Geotechnical Investigation and Geologic Hazards Evaluation Laney College Library Learning Resource Center

Oakland, California

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Peralta Community College District

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March 31, 2023

Dear Ms. Smith,

Fugro is pleased to submit this final geotechnical investigation and geologic hazards evaluation report for the proposed new Library Learning Resource Center project at Laney College in Oakland, California. Our work was authorized by the District professional service agreement (Requisition No. 2-129461, PO No. 3-118689) dated February 19, 2019, Amendment No. 1 (Requisition No. 2-135365, PO No. 3-122826) dated December 11, 2019, Amendment No. 2 (Requisition No. 2-145003, PO No. 3-131458) dated September 1, 2021, and the Amendment No. 3 (PO No. PCCD1-3000135642) dated August 3, 2022 and was executed in general accordance with the scopes listed in our Proposals No. 04.72189129-P-001(Rev.02), No. 04.72190021-P-001(Rev.00), No. 04.72160021-P-002(02), and No. 04.72190021-P-003 (01), dated February 19, 2019, July 19, 2019, July 12, 2021, and April 27, 2022, respectively.

This report was prepared to identify the key geologic and geotechnical aspects of the site and provide geotechnical recommendations for design and construction of the project. This report also summarizes the results of our geotechnical and geologic site data review, field exploration, laboratory testing, and geologic and seismic hazard evaluations for the project site. We appreciate this opportunity to be of service to the District. Should you have any questions or require additional information, please contact us.

Sincerely,

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1. Introduction

This report presents the results of the geotechnical investigation and geologic hazards evaluation conducted by Fugro USA Land, Inc. (Fugro) for the new Library Learning Resource Center on the Laney College campus. The campus is located at 900 Fallon Street in the City of Oakland and County of Alameda, California, as shown on the Vicinity Map (**Plate 1**). A topographic map of the area, along with coordinates for the site (Lat. 37.794899°N and Long.122.262363°W) are presented on the Topographic Site Map (**Plate 2**). Previously, Fugro performed a geotechnical study of the same site in 2002 and the results were presented in a report dated March 27, 2002.

This report was prepared in accordance with guidance from the California Geological Survey (CGS) – Note 48, *Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings*, (CGS, 2019), the American Society of Civil Engineers (ASCE) ASCE/SEI 7-16 Standard*, Minimum Design Loads for Buildings and Other Structures* (ASCE, 2016), and following the regulations of the 2019 California Building Code (2019 CBC; California Building Standards Commission, 2019).

Because the course of the design changed during the review process by California Geology Survey (CGS) and the Department of the State Architects (DSA), relevant information throughout the body of the previous report dated February 28, 2022, is updated in this report.

In addition, the design team decided to use Deep Mixing Method (DMM) ground improvement and shallow foundation system in lieu of proposed deep foundation and retaining wall system in our 2020 report. The detailed design assumptions, discussions, recommendations, and specifications are presented in DMM Design and Recommendations, **Supplement I**. Updated information and discussions shown in **Appendices I** and **J** supersede the similar subject in this report.

1.1 Project Description

According to the preliminary building layout plan provided by Noll & Tam Architects and Planners and as shown on the Site Plan (**Plate 3**), we understand that the proposed Library Learning Resource Center site is in the southeast corner of the Laney College main campus and is bounded by 7th Street on the southwest, Lake Merritt Channel on the east, a cooling tower structure and Building E on the northeast, and a handicap parking lot on the northwest. The site is located about 100 feet southwest of the Bay Area Rapid Transit (BART) underground tube easement. According to site survey information provided by CSW/Stuber-Stroeh Engineering Group, Inc. (April 2019), the existing surface elevations at the proposed building area varies from Elevations of +18 feet to +21 feet (NAVD 88).

The new building is planned to be an at grade, 3-story high building with an estimated footprint area of about 24,197 square feet and project size of 75,622 square feet. The proposed building location is about 130 to 160 feet away from the edge of the Lake Merritt Channel west bank. No significant raising of the existing site grade is anticipated for the project according to the project drawings provide by Noll & Tam Architects dated October 14, 2022.

At the time of our study, the site was occupied by several portable classroom buildings, a small bathroom structure, a small storage shed, and associated concrete walkways and landscaping. Short retaining walls up to about 3 feet high were located to the northeast of the classroom buildings, which retained the existing generally level pad of the existing improvements. Based on available aerial photographs of the site, these existing improvements appeared to be installed between August 2007 and September 2008. These improvements will be removed prior to the new construction.

1.2 Scope of Services

The purpose of our geotechnical investigation and geologic hazards evaluation was to identify key geotechnical, geologic hazards, and seismology aspects of the site in accordance with CGS Note 48 that could impact the project and provide geotechnical recommendations for design and construction of the project. The scope of our services performed included the following:

- Compile and review available geotechnical and geologic data that is contained in our files and provided by others, including existing geologic and seismic hazard maps and other generally available related literature.
- Review previous geotechnical investigation reports for the site and vicinity by Fugro and others, including results of previous exploratory borings, Cone Penetration Tests (CPT), and laboratory testing.
- Conduct a field exploration program including one (1) exploratory boring to a depth of about 76-1/2 feet and eight (8) CPTs to a maximum depth of about 75-1/2 feet;
- Perform geotechnical laboratory testing on selected soil samples for classification, index, strength, consolidation, and corrosivity testing.
- Identify the site geotechnical and geologic conditions (e.g., stratigraphy, subsurface soil characteristic and engineering properties, depths to groundwater, and geologic hazards) that could impact the project, as mandated by CGS Note 48.
- Perform engineering analyses using the field and laboratory data, including detailed liquefaction triggering, post-liquefaction deformation, dynamic densification, lateral spreading, and slope stability evaluations.
- Develop site-specific seismic design criteria per 2019 California Building Code (CBC), including a site-specific ground motion response analysis and a Probabilistic Seismic Hazard Analysis (PSHA).
- Respond the CGS and DSA review comments.

- Provide ground improvement design, drawings, and specifications.
- Communicate with the structural engineer and assist in finalizing the foundation design.
- Prepare this report to summarize the results of our geotechnical and geologic data review, field exploration, laboratory testing, geologic hazards evaluations, and engineering analyses, and to provide geotechnical conclusions and recommendations for design and construction of the project.

Chemical analytical assessment of onsite materials or groundwater for contaminants was beyond our scope of work.

2. Data Review, Exploration and Laboratory Testing

2.1 Review of Existing Data

As part of our study, Fugro reviewed relevant geotechnical, geologic, and seismic data, as well as results of previous explorations and laboratory testing performed in the vicinity of the project site, including the following reports, literature, and maps. The conclusions from our review of the existing data are presented in subsequent sections of this report.

2.1.1 Previous Geotechnical Data and Reports

- Woodward-Clyde-Sherard and Associates, March 9, 1966. *Soil Investigation for the Proposed Peralta Junior College Civic Center Site, Phase 1 – Preliminary Studies*, WCS No. S10312.
- Woodward-Clyde-Sherard and Associates, May 1, 1967. *Peralta College – Chinatown General Neighborhood Renewal Area (GNRA)*, WCS No. 11032.
- Kaldveer Associates, September 9, 1991. *Feasibility Foundation Investigation, Proposed Pool Improvements, Laney College*, Kaldveer No. K1329-1-863.
- Harza Kaldveer, October 22, 1993. *Geotechnical Investigation for Proposed Pool Replacement, Laney College*, Harza No. K1329.
- Fugro, March 27, 2002. *Geotechnical Investigation, New Art Building at Laney College*, Fugro No. 1430.001.
- Fugro, March 29, 2005. *Geotechnical Study and Geologic Hazard Evaluation, Laney College Art Building*, Fugro No. 1430.005.
- Geotechnical Engineering Inc., March 20, 2006. *Additions to Building A & Chiller Room Adjacent to Building B, Laney College*, GEI No. 41357.
- Fugro, August 25, 2006. *Geologic Hazards Evaluation, Laney College Building A Renovation*, Fugro No. 1430.008.
- Fugro, June 10, 2008. *Geotechnical Review, Proposed New Laney College Library Site Study*, Fugro No. 1813.002.
- Terraphase Engineering, May 31, 2012. *Geotechnical Design Report, Proposed Laney College Building Efficiency for a Sustainable Tomorrow (BEST)*, Terraphase No. 0034-001-003.

2.1.2 Geologic Maps, Literature, and Hazard Zonation Maps

- Witter, Knudsen, Sowers, Wentworth, Koehler, and Randolph, 2006. *Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California*, USGS Open File Report 2006-06-1037.
- Helley and Graymer, 1997. *Quaternary Geology of Alameda County, and Parts of Contra Costa, Santa Clara, San Mateo, San Francisco, Stanislaus, and San Joaquin Counties, California: A Digital Database*, USGS Open File Report 97-97.

- Rogers and Figuers, December 30, 1991. *Engineering Geologic Site Characterization of the Greater Oakland-Alameda Area, Alameda and San Francisco Counties, California*, NSF Grant No. BCS-9003785.
- California Geological Survey, *Earthquake Fault Zones, Oakland West Quadrangle*, Revised Official Map, Released: January 1, 1982.
- California Geological Survey, *Seismic Hazard Zones, Oakland West Quadrangle*, Official Map, Released: February 14, 2003.
- California Geological Survey, 2003. *Seismic Hazard Zone Report for the Oakland West 7.5-Minute Quadrangle, Alameda County, California*, Seismic Hazard Zone Report 081.
- Holzer, Bennett, Noce, Padovani, and Tinsley, 2002, revised 2010. *Liquefaction Hazard and Shaking Amplification Maps of Alameda, Berkeley, Emeryville, Oakland, and Piedmont, California: A Digital Database*, USGS Open File Report 2002-02-296.
- Holzer, 1998. *The Loma Prieta, California, Earthquake of October 17, 1989 - Liquefaction*, USGS Professional Paper 1551-B.
- Youd and Hoose, 1978. *Historical Ground Failures in Northern California Triggered by Earthquakes,* USGS Professional Paper 993.
- California Geological Survey, July 31, 2009. *Tsunami Inundation Map for Emergency Planning, Oakland West Quadrangle*.
- Federal Emergency Management Agency, *Flood Insurance Rate Map (FIRM), Panel 06001C0067H (12/21/18)*.
- City of Oakland Community and Economic Development Agency, November 2004, *Safety Element, City of Oakland Safety Plan*.

2.2 Field Exploration

Fugro performed a geotechnical field exploration program that consisted of one (1) exploratory boring to a depth of about 76-1/2 feet and eight (8) CPTs (Cone Penetration Tests) to a maximum depth of about 75-1/2 feet on March 29, 2019, and January 2, 3, and 7, 2020. In addition, three (3) shallow hand auger borings to a maximum depth of about 6 feet were also performed at three (3) CPT locations (2019-CPT-1 through 2019-CPT-3). During the design of the Deep Mixing Method (DMM) ground improvement and to better define the bottom of Young Bay Mud (YBM) layer, we performed 10 additional CPTs to a maximum depth of about 100 feet on November 17, 18, and 22, 2022. The new CPTs are used to develop cross sections for the DMM Design and Recommendations, **Supplement I**. The approximate locations of the borings and CPTs are shown on the Site Plan (**Plate 3**). The locations were determined by pacing or tape measurement from field landmark references; and should be considered accurate only to the degree implied by the method used.

Drilling permits were attained from Alameda County Public Work Agency (ACPWA) for the subsurface explorations. Underground Service Alert (USA) was notified, and a private utility locating company, Bess Testlab, Inc. (BTL) of Hayward, California, was retained to clear the boring and CPT locations prior to explorations. In addition, a hand auger was also used to clear the top 5 to 6 feet of soils for utilities below existing ground surface at some of the boring and CPT locations.

The boring was performed by a State of California C-57 licensed driller, Geo-Ex Subsurface Exploration (GeoEx) of Dixon, California, using a track-mounted CME 75 drill rig equipped with a mud rotary wash system and a 140-lb automatic trip hammer. According to a hammer calibration report provided by Geo-Ex, the 140-pound automatic trip hammer used at the site for soil sampling had been rated as having an average energy transfer ratio of about 91 percent (calibrated on December 18, 2018).

