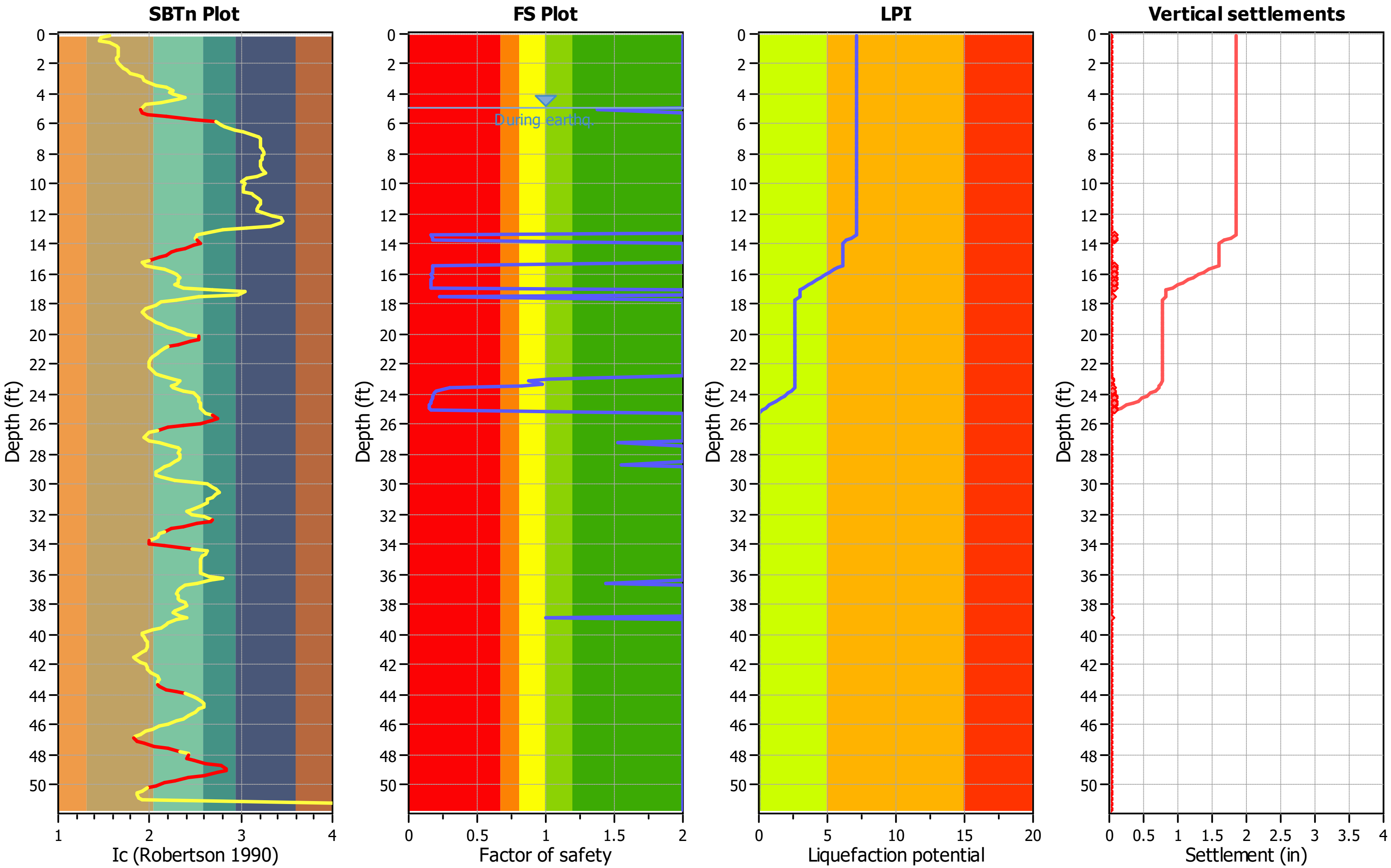


Analysis method:	B&I (2014)	G.W.T. (in-situ):	12.20 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.51	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.72	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Ninyo & Moore
2149 O'Toole Avenue, Suite 30
San Jose, CA 95131
https://ninyoandmoore.com

Project: **403773009 - City of Alameda New Aquatic Center**
Location: **1100 Atlantic Avenue, Alameda, California 94501**

CPT: CPT-10
Total depth: 51.67 ft



Analysis method:	B&I (2014)	G.W.T. (in-situ):	12.20 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.51	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.72	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Ninyo & Moore

A SOCOTEC COMPANY