CPTs were performed by both Fugro and Gregg Drilling, LLC (Gregg) of Martinez, California, in general accordance with ASTM D5778. Fugro used a 25-ton truck-mounted rig with an electronic piezocone penetrometer that has a tip area of 15 cm^2, a friction sleeve area of 225 cm^2, and a tip end area ratio of 0.59. Gregg used a 20 and 25-ton truck-mounted rig and a self-anchoring mini track-mounted rig with an electronic piezocone penetrometer that has a tip area of 15 cm^2, a friction sleeve area of 225 cm^2, and a tip end area ratio of 0.8. The cones were advanced at a standard rate of 2 cm/sec into the ground to measure tip resistance, sleeve friction, and excess pore pressure. Pore water pressure dissipation tests were also performed at selected depths. In addition, in-situ soil shear wave velocity measurements were performed at an approximate 5-foot interval at the 2020-CPT-07 location. The CPT logs and interpretations are presented in **Supplement A**.

Our field engineer continuously logged soils encountered in the borings in the field. The soils are classified in general accordance with the Unified Soil Classification System (ASTM D2487 and D2488). The logs of the borings as well as a key for the classification of the soils are included in **Supplement A**. Upon completion of our field explorations, the borehole and CPT holes were backfilled with neat cement grout in accordance with ACPWA requirements. All drilling derived soil cuttings and fluids from mud rotary wash drilling were containerized in 55-gallon metal drums and transported to appropriate facilities for disposal by Geo-Ex.

Representative soil samples were obtained during drilling using a Modified California split-barrel drive sampler (outside diameter of 3.0 inches, inside diameter of 2.5 inches) and a Standard Penetration Test (SPT) split-barrel drive sampler (outside diameter of 2.0 inches, inside diameter of 1.375 inches). Soil samples were transmitted to laboratories for evaluation and appropriate testing. The sampler types are indicated in the "Sampler" column of the boring log as designated in **Plate A-1**.

Resistance blow counts were obtained with the drive samplers by dropping a 140-pound automatic trip hammer through a 30-inch free fall in general accordance with ASTM D1586. The samplers were driven 18 inches, or a shorter distance where hard resistance was encountered, and the number of blows were recorded for each 6 inches of penetration. The blows per foot recorded on the boring logs represent the accumulated number of blows that were required to drive the last 12 inches. When the SPT split spoon sampler was used, these blow counts are the standard penetration resistance values (N values). However, due to the large diameter of the Modified California sampler, the blow counts recorded for this sampler are not standard penetration resistance values. These values were multiplied by a conversion factor of 0.63 for the Modified California Sampler and the calculated approximate equivalent N values are presented on our logs within parenthesis. No hammer energy correction had been applied on the N values presented on the logs.

Previously, several exploratory borings and CPTs were performed in 2002 by Fugro and in 1965 by Woodward-Clyde-Sherard and Associates (WCS) at the site and vicinity. The approximate locations of these previous explorations are also shown on the Site Plan (**Plate 3**). Logs of these previous explorations and laboratory testing results are included in **Supplement C** for reference. The results of these previous explorations and laboratory testing are also incorporated into this report.

2.3 Laboratory Testing

Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site. This program included:

- Fifteen (15) moisture content and dry unit weight determinations per ASTM D2937,
- Eight (8) hydrometer, sieve, and percent passing #200 sieve analyses per ASTM D422 and D1140,
- One (1) plastic and liquid limits per ASTM D4318,
- Two (2) unconsolidated undrained triaxial shear strength tests (TXUU) per ASTM D2850,
- One (1) incremental consolidation test per ASTM D2435, and
- Three (3) organic content determinations per ASTM D2974.

All tests were performed by Fugro's geotechnical laboratory in Ventura, California and Cooper Testing Laboratory in Palo Alto, California. Our laboratory testing results are included in **Supplement B**. Some of the test results are also presented on the boring logs (**Supplement A**) at the corresponding sample depths.

Corrosivity tests that include redox, pH, chlorides, sulfates, and resistivity were performed by CERCO Analytical, Inc. in Concord, California, on two representative onsite near-surface soil samples (from 2019-CPT-01 at about 2-1/2 feet and 2019-CPT-03 at about 4 feet). The test results and a brief evaluation report prepared by CERCO regarding the onsite near-surface soil corrosivity are also included in **Supplement B**.

3. Geologic and Seismic Setting

This section summarizes the regional geologic and tectonic setting, the local geologic setting, the site geology, and regional active faults and seismicity.

3.1 Regional Geologic and Tectonic Setting

The project site is located near the east shore of the San Francisco Bay in the Coast Ranges geomorphic province (CGS, 2002). The Coast Ranges are northwest-trending mountain ranges, typically rising to 2,000 to 4,000 ft. in elevation, with intervening elongated valleys. The oldest rocks in the range were formed from the subduction of the Farallon Plate beneath the North American Plate during the Jurassic and Cretaceous periods. The Franciscan Formation in the San Francisco Bay area is a complex of graywacke sandstone, shale, and other lithologies that accumulated in the offshore trench in the subduction zone, then were pushed up onto the continent. Later, Tertiary continental sediments and volcanic rocks were deposited over the Franciscan Formation.

Subduction was followed by strike-slip faulting along the San Andreas fault system starting in southern California about 28 million years ago as subduction gradually consumed the Farallon Plate, and the Pacific Plate and the North American Plate boundary migrated northward. The strike-slip motion along faults in the San Francisco Bay Area developed over the past 5 to 10 million years (Atwater, 1970; Wallace, 1990; Atwater & Stock, 1998). At the present time, the San Andreas fault and sub-parallel faults such as the Hayward fault form the boundary zone between the Pacific and North American plates in the San Francisco Bay area. Deformation over the past few million years along various faults of the San Andreas fault system has produced a series of northwest-trending valleys and mountain ranges, including the East Bay Hills, the San Francisco Peninsula, and the intervening San Francisco Bay.

The Hayward fault extends along the western front of the East Bay Hills along the east side of San Francisco Bay and forms an approximate boundary between two distinctly different geologic and physiographic provinces. Based on work by Radbruch (1969), basement rocks underlying the area west of the Hayward fault are primarily those of the Jurassic to Cretaceous Franciscan Complex (about 200 to 80 million years old). East of the Hayward fault, the basement rocks are Jurassic to Cretaceous sedimentary rocks of the Great Valley Sequence (about 140 to 65 million years old). These Mesozoic rocks are overlain by Tertiary volcanic and sedimentary rocks (65 to 2.5 million years old) in the East Bay hills. The San Francisco Bay Area experienced several episodes of uplift and faulting during late Tertiary time (about 25 to 2 million years ago).

The surficial deposits of the flatlands that lie between the hills and the bay are derived from erosion of the Mesozoic and Tertiary rocks in the hills. They are Quaternary in age, or less than about 2 million years old, and consist primarily of alluvial deposits laid down by streams draining the hills. These deposits form and underlie the wide, gently sloping East Bay Plain and provide the relatively level building sites for most of the development in the East Bay. Sediments that reach the bay are deposited as estuarine deposits in the tidal marshes, mud flats, and the floor of the bay.

The position of the Bay shoreline varied throughout the Quaternary as sea level rose and fell in response to glacial cycles. During peak of the last major glaciation, around 15,000 years ago, sea level was about 330 feet lower than it is today and the San Francisco Bay was a wide valley with streams flowing across the valley floor, joining together to flow out the Golden Gate and finally meeting the sea near the Farallon Islands. As the ice from the great continental glaciers melted, sea level began to rise, with the sea entering the Bay about 10,000 years ago. The present sea level was reached within the Bay about 6,000 years ago (Atwater et al., 1977).

As a result of these sea level fluctuations, the thick sequence of sediments in and adjacent to the bay includes layers of estuarine silts and clays deposited during interglacial periods, alternating with layers of sandy alluvial deposits laid down during glacial periods (Atwater et al., 1977; Sloan, 2006). Borehole data to depths of 300 feet in the central part bay show strata from as many as four glacial-interglacial cycles.

3.2 Local Geologic Setting

In the area of the site, thick Quaternary deposits overlie the basement rocks. The Quaternary deposits represent several stages of deposition, which have taken place over the last 2 million years or so. The combined thickness of the sediments above the Franciscan bedrock is estimated to be on the order of 500 feet based on deep boreholes drilled in downtown Oakland (Rogers & Figuers, 1991).

Structurally, the project site is in an area dominated by the active San Andreas Fault system that includes from west to east, the San Gregorio, San Andreas, Hayward-Rodgers Creek, Calaveras, Concord-Green Valley, and Greenville faults, as well as many other minor faults. The Hayward fault borders the western margin of the East Bay Hills in the eastern San Francisco Bay Area. The site lies about 2-1/2 miles southwest of the toe of Oakland Hills, which are part of the Diablo Range that separates the San Francisco Bay from the San Joaquin Valley. The nearest bodies of surface water are the Oakland Inner Harbor, located about 1/2 mile to the south, and Lake Merritt, located 1/4 mile to the north.

3.3 Site Geology

According to Witter et al. (2006), and as shown on the Quaternary Geologic Map (**Plate 4**), the site is located bayward of the historical shoreline, on former tidal flats adjacent to the Lake Merritt Channel that were filled to make land. The site is roughly in the middle of the estimated

500- to 1,400-foot-wide natural outlet channel of Lake Merritt, which had been dramatically reduced in width with development of the region after the 1860s. Filling of this area occurred between 1894 and 1915 based on the study by Rogers and Figuers (1991).

The historical artificial fill overlies Holocene estuarine mud (afem), which is known locally as Young Bay Mud. According to Helley and Graymer (1997), most of the fill placed before 1965 in San Francisco Bay Area was not compacted and consists of dumped or hydraulically emplaced materials. Based on the results of subsurface geotechnical explorations, the site is generally underlain by about 8 to 25 feet thick of heterogenous man-made fills that locally contain various amounts of concrete, brick, and wood debris.

The Young Bay Mud is a water-saturated estuarine deposit, predominantly gray, green and blue clay and silty clay deposited in tidal marshlands and mud flats of San Francisco Bay. The mud generally contains a few lenses of well-sorted, fine sand and silt, a few shelly layers, and peat. The Young Bay Mud was deposited during the post-Wisconsin rise in sea-level, about 12,000 years to present, and interfingers with and grades into fine-grained alluvial deposits at the distal edge of Holocene alluvial fans.

3.4 Regional Faulting and Seismicity

The San Francisco Bay Area is recognized by geologists and seismologists as one of the most seismically active regions in the United States. As described in Section 3.1 and as shown on the Regional Fault and Seismicity Map (**Plate 5**), numerous major fault zones cross through the San Francisco Bay Area, generally trending northwest-southeast. These faults and other local faults have produced many strong earthquakes, magnitude 6.0 and greater, over the last two centuries within about 60 miles (100 km) of the site, as detailed in Section 3.5.

As shown on **Plate 5**, the site is located about 3.5 miles southwest of the Hayward fault zone. The Alquist-Priolo (AP) Earthquake Fault Zone Map of the Oakland West Quadrangle (**Plate 6**) shows that the site is not located within an earthquake fault zone, as designated by the State of California (California Geological Survey (CGS), 1982).

The Hayward fault exhibits typically geomorphic evidence of Holocene (less than 11,000 years) displacement such as shutter ridges, offset drainages, and aligned topographic sags and scarps. The Hayward fault zone varies in width, from relatively narrow traces of 5 to 10 meters in width, to a zone of subparallel strands several hundred meters wide, or more in fault stepovers. Fault creep occurs at the ground surface along most of the Hayward fault, with average measured creep rates of about 4 to 5 mm/year in the Oakland-Berkeley area (WGCEP, 2003).

Active faults located within about 60 miles (100 km) of the project site, and their generalized fault rupture parameters from the U.S. Geological Survey (USGS) are summarized in Table 3.1**.**

Table 3.1: Regional Active Faults and Generalized Rupture Parameters

Fault	Approximate Closest Distance from Site to Fault (miles)	Direction from Site to Fault	Estimated Maximum Moment Magnitude (Mw)	Fault Length (km)	Slip Rate (mm/yr)
Hayward-Rodgers Creek	3.4	NE	7.3	150	9
Mount Diablo	13.4	N	6.7	25	2
Calaveras	13.9	NE	7.0	123	15
San Andreas	14.6	SW	8.0	472	17
Green Valley	16.5	NE	6.8	56	4.7
San Gregorio	18.9	SW	7.5	176	5.5
Greenville	24.2	E	7.0	50	2
Monte Vista-Shannon	24.6	S	6.5	45	4
West Napa	25.5	N	6.7	30	1
Great Valley 5 Pittsburg Kirby Hills (Closest Section)	27.4	NE	6.7	32	1
Point Reyes	32.9	NW	6.9	47	0.3
Hunting Creek-Berryessa	45.5	N	7.1	60	6
Zayante-Vergeles	51.2	SE	7.0	58	0.1
Maacama-Garberville	58.7	NW	7.4	221	9
Sources: *USGS, 2008. National Seismic Hazard Maps – Source Parameters* website					

Earthquakes on the faults in Table 3.1 or on smaller, mapped or unmapped faults could cause strong ground shaking at the site. A USGS Fact Sheet (Aagaard et al., 2016) indicates there is a 72 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2014 and 2043. Earthquake intensities will vary throughout the San Francisco Bay Area depending upon the magnitude of the earthquake, the distance of the site from the causative fault, the type of materials underlying the site, and other factors.

According to 2019 CBC and ASCE 7-16, and based on an average soft clay soil site condition (Site Class E), the site geometric mean peak ground acceleration (PGA_M) from the Maximum Considered Earthquake (MCE) is estimated to be about 0.80g. The MCE peak ground acceleration has a 2 percent probability of being exceeded in 50 years (a mean return period of 2,475 years), except where deterministically capped along highly active faults.

3.5 Historical Seismicity

Major earthquakes have been recorded along the San Andreas Fault system and across California since the late 1700s. Table 3.2 presents large magnitude (M≥6.0) regional earthquakes within about 60 miles (100 kilometers) of the site from 1800 to 2018, arranged in chronological order. The Northern California Earthquake Data Center (NCEDC) and National Atlas of United States database was accessed to obtain the historical seismicity information presented in Table 3.2. The epicenter locations are shown on the Regional Fault and Seismicity Map (**Plate 5**).

Table 3.2: Large Magnitude (M≥6.0) Earthquakes Within About 60 Miles (100 km) of the Site

Epicenter Location	Date	Magnitude	Distance (mi)	Direction from Site to Epicenter
Near San Francisco	6/21/1808	6.0	13.1	W
In the San Francisco Bay Area	6/10/1836	6.8	3.4	E
In the San Francisco Area	6/1838	7.0	15.5	SW
North of San Jose	11/26/1858	6.1	28.4	SE
In the Santa Cruz Mountains	10/8/1865	6.3	45.6	SSE
Near Hayward	10/21/1868	6.8	11.0	SE
West of Antioch	5/19/1889	6.0	24.3	NE
Near Vacaville	4/19/1892	6.4	44.4	NNE
Near Winters	4/21/1892	6.2	52.5	NNE
Near Mare Island	3/31/1898	6.2	28.9	NNW
Near San Francisco	4/18/1906	7.8	14.8	SW
Near Coyote Hills	7/1/1911	6.6	46.9	SE
Near Morgan Hill	4/24/1984	6.2	45.1	SE
Loma Prieta	10/17/1989	6.9	56.3	SSE
Napa	8/24/2014	6.0	29.1	N

Several of these events were strong enough to cause structural damage to buildings in Oakland. The estimated M6.8 1868 Hayward earthquake ruptured the Southern Hayward fault from Hayward northward to Oakland and damaged or destroyed numerous buildings in Hayward, San Leandro, Oakland, and San Francisco (Lawson, 1908). The 1868 Hayward earthquake apparently resulted in damage in Oakland corresponding to Modified Mercalli Intensity (MMI) VIII (partial damage to buildings, walls); however, there is little reported information on ground shaking and damage in Oakland, largely because the area was sparely populated at the time of these earthquakes (Toppozada & Park, 1982; Toppozada, 2000).

During the moment magnitude (M_W) 7.8 1906 San Francisco earthquake, the San Andreas fault ruptured over a distance of about 296 miles (474 km) from Shelter Cove near Cape Mendocino southward to near San Juan Bautista. Maximum lateral displacements of 15 to 20 feet (4.6 to 6.1 meters) occurred north of the Golden Gate at Olema in Marin County (Lawson, 1908). Landslides, liquefaction, and ground settlement occurred throughout the Bay Area and in the vicinity of the surface rupture as a result of this earthquake. The ground shaking in Oakland during the 1906 earthquake is characterized as MMI VII to IX (minor to major damage to and collapse of structures; Lawson, 1908; Boatwright & Bundock, 2005). Significant damage occurred to masonry buildings across the city (Lawson, 1908). Ground failure effects, including liquefaction, lateral spreading, and settlement occurred in several areas along the Oakland-Alameda Estuary and at the southern end of Lake Merritt, (Youd & Hoose, 1978; Knudsen et al., 2000).

The most significant recent seismic event to occur in the San Francisco Bay Area was the October 17, 1989, Loma Prieta earthquake. The epicenter of this earthquake was located approximately 56 miles southeast of the site. This moment magnitude 6.9 earthquake ruptured a 22-mile (35-km) section of a splay of the San Andreas fault. The 1989 Loma Prieta earthquake produced MMI VII to VIII effects in the vicinity of the site (McNutt & Toppozada, 1990). Specific ground failure effects near the project site at Lake Merritt and the estuary channel resulting from the 1989 Loma Prieta and 1906 San Francisco earthquakes are described in the Section 5.2.1.

In addition to the damage from liquefaction near the site, the 1989 Loma Prieta Earthquake caused minor to significant damage to structures in the vicinity of the project site, including collapse of the elevated Cypress Structure on the west side of Oakland, and damage to buildings in downtown Oakland. The recorded peak ground acceleration from strong ground motion stations near the site is listed in Table 3.3; the nearest sites (within one mile of the project site) had PGAs of 0.18 to 0.26g.

Table 3.3: Strong Ground Motion Recordings from the 1989 Loma Prieta Earthquake

Station Number and Name	Site Conditions	Peak Ground acceleration (g)	Station Distance from Site (Miles)
58483 – Oakland – 24 story residential building	Alluvium	0.18 (ground)	0.35 NE
58224 – Oakland Title Ins. & Trust – 2 story building	Alluvium	0.26 (ground) 0.21 (revised NGA)	0.8 NNW
58334 – Piedmont – 3 story school office building	Serpentinite	0.08 (ground) 0.18 (structure)	2.4 NE
58338 – Piedmont Junior High School grounds	Weathered serpentinite	0.08 (ground)	2.5 NE
58472 – Oakland-Outer Harbor Warf	Fill/Bay Mud	0.29 (ground)	3.4 WNW
1662 – Emeryville – 6363 Christie	Alluvium	0.25 (ground, revised NGA)	3.8 NW

Station Number and Name	Site Conditions	Peak Ground acceleration (g)	Station Distance from Site (Miles)
Data from U.S. National Center for Engineering Strong Motion Data (URL: http://www.strongmotioncenter.org/), and the Pacific Earthquake Engineering Research (PEER) Center Next Generation Attenuation (NGA) Database (http://peer.berkeley.edu/nga/earthquakes.html)			

4. Site Conditions

This section describes historical and present land use and topography at the project site, subsurface soils and geologic strata, and groundwater conditions based on project geotechnical data.

4.1 Surface Conditions

At the time of our study and as shown on the attached Site Plan (**Plate 3**), the proposed Library Learning Resource Center site is in the southeast corner of the Laney College main campus and is bounded by 7th Street on the southwest, Lake Merritt Channel on the east, a cooling tower structure and Building E on the northeast, and a handicap parking lot on the northwest.

The site is occupied by several portable classroom buildings, a small bathroom structure, a small storage shed, and associated concrete walkways and landscaping. Several large and small diameter trees were located around the perimeter of the site. Short retaining walls up to about 3 feet high are located to the northeast of the classroom buildings, which retained the existing generally level building pad. Based on available aerial photographs of the site, these existing improvements appeared to be installed between August 2007 and September 2008.

According to site survey information provided by CSW/Stuber-Stroeh Engineering Group, Inc. (April 2019), the existing surface elevations at the proposed building location varies from Elevations of +18 feet to +21 feet (NAVD 88). The areas to the east of the proposed building location sloped gently downward toward the Lake Merritt Channel with inclinations of about 6:1 (horizontal to vertical) to 10:1. The top of the adjacent channel bank is at about Elevation of 7 feet.

Comparing the topographic information contained on the site plan Figure 1 of the 2002 Fugro report, the current site grade appears to have been modified to create the generally level pad for the portable classroom buildings. We estimated minor cut and fill grading of up to about 2 to 3 feet had been performed at the site during the portable classroom development in 2007 or 2008. The actual details of the previous grading are unknown. We recommend any available previous grading and construction records be forwarded to us for further review.

In addition, based on our review of historical USGS topographic maps from 1915 to 1980 and aerial photographs of the site vicinity from 1993 to 2018, it is our understanding that the Lake Merritt Channel had been re-aligned and widened in 1970s to the current alignment. In the site area, the old channel west bank was located about 140 feet east of the current west bank.

4.2 Subsurface Conditions

The subsurface soil conditions encountered by our borings and CPTs at the proposed Library Learning Resource Center site are consistent with Quaternary geologic mapping of the project site vicinity that shows artificial fill overlying estuarine mud. Similar subsurface soil conditions were also reportedly encountered by previous borings and CPTs by Fugro and others in 1965 and 2002 at the site and vicinity. Our interpretations of the site subsurface soil conditions are presented on the Cross-Sections A-A' through E-E' (**Plates 7** through **11**, respectively).

The subsurface soils below the site generally consisted of predominately medium dense sandy fills that extended to depths of about 8 to 25 feet (Elevation of about +8 feet to -5 feet). Clayey fills of about 2 to 4 feet thick were also encountered in some areas. These fills are heterogenous and locally contain various amounts of concrete, brick, and wood debris. An unknown obstruction was also previously encountered at about 5 feet deep at the 2002-CPT-1 location. Most of these fills appear to be derived from the historical filling of the natural Lake Merritt outlet channel between 1860s and 1940s, and the later development of the Laney College campus in 1960s. Most likely these fills were not compacted to current acceptable geotechnical engineering standards.

Below the surficial fill layer, very soft to soft, high moisture content, and low shear strength Young Bay Mud was encountered to a depth of about 30 feet (Elevation of about -10 feet) at the northwest side of the proposed building location and about 50 feet (Elevation of about -30 feet) at the southeast side of the proposed building location. Some thin loose to medium dense sand lenses about 2 to 6 feet thick were also encountered within the Young Bay Mud layer. About 15-feet of loose to medium dense sands were also encountered between the surficial fill and the Young Bay Mud layers in 2019-CPT-3. These sands could be either historical fills placed in the natural Lake Merritt outlet channel or natural sand deposits that existed within the channel.

Underlying the Young Bay Mud layer, medium dense to very dense sands and stiff to hard clays were encountered to the maximum depth explored of about 76-1/2 feet (or elevation of about -60 feet).

The thin surficial layers of clayey fills are considered to have a low to medium plasticity and low to moderate expansion potential; the sandy fills are non-expansive. Our logs and interpretations of borings and CPTs are presented in **Supplement A**. Our laboratory testing results of the onsite soil samples are included in **Supplement B**. Logs of historic explorations and results of lab testing are included in **Supplement C** for reference.

4.3 Groundwater

Based on CPT pore pressure dissipation tests at selected depths, the site groundwater table is estimated to be at depths of about 5 to 18 feet (Elevations of about 0 to +9 feet). In addition, groundwater was reportedly encountered at 2002-CPT-2 location at a depth of about 11 feet (Elevation about +8 feet). The previous borings (2002-EB-1 through 2002-EB-3) also reportedly encountered groundwater at depths of about 15 to 45 feet (Elevations of about +5 to -27 feet). It should be noted that these borings might not have been left open for a sufficient period of time to establish equilibrium groundwater conditions. Fluctuations in the groundwater level could occur due to change in seasons, variations in rainfall, tidal effects, and other factors. According to CGS Seismic Hazard Zone report for the Oakland West Quadrangle (CGS, 2003), as shown on **Plate 13**, historically high groundwater in the site region had been reported at a depth of about 10 feet.

We recommend a design groundwater Elevation of +8 feet be used for the project designs, which generally corresponds to both the top elevation of Young Bay Mud layer within the project area and the top elevation of the adjacent Lake Merritt Channel bank.

5. Geologic Hazards Evaluation

Site geologic hazard evaluations were performed in accordance with guidance from the CGS Note 48 (CGS, 2019). The opinions, conclusion, and recommendations in the following sections were based on the results of our review of available information relating to geotechnical, geologic, and seismic data within the vicinity of the site, project field exploration and laboratory programs, and site-specific engineering analyses.

Hazard evaluations are grouped into five sections, addressing: 1) fault rupture, 2) seismic ground shaking effects (liquefaction, dynamic densification, and lateral spreading, 3) slope stability, 4) compressible soils (settlement of non-engineered fills and young sediments), and 5) other hazards (expansive soils, corrosive soils, volcanic eruptions, flooding and dam inundation, tsunami and seiche, naturally occurring asbestos, and hydrocompaction).

5.1 Fault Rupture Hazard Evaluation

Surface fault rupture occurs when an earthquake results in displacement of the ground surface along the trace of an active fault. Based on existing geologic maps and literature, there are no known active fault traces within, adjacent to, or trending towards the project site. The closest known active fault is the Hayward Fault, located approximately 3.6 miles (5.8 kilometers) to the northeast. The site is not located within a Fault-Rupture Hazard Zone, as shown on the Earthquake Fault Zone Map for the Oakland West Quadrangle (CGS, 1982). No other faults are mapped or know to occur near the project site. Based on this information, the potential for surface fault rupture at the site is very low.

5.2 Seismic Ground Shaking Effects

Strong ground shaking at the project site is anticipated during a moderate to severe earthquake occurring anywhere in the Bay Area. Strong ground shaking can cause direct damage to structures; and has the potential of inducing other phenomena that can cause indirect damage to structures. These phenomena include soil liquefaction, dynamic densification of dry soils, lateral spreading, and ground cracking, seismically induced waves, such as tsunamis and seiches, inundation due to dam or embankment failure, and landsliding.

Detailed discussions of liquefaction, dynamic densification and lateral spreading with respect to the site are presented in the subsequent paragraphs of this section. Discussions of landsliding, both static and seismically induced, are presented in Section 5.3, and discussions of tsunami, seiche, and flooding due to dam or embankment failure are presented in Section. 5.5.

5.2.1 Liquefaction and Dynamic Densification

Soil liquefaction is a phenomenon primarily associated with saturated cohesionless soil layers. These soils can dramatically lose strength due to increased pore water pressure during cyclic loading, such as imposed by earthquakes. During the loss of strength, the soils acquire mobility sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated sands that lie close to the ground surface; although, liquefaction can also occur in fine-grained soils, such as low-plasticity silts. In addition, dynamic densification may occur within loose to medium dense, dry sand layers located above groundwater level.

According to Witter et al. (2006) and the Association of Bay Area Governments (ABAG) Resilience Program Liquefaction Susceptibility Map, the site (as shown on **Plate 12**) is located in an area that has been characterized as having a very high liquefaction susceptibility. The Seismic Hazard Zones Map of the Oakland West Quadrangle (CGS, 2003) indicates the site is located within a liquefaction seismic hazard zone (as shown on **Plate 6**), as designated by the State of California.

Our site liquefaction evaluations, which included liquefaction history review and liquefaction triggering and post-liquefaction deformation analyses, are presented in the following sections. In addition, potential for dry sand dynamic densification was also evaluated.

5.2.1.1 Historical Liquefaction in Site Region

According to the seismic hazard zone report of Oakland West Quadrangle (CGS, 2003), several historical liquefaction events had been documented from past earthquakes. Youd and Hoose (1978) compiled observed ground failures caused by earthquake shaking in northern California, including the 1906 San Francisco and 1868 Hayward earthquakes. Following the 1906 earthquake, a 24-inch steel pipe crossing 12th Street at Lake Merritt dam (Site 175 as indicated on **Plate 13**) was reportedly snapped from the settling of the flood gate. The foundation of Lake Merritt dam was also reported as “cracked and broken”. Along the west shore of Lake Merritt, the bank had been cracked and broken, and caved off into the lake.

In addition, liquefaction related ground failures caused by earthquake shaking occurred during the 1989 Loma Prieta earthquake throughout the San Francisco Bay Area and are summarized by Tinsley et al. (1998) . Ground settlement and several sand boils (Site 43) were observed along Lake Merritt Channel Park and Peralta Park, adjacent to the Laney College campus. The ground settlement resulted in the rupture of 6-, 12-, and 36-inch diameter main pipelines. Lateral spreading apparently occurred on the western bank of Lake Merritt during the 1906 event, but this bank was not distressed during the 1989 earthquake.

It is also our understanding damage to the original Laney College swimming pool, located to the north of Building E, was reported after the 1989 earthquake (Kaldveer, 1991), probably as the result of soil liquefaction. A replacement swimming pool was constructed in mid-1990s.

5.2.1.2 Liquefaction Evaluation Methodology

We performed both CPT- and SPT-based liquefaction triggering and post-liquefaction deformation analyses for the site generally in accordance with the guidelines listed in the CGS Special Publication 117A (2008) and the recommended procedures by Southern California Earthquake Center (SCEC, 1999).

Our analyses were based on a peak ground acceleration from a Maximum Considered Earthquake (MCE) event. A geometric mean MCE peak ground acceleration (PGA_M) of 0.81g (adjusted for a Site Class E soil condition) with a mean earthquake magnitude of Mw 7.0 and a modal magnitude of Mw 7.5 were determined for the site per ASCE 7-16 and seismic hazard deaggregation (USGS 2014 model). Our recommended project design groundwater level, at Elevation of +8 feet, was used in the analyses to assess its impacts on liquefaction and liquefaction induced ground surface damage potential.

For comparison and sensitivity evaluation purposes, both methodologies described by NCEER (2001) and by Boulanger and Idriss (BI, 2014) were used for CPT-based analyses. Post-liquefaction deformations were calculated for all layers by using Ishihara and Yoshimine procedures (1992) for NCEER method and EERI Monograph 12 procedures (Idriss & Boulanger, 2008) for BI 2014 Method. Sensitivity analysis was performed with changing earthquake Magnitude to Mw 7.6. Sensitivity analyses show the estimated settlements are not sensitive to earthquake Magnitude as the volumetric strain models saturate at the already low Factors of Safety estimated in this study.

The SPT-based analyses generally followed the methodology described in the EERI Monograph 12 (MNO-12, Idriss & Boulanger, 2008). Per CGS Note 48 requirements, post-liquefaction deformations were calculated for soil layers that have a factor of safety against liquefaction less than 1.3.

5.2.1.3 Liquefaction Evaluation Results and Conclusions

Our results from both CPT- and SPT-based analyses generally indicate that the saturated, loose to medium dense sand layers of various thicknesses located both above and within the Young Bay Mud layer have a high potential for liquefying when they are subjected to an MCE earthquake event. The majority of these sand layers were encountered in borings and CPTs at the site within depths of about 30 to 40 feet (above Elevation of about -15 feet). The extent of the potentially liquefiable soils, factors of safety against liquefaction triggering, and calculated liquefaction-induced cumulative ground settlements at each boring and CPT location are presented in **Supplement D**.

We calculated that the MCE earthquake-induced liquefaction in these sand layers would result in residual volumetric strains varying from about 1 to 4 percent and total ground surface settlements (without reduction associated with the depth of occurrence) ranging from as little as 1 inch to up to about 6-1/2 inches. The table below summarizes the calculated liquefaction-induced settlement using the three different methods referenced above for the site boring and CPT locations. It should be noted the actual ground settlements may differ from our estimates due to uncertainties in the current liquefaction triggering and settlement analysis methodology. In addition, it is a generally accepted idea that the contribution of liquefiable soil layers to surface settlement diminishes as the depths of the layers increase.

Table 5.1: CPT- and SPT-Based Liquefaction Analysis Results

Location	Liquefiable Soil Elevation (ft)	Calculated Cumulative Ground Settlement (inches)		
		MNO-12 SPT Method	NCEER 2001 CPT Method	BI 2014 CPT Method
2019-CPT-01	+8 to +1.5 -5 to -7.5	-	3-1/4	3-1/2
2019-CPT-02	+7 to -2.5 -26.5 to -31	-	2-1/2	3
2019-CPT-03	+7 to +3.5 +2 to -14 -37.5 to -39	-	5	6-1/2
2020-CPT-04	+8 to +6 -9 to -13	-	1-1/2	1-3/4
2020-CPT-05	+7 to +5 -12.5 to -14.5 -17 to -19 -24 to -29	-	2-1/4	2-3/4
2020-CPT-06	+8 to +4.5 -0.5 to -3 -38 to -40 -43 to -45.5	-	2	2-3/4
2020-CPT-07	+8 to +7 -33 to -35 -38 to -40.5	-	1	1
2020-CPT-08	+3.5 to 2 -12.5 to -16 -27 to -31	-	2	2-1/4
2002-CPT-2	+7 to +6 -9.5 to -12	-	1	1-1/2
2020-B-01	+7 to +0.5 -13 to -18.5 -31 to -34	3-1/2	-	-
2002-EB-1	-12 to -17 -22 to -27	3-1/4	-	-
2002-EB-2	-	0	-	-
2002-EB-3	-10 to -16 -28 to -33	2-1/2	-	-

Based on our review of available maps and literature, and the results of our site evaluations, it is our opinion, when the site is subjected to a Maximum Considered Earthquake (MCE) event, the likelihood of liquefaction occurring at the site is high.

5.2.2 Dynamic Densification Evaluations

We performed both CPT-based and SPT-based dynamic densification evaluations based on procedures developed by Tokimatsu and Seed (1987) and Robertson and Shao (2010). A geometric mean MCE peak ground acceleration (PGA_M) of 0.81g, a mean earthquake magnitude of 7.0, and the project design groundwater level at Elevation of +8 feet were used in our analyses. The potential dynamic densification settlements of the near-surface unsaturated sandy fills of about 8 to 13 feet in thickness at the site are estimated to be on the order of 1/4 to 1/2 inch after the MCE event. The detailed results of each boring and CPT location are presented in **Supplement E**. It is our opinion that the potential for soil dynamic densification to impact the site is low.

5.2.3 Lateral Spreading

Lateral spreading occurs when soils liquefy during an earthquake event and the liquefied soils, along with the overlying soils, move laterally toward a free face or unconfined space, such as the west bank of the Lake Merritt Channel. Lateral spreading can result in significant horizontal ground displacements.

Our site lateral spreading evaluations generally followed methodology described in the EERI Monograph 12 (MNO-12, Idriss and Boulanger, 2008) to estimate the maximum shear strain of each liquefiable soil layer and calculate the Lateral Displacement Index (LDI) (Zhang et al., 2004) at each CPT and boring location. The detailed results are included in **Supplement D**.

In addition, empirical correlations developed by Youd et al. (2002) were also used to identify the potential soil layers that are prone to trigger ground lateral spreading and to provide estimates for possible ground lateral displacement. According to Youd et al. (2002), saturated cohesionless soil sediments with SPT $N_{1,60}$-value equal or more than 15 are considered as not likely to have significant displacement during earthquakes smaller than magnitude 8. Our calculated LDIs and order of ground lateral displacements (from soil layers having $N_{1,60}$-value less than 15) at the site CPT and boring locations are summarized in the table below. It should be noted these values should be considered as an index due to the limitations of the current engineering knowledge and analysis methodology. The Table 5.2 lateral displacement values are for Mw 7.0 earthquake.

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Table 5.2: CPT- and SPT-Based LDI and Lateral Displacement Analysis Results

Location	Liquefiable Soil Elevation (ft)	Calculated Lateral Displacement Index - LDI (inches)	Potential Lateral Spreading Triggering Soil Elevation (ft)	Estimated Ground Lateral Displacement (inches)
2019-CPT-01	+8 to +1.5 -5 to -7.5	42	-	0
2019-CPT-02	+7 to -2.5 -26.5 to -31	30 to 33	+7 to -2.5	12 to 24
2019-CPT-03	+7 to +3.5 +2 to -14 -37.5 to -39	56 to 59	+7 to -8	18 to 36
2020-CPT-04	+8 to +6 -9 to -13	18	-	0
2020-CPT-05	+7 to +5 -12.5 to -14.5 -17 to -19 -24 to -29	22 to 24	+7 to -2	12 to 24
2020-CPT-06	+8 to +4.5 -0.5 to -3 -38 to -40 -43 to -45.5	25 to 27	+8 to -5	12 to 30
2020-CPT-07	+8 to +7 -33 to -35 -38 to -40.5	10	+8 to -5	12 to 24
2020-CPT-08	+3.5 to 2 -12.5 to -16 -27 to -31	22	-	0
2002-CPT-2	+7 to +6 -9.5 to -12	12	-	0
2020-B-01	+7 to +0.5 -13 to -18.5 -31 to -34	23	+7 to +3.5	6 to 18
2002-EB-1	-12 to -17 -22 to -27	30	-	0
2002-EB-2	-	0	-	0
2002-EB-3	-10 to -16 -28 to -33	23	-	0

Our results generally indicate the loose to medium dense sand layers encountered by CPTs and borings in the area adjacent to the Lake Merritt Channel at Elevations between +8 and -5 feet have a high potential to trigger ground surface lateral spreading during soil liquefaction from an

MCE event. The other onsite liquefiable sand layers are considered as having low potential to trigger ground lateral spreading due to their presence in isolated thin pockets and/or being located at deeper depths in relation to the bottom of the Lake Merritt Channel. Our estimated lateral extent of potential ground lateral spreading/slope instability is shown on **Plate 3.**

5.3 Slope Stability Analysis

The project proposed building is located about 130 to 160 feet away from the edge of the west bank of the channel. Our evaluations are only meant to assess the global stability of the proposed development and the potential lateral extents of ground failures caused by the possible lateral spreading of the channel bank during an MCE event (if it occurs). Detailed stability evaluation of the existing channel west bank is beyond our scope of work, since soil stratigraphy below the bank and channel were extrapolated from data developed for the project area.

The global site slope stability was evaluated using a two-dimensional, limit equilibrium computer program, SLOPE/W (GeoStudio 2016, Ver. 8.16.1.13452), and Spencer analysis method. The recommended analysis procedures by South California Earthquake Center (SCEC, 2002) were generally followed. The representative Cross-Sections A-A', D-D' and E-E' (**Plates 7, 10** and **11**) were used in our analyses to evaluate the following four (4) design loading cases:

- Case 1: Long Term (Static)
- Case 2: Seismic Event Yield Acceleration (Pseudo-static)
- Case 3: Seismic Event k = 0.15g (Pseudo-static); Fixed Slip Surface at Edge of Building
- Case 4: Post-Liquefaction (Static)

Factors of safety against slope stability failures were calculated for the Cases 1, 3, and 4. Pseudo-static yield acceleration (ky to achieve a factor of safety equals 1.0) was calculated for Case 2.

5.3.1 Subsurface Soil Engineering Properties

Soil engineering properties were developed based on the field exploration and laboratory testing results by Fugro and others, and typical engineering correlations. The table below summarizes the soil properties used in our analyses.

Table 5.3: Soil Engineering Properties Used in Site Slope Stability Analyses

Material	Unit Weight (pcf)	Material Shear Strength	
		Cohesion c′ (psf)	Friction Angle Φ′ (degree)
Sandy Fill	120	0	35
Young Bay Mud with Sand Lenses	90	0.35 x Effective Overburden Stress (psf)	0
Interbedded Clays and Sands	130	0	40
Highly Liquefiable Sands	110	0	33
Post-Liquefaction Sands (Residual Strength)	110	100 + 20 x Depth (ft)	-

5.3.2 Slope Stability Analysis Results and Conclusions

The results of our slope stability analyses are presented in the table below. Our interpreted cross-section stratigraphic profiles, soil engineering properties used in the analyses, and the detailed results of the analyses are presented on the computer program printouts in the attached **Supplement F**.

Table 5.4: Slope Stability Analysis Results

Cross-Section	Case 1 Long Term	Case 2 Seismic Event Yield Acceleration	Case 3 Seismic Event k = 0.15g; Fixed Slip Surface at Edge of Building	Case 4 Post-Liquefaction
	Factor of Safety	ky	Factor of Safety	Factor of Safety
A-A′	2.8	0.12	0.9	2.6
D-D′	2.2	0.12	0.9	2.0
E-E′	1.7	0.11	0.9	1.5

The results of our slope stability analyses generally indicate that the factors of safety against slope failures for the Case 1 (Long Term, Static) are 2.8, 2.2, and 1.7, respectively, for Sections A-A′, D-D′ and E-E′, which exceed the generally accepted minimum allowable value of 1.5 for long term conditions.

For the Case 2 (Seismic Event Yield Acceleration, Pseudo-static), the yield accelerations (ky) are determined to be 0.12g, 0.12g, and 0.11g, respectively, for Sections A-A′, D-D′, and E-E′. Using the Bray (1998) procedure as recommended by the SCEC publication (2002), we calculated slope displacements on the order of about 15 to 24 inches (38 to 61 centimeters) may occur during an MCE event (with a maximum horizontal acceleration of 0.81g from a mode magnitude 7.5

causative earthquake located at 6.8 kilometers from the site). These calculated displacements exceed the threshold of 6 inches (15 cm) defined by the SCEC publication (2002), which likely distinguishes conditions in which small to moderate displacements are likely from conditions in which large displacements are likely. However, as indicated on the result printouts in **Supplement F**, the most critical slip surfaces along these cross-sections do not daylight within the proposed building location.

In addition, by fixing the slip surface daylight location at the edge of the proposed building location, factors of safety against slope failures for the Case 3 (Seismic Event k = 0.15g, Pseudo-Static) are all 0.9 for Sections A-A', D-D' and E-E', which also fail to meet the commonly accepted minimum value of 1.15 for seismic performance (Seed, 1979)[34]. It should also be noted, due to the low undrained shear strength of Young Bay Mud used in the Case 2 and Case 3 analyses (pseudo-static), the calculated low factors of safety and the estimated large and deep slip surfaces (35 to 45 feet deep below the top of channel bank) may not fully represent the seismic global slope stability at the proposed building location (which is about 130 to 160 feet away from the edge of the channel bank). Seismic slope stability of site is most likely governed by the extent of possible ground lateral spreading during major liquefaction events.

In Case 4 (Post-Liquefaction, Static), post-liquefaction residual shear strength was used for the highly liquefiable sands. The factors of safety against slope failures are 2.6, 2.0, and 1.5, respectively, for Sections A-A', D-D' and E-E', which exceed the generally accepted minimum value of 1.3 for short term conditions after major liquefaction events.

Due to the high degree of uncertainties on site subsurface conditions, seismic characteristics of the triggering earthquake, and analysis methodology, the results of our seismic slope stability and lateral spreading analyses should be considered as an index of site performance during major earthquake events. It is our opinion that the potential for slope instability during an MCE event and/or after major liquefaction event to impact the proposed building location is low to moderate. However, extensive slope failures may occur for the areas immediately adjacent to the Lake Merritt Channel if soil liquefaction and ground lateral spreading do occur at the site region during major earthquake events. Our estimated lateral extent of potential ground lateral spreading/slope instability is shown on **Plate 3.**

5.4 Compressible Soils

The site is blanketed by historical sandy or clayey fills that extend to depths of about 8 to 25 feet (Elevation of about +8 feet to -5 feet). Most of these fills appear to be derived from the historical filling of the natural Lake Merritt outlet channel between 1860s and 1940s, and the later development of the Laney College campus in 1960s. These fills are heterogenous and locally

[34] Seed, 1979. *Considerations in the Earthquake-Resistant Design of Earth and Rockfill Dams*, Geotechnique, V. 29 (3), p. 215-263.

contain various amounts of concrete, brick, and wood debris. These historical fills were most likely not compacted to the current acceptable geotechnical engineering standards and are potentially compressible. In addition, we estimated minor cut and fill grading of up to about 2 to 3 feet had been performed at the site during the portable classroom development in 2007 or 2008. The actual details of the previous grading are unknown.

Below the surficial fill layer, Young Bay Mud was encountered to about 30 feet deep (Elevation of about -10 feet) at the northwest side of the proposed building location and about 50 feet deep (Elevation of about -30 feet) at the southeast side of the proposed building location. This 15- to 35-foot-thick layer of slightly over-consolidated to normally consolidated Young Bay Mud is very soft to soft, has a high moisture content and a low shear strength, and is highly compressible. Under additional new loads, such as weights of the new fills and structures, the Young Bay Mud will consolidate while the induced excess pore water pressures are dissipating, which may cause detrimental total and differential settlements to the imposing structures and improvements.

We estimate the primary consolidation settlement due to the historical fills placed prior to 1960s at the site should have been completed. Additional settlements from the recent fill placement during the portable classroom development in 2007 or 2008 may be still ongoing. We recommend any available previous grading and construction records be forwarded to us for further review.

No significant raising of the existing site grade is anticipated for the project. If new fills will be placed to raise the existing grade, we anticipate that additional settlement will occur in the future. Our analyses indicate that for every foot of new fills that will be placed, it would induce an additional ultimate settlement of about 2 to 3 inches over the next 10 to 30 years. This additional settlement will also likely affect the integrity of the existing and/or new utility lines. In addition, this settlement will also cause downdrag forces to the pile-supported structure.

5.5 Other Geologic Hazards

Below we briefly review other geologic hazards identified by the CGS (2019) Note 48 as exceptional geologic hazards or adverse site conditions that do not occur statewide. This section addresses expansive soils, corrosive soils, volcanic eruptions, flooding and dam inundation, tsunami and seiche, naturally occurring asbestos, and hydrocompaction.

5.5.1 Expansive Soils

The near-surface soils encountered at the site were predominately man-made fills that consist of silty sands and lean clays. The expansion potential of the near-surface soils at this site is considered low to moderate. The potential expansive soil hazard can be further reduced provided our recommendations in the report are followed.

5.5.2 Corrosive Soils

Corrosivity tests, that include redox, pH, chlorides, sulfates, and resistivity were performed by CERCO Analytical, Inc. on two representative onsite near-surface soil samples (from Boring 2019-CPT-01 at about 2-1/2 feet and 2019-CPT-03 at about 4 feet). The test results and a brief evaluation report prepared by CERCO regarding the onsite soil corrosivity are also included in **Supplement B**. According to the evaluation report, the onsite near-surface soils should be considered as "moderately" and "slightly" corrosive based on resistivity and redox potentials measurements, respectively.

5.5.3 Volcanic Eruption

The hazards of volcanic eruption include impact and inundation by lava flows, volcanic mudflows, or pyroclastic flows, and the effects of airborne volcanic ash and gases. No active volcanoes occur in the San Francisco Bay area. The nearest active volcano is the Clear Lake Volcanic Field, located about 90 miles north of the site. Volcanic flows would not extend far enough to affect the site. Airfall ash, which is known to travel great distances, would likely travel eastward based on prevailing winds. Potential hazards associated with volcanic activity in the site region are estimated to be very low (Miller, 1989).

5.5.4 Flooding and Dam Inundation

In this section we provide a brief discussion of flooding and dam inundation hazard based on a review of readily available information. A detailed risk evaluation of flooding and inundation at the site was not performed because the initial screening evaluation did not identify any significant flooding or inundation hazards.

According to the FEMA (2018) flood insurance rate map for Oakland, the project building area is located outside a 100-year flood zone. The site is adjacent to the Lake Merritt Channel, which serves as the outlet for Lake Merritt and whose level is controlled by tide level in the Oakland Inner Harbor and the level of Lake Merritt, which is partially tidal. Tide gates at the 7th Street bridge regulate the water flows into and out of Lake Merritt. The elevation of the site is 18 to 21 ft (NAVD88) and the highest astronomical tide (HAT) in the Oakland Inner Harbor is about 8 feet (NAVD88) (https://tidesandcurrents.noaa.gov/datums.html?id=9414764), at least 10 feet lower than the site. Runoff into Lake Merritt from heavy storms could raise the water level in the tidal channel if lake waters were released through the tide gates, but this is not likely to exceed the HAT.

The City of Oakland notes that there are 13 active dams, reservoirs, and clearwells that, in case of failure, would cause flooding in Oakland. These facilities include:

- Central, Claremont, Dingee, Dunsmuir, Estates and 39th Avenue reservoirs, the dams at Lake Chabot and at Upper San Leandro reservoir, and the Upper San Leandro filtration plant no. 1 and no. 2 clearwells (owned by the East Bay Municipal Utility District, EBMUD);
- Lake Temescal dam (owned by the East Bay Regional Park District);
- Lower Edwards and Upper Edwards reservoirs (owned by the Mountain View Cemetery Association); and
- Lower and Upper Edwards reservoirs, owned by the Mountain View Cemetery Association.

However, according to Figure 6.1 of the City of Oakland General Plan Safety Element (2004), the site is not located within any of the dam failure inundation areas of any of these above facilities. Based on this information, the potential for flooding or inundation of the project site by dam failure is judged to be very low.

5.5.5 Tsunami and Seiche

During a major earthquake, strong waves such as tsunamis or seiches may be generated in large bodies of water and may cause damage to structures at or near the shoreline. Tsunamis are large waves generated by displacement of the seafloor by earthquakes, coastal or submarine landslides, or volcanoes. Damaging tsunamis are a potential hazard along the California coast. Most historical California tsunamis were associated with distant earthquakes (such as those in Alaska or Pacific Ocean), not with local earthquakes. However, they may occur, especially along the far northern coast of California where seafloor displacement is associated with major subduction zone earthquakes. Devastating tsunamis have not occurred in historic times in the San Francisco Bay Area.

The existing surface elevations at the project building area are about +18 feet to +21 feet (NAVD 88) and the site is located about 1/4 mile from the Oakland Inner Harbor, bounded by the Alameda Island and the Oakland bay shore. According to the Tsunami Inundation Map for Emergency Planning of the Oakland West Quadrangle (CGS, 2009), the project building area is located adjacent to but outside the mapped boundary of an identified potential tsunami inundation area. It appears the mapped boundary lies approximately at Elevation of +15 feet. In our opinion, the potential inundation hazard by a tsunami at the project building area is low.

A seiche is a wave that occurs in an enclosed basin as a result of displacement in the basin bottom, large landslides into the basin, or periodic oscillation or sloshing of the water in the basin. According to City of Oakland General Plan Safety Element (2004), the nearby by Lake Merritt, with depths greater than 2 to 3 feet only near its center, is likely too shallow to be able to generate devasting seiches. In our opinion the potential for damage due to a seiche is negligible.

5.5.6 SupplementNaturally Occurring Asbestos (NOA)

Inhalation of asbestos fibers may cause cancer. Most commonly, asbestos occurrences are associated with serpentinite and partially serpentinized ultramafic rocks.

Asbestos occurs naturally in certain geologic settings in California. Exposure and disturbance of rock and soil that contains asbestos can result in the release of fibers to the air and consequent exposure to the public. Asbestos most commonly occurs in ultramafic rock that has undergone partial or complete alteration to serpentine rock (proper rock name serpentinite) and often contains chrysotile asbestos. In addition, tremolite, another form of asbestos, can be found associated with ultramafic rock, particularly near faults. Sources of asbestos emissions include:

- Unpaved roads or driveways surfaced with ultramafic rock,
- Construction activities in ultramafic rock deposits or soils, or
- Rock quarrying activities where ultramafic rock is present.

The bedrock underlying the site is estimated to be on the order of 500 feet below the surface. In addition, no serpentinite gravels were reportedly encountered in the previous borings at the site, and no serpentinite outcrops or serpentine derived soils are identified in the hills that drain into Lake Merritt. Therefore, we consider the possibility of NOA at the site to be very low.

5.5.7 Hydrocompaction

Hydrocompaction; also referred to as hydro-collapse, is a process of settlement and resulting volume change that occurs in, low density, fine sand with minor amounts of silt and clay. Near-surface soils above groundwater encountered at the site predominately consist of medium dense silty sands and gravels or medium stiff clays; therefore, the potential for hydrocompaction or hydrocollapse is very low.

6. Discussion and Conclusions

It is our opinion that the project is feasible from a geotechnical and engineering geologic standpoint, provided that the conclusions and recommendations presented in this report are incorporated into the project design and specifications. The principal geotechnical considerations are discussed in the following sections.

6.1 Seismic and Geologic Hazards

The site is in a seismically active region of California. Significant earthquakes in the San Francisco Bay Area have been associated with movements within the fault zones. Earthquakes occurring along faults in the area have the potential to produce strong ground shaking at the site. Structures within the San Francisco Bay Area will experience similar shaking effects during a moderate to strong earthquake. Details discussions regarding the site geologic hazards are presented in **Section 5.0**.

Based on the results of our review and evaluation, geologic hazards at the project site consist of the potential for strong ground shaking, liquefaction, lateral spreading, landsliding, compressible fills and soils, corrosive soils, and expansive soils. Detailed measures to mitigate these geologic hazards are incorporated in our recommendations presented in **Section 7.0**.

However, the potential for surface fault offset, dynamic densification, seismically induced waves, flooding, dam inundation, hydrocompaction, NOA, and volcanic eruption at the project building area appear to be low to negligible.

6.2 Liquefaction, Lateral Spreading, and Slope Instability

As described previously, the results of our site liquefaction evaluations generally indicate the saturated, loose to medium dense sand layers of various thicknesses located both above and within the Young Bay Mud layer have a high potential for liquefying when they are subjected to an MCE earthquake event. The majority of these sand layers were encountered by borings and CPTs at the site within depths of about 30 to 40 feet (above Elevation of about -15 feet). We calculated that the MCE induced liquefaction in these sand layers would result in residual volumetric strains varying from about 1 to 4 percent and total ground surface settlements (without reduction associated with the depth of occurrence) ranging from as little as 1 inch to up to about 6-1/2 inches.

Our lateral spreading analysis results generally indicate the loose to medium dense sand layers encountered by CPTs and borings in the area adjacent to the Lake Merritt Channel at Elevations between +8 and -5 feet have a high potential to trigger ground surface lateral spreading during soil liquefaction from an MCE event. The other onsite liquefiable sand layers are considered as having low potential to trigger ground lateral spreading due to their presence in isolated thin pockets and/or being located in deeper depths in relation to the bottom of the Lake Merritt Channel.

In addition, it is our opinion the potential for slope instability during an MCE event and/or after major liquefaction event to impact the proposed building location is low to moderate. However, extensive slope failures may occur for the areas immediately adjacent to the Lake Merritt Channel if soil liquefaction and lateral spreading do occur at the site region during major earthquakes.

We recommend the proposed new building be supported on a deep foundation system that provides proper bearing support during the potential soil liquefaction events. The deep foundation should be designed to resist downdrag loads that would be imposed upon the foundations due to soil liquefaction.

In addition, the southeast side of the proposed new building foundation should also include a permanent shoring system, or a ground improvement technique should be used to mitigate the detrimental impacts from the potential lateral spreading and slope instability from the areas immediately adjacent to the Lake Merritt Channel. Our estimated lateral extent of potential ground lateral spreading/slope instability is shown on **Plate 3.**

Based on the proposed building layout, we recommend the permanent shoring system along the southeast side of the proposed building (estimated lateral spreading/slope instability lateral extent) be designed to retain a 12-foot-high column of soils, assuming the loss of adjacent ground support due to slope failure. The small portion of the shoring system located further east of the area of estimated lateral spreading/slope instability should be designed to retain an 18-feet high column of soils. Our recommended lateral pressures for the shoring system designs are shown on **Plates 14** and **15**. Recommendations and specifications for ground improvement technique are presented in **Supplement I and J**.

The site and any new improvements not supported on deep foundations may experience total areal ground surface settlements on the order of about 1 to 4 inches with locally up to about 6-1/2 inches of settlement. In the area immediately adjacent to the channel bank, the ground settlements may be larger than the above estimates if lateral spreading occurs. Underground pipelines (gas lines, sanitary sewers, water services, etc.) should be properly designed to accommodate for the settlement caused by the liquefaction of the underlying supporting soils. Consideration should be given to using flexible pipe connections to mitigate potential damage from the estimated potential liquefaction-induced settlement of 4 inches at locations where the pipes are connected to pile-supported structures.

It should be noted that after a major liquefaction event, phenomena such as sand boils, ground cracking, and differential movement of overlying improvements such as roadways and utilities may be observed and may require repair.

Alternatively, soil liquefaction ground improvement options that involve densification, drainage, reinforcement, mixing, or replacement of the liquefiable soils can be used to mitigate the site liquefaction, lateral spreading, and slope instability potentials. If needed, we can provide additional recommendations during project design, once the building and development layouts are finalized.

6.3 Compressible Soils

As described previously, the site is blanketed by sandy or clayey fills that extended to depths of about 8 to 25 feet (Elevation of about +8 feet to -5 feet). Below the surficial fill layer, Young Bay Mud was encountered to about 30 feet deep (Elevation of about -10 feet) at the northwest side of the proposed building location and about 50 feet deep (Elevation of about -30 feet) at the southeast side of the proposed building location. This 15- to 35-foot layer of slightly over-consolidated to normally consolidated Young Bay Mud is very soft to soft, has high moisture content and low shear strength, and is highly compressible under new additional loads. Besides the areas of the recent fills placed during the portable classroom development in 2007 or 2008, we estimated the site primary consolidation settlement due to the historical fills placed prior to 1960s should have been completed.

In our 2020 report, we recommend the proposed new building be supported on a deep foundation system that extends to a depth of at least 70 feet (or to a pile tip Elevation of -50 feet) to transfer bearing loads to the sand and clay layers below the Young Bay Mud layer. Either precast pre-stressed concrete driven piles or drilled piles, such as Case-in-Drilled-Hole (CIDH) piers and auger cast piles, can be used at the site. We note that 70- to 110-foot long, 14-inch square, precast, pre-stressed concrete driven piles were used to support the existing Art Building (built in 2005) that is also located adjacent to the Lake Merritt Channel and is about 500 feet northeast of the proposed Library Learning Resource Center site. Furthermore, the new Building Efficiency for a Sustainable Tomorrow (BEST) Center built in 2016 also is reportedly supported by 95- to 105-foot long, 14-inch square, precast, pre-stressed concrete driven piles.

The design team has decided to use a DMM ground improvement technique in combination with a shallow foundation for the LLRC building. The details of the design and specifications are presented in **Supplement I and J**.

In addition, to reduce the soil consolidation-induced downdrag forces on the pile foundations, we recommend the proposed project site grading activities, construction of the new surface improvements (such as exterior flatwork), and backfill for deeply buried pipelines (if any) be designed so “zero net load” will be imposed on the underlying Young Bay Mud. A “zero net load" condition can be achieved by over-excavating the fills (and possibly a portion of the Young Bay Mud if necessary) and backfilling the excavation with lightweight fill materials. Lightweight fills or concrete materials should also be used to backfill deep pipe trenches. The weight combination of new fills, at-grade new improvements, and new lightweight fills and/or concrete materials should not exceed the weight of the soils removed.

Our recommended unit weights of the fills and Young Bay Mud to be used in the "zero net load" analyses are shown in the table below. The site grade prior to the portable classroom development in 2007 or 2008 should be used in the analyses as the base line. We also recommend a groundwater level at Elevation of +8 feet be used in the analysis.

Table 6.1: Recommend Fill and Young Bay Mud Unit Weight

Soil Unit	Elevation	Unit Weight (pcf)
Existing Fill and Soil Above Groundwater	Above +8 Feet	110
Young Bay Mud Below Groundwater	Below +8 Feet	30

Alternatively, lightweight concrete materials such as Elastizell and Geofoam can be used as lightweight fills. We note that with the use of these lightweight materials below the ground water level would likely require dewatering of the excavation until sufficient weight from fills and/or structure loads are imposed to prevent potential uplift water pressures from lifting the lightweight fill materials.

6.4 Deep Mixing Method (DMM)

The design team decided to use shallow foundation and ground improvement technique in lieu of deep foundation and retaining wall system to create a more competent bearing layer for the shallow foundation and reduce the ground displacements due to lateral spreading. DMM ground improvement is one of the many techniques that is an in-situ soil treatment in which native soils or fills are mixed and blended with cement or other binders and water. The final mixed soil-binder product has enhanced engineering properties such as increased strength, lower permeability, and reduced compressibility. Two types of DMMs are used in the United States: wet mixing and dry mixing. Wet mixing involves injecting binders in slurry (wet) form to blend with the soil. Primarily single-auger, multi-auger, or cutter-based mixing processes are used with cement-based slurries to create isolated elements, continuous walls or blocks for large-scale foundation improvement, earth retaining systems, hydraulic barriers, and contaminant/fixation systems. Dry mixing uses binders in powder (dry) form that react with the water already present in the soil. Primarily single-auger dry mixing processes are used with lime and lime-cement mixtures to create isolated columns, panels, or blocks for soil stabilization as well as reinforcement of cohesive soils.

Soils best suited to DMM include cohesive soils with high moisture contents and loose, saturated, fine granular soils. DMM has also been used successfully in a wide range of less cohesive soils and fills, but it is typically not feasible in very dense or stiff materials or in ground with obstructions such as cobbles or boulders. The treated soil properties obtained by DMM reflect the characteristics of the native soil, binder characteristics, construction variables, operational parameters, curing time, and loading conditions. The generic term DMM is inclusive of other terms such as deep soil mixing (DSM) and cement deep soil mixing (CDSM). A detailed design and recommendations are presented in DMM Design Recommendations, **Supplement I and J.**

6.5 Preliminary Corrosion Evaluation

Corrosivity tests that include redox, pH, chlorides, sulfates, and resistivity were performed by CERCO Analytical, Inc. in Concord, California, on two representative onsite near-surface soil samples (from Boring 2019-CPT-01 at about 2-1/2 feet and 2019-CPT-03 at about 4 feet). The test results and a brief evaluation report prepared by CERCO regarding the onsite soil corrosivity are also included in **Supplement B**. We recommend these test results and the report be forwarded to the project underground contractors, pipeline designers, and foundation designers and contractors, so that they can design and install corrosion protection measures for buried concrete structures and ferrous metal. We also recommend additional testing be performed if the test results in **Supplement B** are deemed insufficient by the designers of the corrosion protection.

6.6 Construction Considerations

Excavations will be required to construct building foundations and elevator pit (if any), install utilities, and to remove locally weak or unsuitable soils. All excavations that will be deeper than 5 feet and will be entered by workers should be shored or sloped for safety in accordance with Occupational Safety and Health Administration (OSHA) standards.

If earthwork is performed during the dry season, moisture conditioning will be required to raise the onsite soil moisture contents to the engineered fill placement and compaction recommendation presented in this report. If earthwork is performed during or shortly after wet weather conditions, the moisture content of the soils could be appreciably above optimum. Consequently, subgrade preparation and fill placement may be difficult. Additional recommendations for wet weather construction can be provided at the time of construction, if required.

7. Recommendations

7.1 Seismic Design

The proposed new building should be designed to resist the lateral forces generated by earthquake shaking in accordance with Chapter 16 of the 2019 California Building Code (CBC). This section presents seismic design criteria according to 2019 CBC, which has adopted the seismic hazard assessment procedures provided by ASCE 7-16, Minimum Design Loads for Buildings and Other Structures. Per Section 11.6 of ASCE 7-16, structures of Risk Category I, II, and III (defined in ASCE 7-16 Table 1.5-1) should be designed according to Seismic Design Category "D".

Our liquefaction triggering hazard assessment indicated that the soils at the site are potentially liquefiable. Therefore, according to ASCE 7-16, the site is classified as Site Class F, and site response analyses, as defined in Section 21.1 of ASCE7-16, are required to calculate the design ground motions at the ground surface. Additionally, due to the large ground motion amplitudes expected at the site, ASCE 7-16 also requires the performance of a site-specific seismic hazard assessment according to Section 21.2 of ASCE 7-16. Detailed discussions of these site-specific ground motion analyses are included in **Supplement G**.

Table 7.1 tabulates the spectral ordinates of the recommended site-specific MCE_R and design response spectra per ASCE 7-16 for the ground surface. The corresponding design acceleration parameters S_{MS}, S_{M1}, S_{DS}, S_{D1}, S_S, and S_1 are tabulated in **Table 7.2**. The MCE_R and design response spectra per ASCE 7-16 at the base of the Young Bay Mud layer is provided in **Supplement G**.

Table 7.1: MCE_R and Design Response Spectra per ASCE 7-16 at the Ground Surface, 5% Damping

Period (sec)	Horizontal Spectral Acceleration (g)	
	Site-Specific MCE_R	Design Response Spectrum
0.01 (PGA)	0.584	0.389
0.03	0.639	0.426
0.05	0.694	0.463
0.075	0.763	0.508
0.1	0.831	0.554
0.15	0.969	0.646
0.2	1.11	0.738
0.25	1.24	0.829
0.3	1.38	0.921
0.304	1.39	0.927
0.4	1.39	0.927
0.5	1.39	0.927
0.75	1.39	0.927
1	1.39	0.927
1.5	1.39	0.927
1.52	1.39	0.927
2	1.06	0.704
3	0.827	0.551
4	0.733	0.489
5	0.561	0.374
7.5	0.282	0.188
8	0.264	0.176
10	0.169	0.113

Table 7.2: Design Acceleration Parameters per ASCE 7-16 at the Ground Surface, 5% Damping

Parameter	Value
S_{MS}	1.39 g
S_{M1}	2.93 g
S_{DS}	0.927 g
S_{D1}	1.96 g
S_s	1.74 g
S_1	0.66 g

7.2 Earthwork

7.2.1 Site Clearing and Preparation

The site should be cleared of all obstructions, including any existing structures and their entire foundation systems, concrete slabs-on grade, existing utilities and pipelines and their associated backfill, designated trees and their associated entire root systems, landscaping, and debris. Concrete/asphalt concrete, baserock, and trench backfill materials can be reused as new fills provided debris is removed and concrete/asphalt concrete are broken up to meet the engineered fill size requirements presented in this report.

Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with engineered fills and compacted to the requirements presented in this report. We recommend backfilling operations for any excavations to remove underground obstructions be performed under observations and testing of the project Geotechnical Engineer. After clearing, areas containing heavy surface vegetation should be stripped to an appropriate depth to remove these materials. We estimate the stripping depth to be about 6 inches. The amount of actual stripping should be determined in the field at the time of construction. Stripped materials should be removed from the site or stockpiled for later use in landscaping, if desired.

7.2.2 Subgrade Preparation

Following the site clearing and preparation, soil subgrades in areas to receive engineered fill, slabs-on-grade, or pavements be scarified to a depth of at least 12 inches, moisture conditioned to approximately 3 percent above optimum water content and compacted to the requirements for engineered fills. Locally weak fills and soils, if encountered, should also be excavated and replaced, or otherwise stabilized as recommended by the project Geotechnical Engineer at the time of earthwork operations.

The prepared subgrade surface should be firm, unyielding, and kept moist during construction. The subgrades should be protected from damage caused by weather and construction traffic. If the subgrades are left exposed to weather for extended periods of time or are disturbed by construction traffic, the project Geotechnical Engineer should be consulted on the need for subgrade moisture reconditioning and/or scarifying and recompacting to eliminate shrinkage cracks and disturbances.

7.2.3 Engineered Fill Materials

Any new fills placed at the site should consist of engineered fills that meet the requirements presented in this report, except for landscaping materials which are placed on level ground. All engineered fills should have an organic content of less than 3 percent by volume and should not contain rocks or lumps larger than 4 inches in greatest dimension with not more than 15 percent larger than 2.5 inches.

Onsite soils (except for Young Bay Mud) and fills can be used as new fills. Imported fills not used as non-expansive fills should be predominantly granular, have a liquid limit less than 40 percent, and have a plasticity index not exceeding 20. Imported, non-expansive fills should consist of sub-angular to angular particles, have a plasticity index not exceeding 12, and have a significant fine content. All imported fills should not contain environmental contaminants or debris and should be non-corrosive.

7.2.4 Fill Placement and Compaction

Within the upper 5 feet of the finished ground surface, we recommend engineered fills be compacted to at least 90 percent relative compaction, as determined by ASTM D1557. Engineered fills below a depth of 5 feet should be compacted to at least 95 percent relative compaction. The upper 6 inches of subgrade soils beneath pavements should be compacted to at least 95 percent relative compaction. Fill material should be spread and compacted in lifts not exceeding approximately 8 inches in uncompacted thickness.

We recommend engineered fills be moisture conditioned to approximately 3 percent above optimum water content. To achieve satisfactory compaction of fill materials, it may be necessary to adjust the water content at the time of earthwork operations. This may require that water be added to soils that are too dry, or that aeration be performed in any soils that are too wet. To achieve satisfactory compaction of onsite excavated soils from near or below the existing groundwater level will require drying at the time of construction.

7.2.5 Trench Backfill and Pipe Bedding

To prevent imposing additional load to the underlying soils and to reduce potential settlement along deeply buried pipelines, trench backfill materials should be properly selected so that the unit weight of backfill materials is less or equivalent to the unit weight of the removed onsite soil materials (zero net load). Considerations should be given to increasing the hydraulic gradient of gravity flow pipes to account for potential soil differential consolidation settlements below the pipes and also using flexible connections for all pipes.

Pipeline trenches should be backfilled with engineered fills placed in lifts of approximately 8 inches or less in uncompacted thickness. Thicker lifts can be used provided the method of compaction is approved by the project Geotechnical Engineer and the required minimum degree of compaction is achieved. Backfill should be placed by mechanical means only; jetting is not permitted. Onsite soils, and onsite and imported fills when used for trench backfill should be compacted to at least 90 percent relative compaction. Imported sands and aggregate bases when used for trench backfill should be compacted to at least 95 percent relative compaction and sufficient water is added during backfilling operations to prevent the soil from "bulking" during compaction. The upper 3 feet of trench backfill in foundation, slab, and pavement areas should be entirely compacted to at least 95 percent relative compaction.

Sand or gravel backfilled trench laterals that extend from irrigated landscaped areas, such as lawns or planting strips, toward pavements, exterior slabs, and building foundations, should be plugged with onsite or imported clayey soils, low strength concrete, or sand-cement slurry mixture below the edges of pavements and exterior slabs, and under perimeters of the foundations. The plugs for the trench laterals should be at least 24 inches thick, extend at least 24 inches beyond the trench walls, and extend from the bottom of the trench to the top of the sand or gravel backfills.

Bedding material should consist of Caltrans Class 2 Aggregate Base or Aggregate Base Course (ABC) meeting the requirements of Section 26 of Caltrans Standard Specifications. All bedding material shall have 3/4-inch maximum aggregate size and be free from organic or vegetable matter, lumps, or balls of silt/clay, or any other deleterious matter. ABC material shall conform to the following gradations when tested in accordance with ASTM C136 or California Test 202.

Table 7.3: Aggregate Base Course Gradation Requirements

Sieve Size (Square Openings)	Percentage by Weight Passing Sieves
1 inch Screen	100
3/4 inch Screen	90 to 100
No. 4 Sieve	35 to 60
No. 30 Sieve	10 to 30
No. 200 Sieve	2 to 9

In addition to the above requirements, all material used shall conform to the following quality requirements:

- Resistance (R-Value) with the minimum test results of 78;
- Sand Equivalent with the minimum test result of 22; and
- Durability Index with the minimum test result of 35.

7.2.6 Exterior Flatwork

We recommend exterior slabs, such as sidewalks and patios, be placed directly on the properly prepared subgrades in accordance with the recommendations presented in this report. Eliminating aggregate base, gravel, or crushed rock base beneath exterior slabs will reduce the potential for landscape irrigation water to seep through the granular materials and cause the underlying soil subgrades to saturate or pipe. Prior to placing concrete, subgrade soils should be moisture conditioned to increase their moisture content to approximately 3 percent above laboratory optimum moisture (ASTM D-1557).

The expansive clayey soils and fills at the site could be subjected to volume changes during fluctuations in moisture content. As a result of these volume changes, some vertical movement of exterior slabs should be anticipated. This movement could result in damage to the exterior slabs and might require periodic maintenance or replacement. Adequate clearance should be provided between the exterior slabs and building elements that overhang these slabs, such as doors that open outward. We recommend reinforcing exterior slabs with steel bars in lieu of wire mesh. To reduce potential crack formation, considerations should be given to installing of #4 bars spaced at approximately 18 inches on center in both directions. Both score joints and expansion joints can be used to control cracking and allow for expansion and contraction of the concrete slabs.

We recommend appropriate flexible, relatively impermeable fillers be used at all expansion and cold joints. The installation of dowels at all expansion and cold joints will reduce differential slab movements; if used, the dowels should be at least 30 inches long and should be spaced at a maximum lateral spacing of 18 inches. Although exterior slabs that are adequately reinforced will still crack, trip hazards requiring replacement of the slabs will be reduced.

It should be noted, movements or failures of the exterior slabs should be anticipated after major liquefaction events. Repair of the exterior slabs, as well as site regrading, may be needed after the events.

7.2.7 Surface Drainage and Landscaping

We recommend exaggerated positive surface gradients that take into account potential differential ground settlements be provided adjacent to structures and for pavements to direct surface water toward suitable discharge facilities. Roof downspouts and landscaping drainage inlets should be connected to solid pipes that discharge into appropriate facilities. Ponding of surface water must not be allowed adjacent to structure foundations and exterior slabs, adjacent to pavements, at the top or adjacent to retaining walls.

To reduce moisture changes in the soils below and adjacent to structure foundations and exterior slabs, landscaping and irrigation systems should be designed and installed in a uniform and systematic manner as equally as possible on all sides of the foundations and adjacent to exterior slabs. If landscaping plans include trees, they should be planted a minimum distance of one-half the anticipated mature height of the trees from improvements to reduce the adverse effects from the tree roots. We recommend that drought resistant plants and low flow/drip irrigation watering systems be used. All irrigation systems should be regularly maintained and inspected for leakage. Over-watering must be avoided.

For bio-retention swales and basins (if planned), where they are located within 10 feet of infrastructure improvements (such as structure foundations, exterior flatwork, and pavements), we recommend they be lined with a relatively impermeable membrane to reduce water seepage and the potential for damage to other infrastructure improvements (such as foundations, exterior slabs, and pavements). The membrane can consist of a layer of STEGO Wrap 15-mil or equivalent installing below and along the sides of these facilities to direct the collected water into subdrain pipes. The membrane should be lapped and sealed in accordance with the manufacturer's requirements, including sealing joints where pipes penetrate the membrane.

The bio-treatment soil mix materials within swales and basins should be considered as having no lateral load resistant. We recommend the sidewall slopes of the swales and basins not to exceed 2:1 (horizontal to vertical) to reduce potential vertical and lateral movements of surrounding ground surface. In addition, we recommend either improvements (foundations, exterior slabs, and pavements) be setback beyond an imaginary 1:1 (horizontal to vertical) plane projected upward from the bottom edges of the swales and basins or the affected areas of the improvements be supported by deepening foundations or edges. Alternatively, properly designed below-grade enclosure structures can be used to build the swales and basins and to retain surrounding ground and improvements.

7.2.8 Construction During Wet Weather Conditions

If construction proceeds during or shortly after wet weather conditions, the moisture content of the onsite soils could be appreciably above optimum. Consequently, subgrade preparation, placement of onsite soil as structural fill might not be possible. A geotechnical engineer can provide alternative wet weather construction recommendations in the field at the time of construction, if appropriate.

7.3 Building Foundation System

The proposed new building foundation should be designed to provide proper bearing supports during the potential soil liquefaction events. Two foundation system are proposed: 1) deep foundation in combination with a permanent shoring system on the southeast side of the proposed new building foundation and 2) shallow foundation in combination with DMM ground improvement technique to mitigate the detrimental impacts from the potential lateral spreading and slope instability from the areas immediately adjacent to the Lake Merritt Channel. Our estimated lateral extent of potential ground lateral spreading/slope instability is shown on **Plate 3.**

Based on the proposed building layout, we recommend the permanent shoring system along the southeast side of the proposed building (estimated lateral spreading/slope instability lateral extent) be designed to retain a 12-foot-high column of soils, assuming the loss of adjacent ground support due to slope failure. The small portion of the shoring system located to further east of the estimated lateral spreading/slope instability lateral extent should be designed to retain an 18-foot high column of soils. Our recommended lateral pressures for the shoring system designs are shown on **Plates 14** and **15**.

We recommend the proposed new building be supported on a deep foundation system that extends to a depth of at least 70 feet (or to a pile tip Elevation of -50 feet) to transfer bearing loads to the sand and clay layers below the Young Bay Mud layer. Either precast pre-stressed concrete driven piles or drilled piles, such as Case-in-Drilled-Hole (CIDH) piers and auger cast piles, can be used at the site. The deep foundation should also be used to support any exterior elements that are considered essential parts of the building. Structural slabs should be designed to span between pile foundations. Detailed descriptions of the ground improvement technique are presented in DMM Design and Recommendations, **Supplement I and J**.

The deep foundation should be designed to resist downdrag loads that would be imposed upon the foundations due to soil liquefaction. Consideration should also be given to using flexible pipe connections to mitigate potential damage from the estimated potential liquefaction-induced settlement of 4 inches at locations where the pipes are connected to pile-supported structures.

Structures not supported on deep foundations may experience total areal ground surface settlements on the order of about 1 to 4 inches with locally up to about 6-1/2 inches of settlement. In the area immediately adjacent to the channel bank, the ground settlements may be larger than the above estimates if lateral spreading occurs.

7.3.1 Pile Axial Load Capacity

The new building can be supported by a deep foundation system that develops its load carrying capacity from soil friction/adhesion within the competent sand and clay layers below the Young Bay Mud. Either precast pre-stressed concrete driven piles or drilled piles, such as Case-in-Drilled-Hole (CIDH) piers and auger cast piles, can be used at the site

Piles should be at least 14 inches in square or diameter, extend to a depth of at least 70 feet (or to a pile tip Elevation of -50 feet), and have a center-to-center spacing of at least 3 times the pile dimension. The actual design lengths of the piles should also be determined using an ultimate skin friction of 1,500 psf (pounds per square feet) for the pile section located below the bottom of the Young Bay Mud layer. As indicated on **Plates 7** through **11**, the bottom of the Young Bay Mud layer is located at about 30 feet deep (Elevation of about -10 feet) at the northwest side of the proposed building location and about 50 feet deep (Elevation of about -30 feet) at the southeast side of the proposed building location. The pile section within and above the Young Bay Mud layer should be neglected in design for axial loading. The allowable axial capacity should be calculated by dividing the ultimate axial capacity by the factors of safety provided in the table below or the project structural design over strength factor (if applicable). Eighty percent (80 percent) of the skin friction value can be used to resist uplift.

Table 7.4: Recommended Factors of Safety for Axial Loading of Pile Foundation

Load Condition	Factor of Safety
Dead Load	3
Dead plus Live Loads	2
Total Loads (including wind or seismic)	1.5

The piles should also be designed to resist downdrag loads that would be imposed upon the foundations due to potential liquefaction of the isolated sand layers above Elevation of about -15 feet. We recommend an average negative skin friction of 650 psf be included along the upper 35 feet of the pile shaft to account for the potential liquefaction-induced downdrag forces from about 15 feet of fills, and 20 feet of Young Bay Mud with liquefied sand lenses. This value should be subtracted from the ultimate pile axial capacity.

A viscous bituminous coating can be applied on the upper 35 feet of pile shaft to reduce the downdrag loads. A fifty percent (50 percent) reduction is applicable to the above downdrag value when bituminous coating is used.

Static total and differential settlements of the pile supported structure are estimated to be insignificant (i.e., less than 0.5 inch) and within tolerable limits for the proposed structure. Seismic settlement of the pile is estimated to be less than 1 inch assuming the pile is designed to resist the downdrag force only using pile skin friction.

Regardless of the calculated pile lengths to meet axial capacity demands, a minimum of 35 feet of pile embedment is also needed to provide pile “fixity” to resist lateral loading based on the LPILE analysis results.

7.3.2 Pile Lateral Load Capacity

We evaluated pile lateral load capacities using the computer program LPILE (Ensoft, Ver. 2017.11.01) to model subsurface soils as a series of discrete springs with nonlinear behavior. Our analyses assumed a 70-foot long, 14-inch square elastic pile with a design concrete strength of 5,000 pounds per square inch (psi). The estimated flexural rigidity (EI) of the pile was reduced by fifty percent (50 percent) to account for an assumed twenty percent (20 percent) of pile section concrete crack in the direction of lateral loading. Pile axial loads were not included in our analyses.

Four (4) different soil profiles (1, 2, 3A & 3B) along the Cross-Section A-A’ (**Plate 7**) were established in our analysis models based on the idealized subsurface soil conditions at the site. The locations of these profiles are shown on **Plate G-1** for reference, included in **Supplement G**.

Both Profiles 1 and 2 have the same soil stratigraphy, besides the thickness of the Young Bay Mud layer. An additional saturated highly liquefiable sand layer was also included in Profiles 3A and 3B between the surficial fill layer and the underlying Young Bay Mud layer. In Profile 3B, this sand layer was assumed to be liquefied during earthquake events. A design groundwater table at an elevation of +8 feet were used for all profiles. The detailed soil stratigraphy and engineering properties used in our analyses are in the tables below.

Table 7.5: Soil Engineering Properties for Profile 1

Depth Below Ground Surface	Soil Layer	Model Used	Effective Unit Weight (pcf)	Material Properties					
				Undrained Cohesion c (psf)		Strain at 50% Stress		Friction Angle Φ′ (degrees)	p-y Modulus, k (pci)
				Top	Bottom	Top	Bottom		
0 to 12 feet	Sandy Fill	Reese (Sand)	120	-	-	-	-	32	90
12 to 30 feet	Young Bay Mud with Sand Lenses	Soft Clay (Matlock)	26	504	668	0.02	0.01	-	-
Below 30 feet	Sand and Clays	Reese (Sand)	66	-	-	-	-	40	125

Table 7.6: Soil Engineering Properties for Profile 2

Depth Below Ground Surface	Soil Layer	Model Used	Effective Unit Weight (pcf)	Material Properties					
				Undrained Cohesion c (psf)		Strain at 50% Stress		Friction Angle Φ′ (degrees)	p-y Modulus, k (pci)
				Top	Bottom	Top	Bottom		
0 to 12 feet	Sandy Fill	Reese (Sand)	120	-	-	-	-	32	90
12 to 43 feet	Young Bay Mud with Sand Lenses	Soft Clay (Matlock)	26	504	786	0.02	0.01	-	-
Below 43 feet	Sand and Clays	Reese (Sand)	66	-	-	-		40	125

Table 7.7: Soil Engineering Properties for Profile 3A

Depth Below Ground Surface	Soil Layer	Model Used	Effective Unit Weight (pcf)	Material Properties					
				Undrained Cohesion c (psi)		Strain at 50% Stress		Friction Angle Φ′ (degrees)	p-y Modulus, k (pci)
				Top	Bottom	Top	Bottom		
0 to 7 feet	Sandy Fill	Reese (Sand)	120	-		-	-	32	90
7 to 18 feet	Highly Liquefiable Sands	Reese (Sand)	46	-	-	-	-	33	60
18 to 41 feet	Young Bay Mud with Sand Lenses	Soft Clay (Matlock)	26	504	668	0.02	0.01	-	-
Below 41 feet	Sand and Clays	Reese (Sand)	66	-		-		40	125

Table 7.8: Soil Engineering Properties for Profile 3B

Depth Below Ground Surface	Soil Layer	Model Used	Effective Unit Weight (pcf)	Material Properties					
				Undrained Cohesion c (psi)		Strain at 50% Stress		Friction Angle Φ′ (degrees)	p-y Modulus , k (pci)
				Top	Bottom	Top	Bottom		
0 to 7 feet	Sandy Fill	Reese (Sand)	120	-		-	-	32	90
7 to 18 feet	Highly Liquefiable Sands	Liquefied Sand (Rollins)	46	-	-	-	-	-	-
18 to 41 feet	Young Bay Mud with Sand Lenses	Soft Clay (Matlock)	26	504	668	0.02	0.01	-	-
Below 41 feet	Sand and Clays	Reese (Sand)	66	-		-		40	125

Both free and fixed pile head conditions were examined in our analyses. Our estimated lateral loads for 1/4-inch, 1/2-inch, and 1 inch of lateral displacements at pile heads for each pile head condition and loading case (1 through 6) are presented in the tables for each soil profile. The calculated pile head deflection, bending moment, and shear force versus embedment depth are presented in **Supplement H**. It should be noted that no factor of safety was applied to the estimated loads or deflections.

Table 7.9: Estimated 70′ Long 14″ Square Pile Lateral Load Capacities – Profile 1

Loading Case	Pile Head Condition	Pile Head Displacement (in)	Lateral Load at Pile Head (kips)	Maximum Moment in Pile (kip-ft)
1	Free	0.25	9	25
2	Free	0.5	13	44
3	Free	1.0	21	77
4	Fixed	0.25	20	71
5	Fixed	0.5	33	125
6	Fixed	1.0	53	221

Table 7.10: Estimated 70′ Long 14″ Square Pile Lateral Load Capacities – Profile 2

Loading Case	Pile Head Condition	Pile Head Displacement (in)	Lateral Load at Pile Head (kips)	Maximum Moment in Pile (kip-ft)
1	Free	0.25	9	25
2	Free	0.5	13	44
3	Free	1.0	21	77
4	Fixed	0.25	20	71
5	Fixed	0.5	33	125
6	Fixed	1.0	53	221

Table 7.11: Estimated 70′ Long 14″ Square Pile Lateral Load Capacities – Profile 3A

Loading Case	Pile Head Condition	Pile Head Displacement (in)	Lateral Load at Pile Head (kips)	Maximum Moment in Pile (kip-ft)
1	Free	0.25	8	25
2	Free	0.5	13	43
3	Free	1.0	21	78
4	Fixed	0.25	20	70
5	Fixed	0.5	33	124
6	Fixed	1.0	53	220

Table 7.12: Estimated 70′ Long 14″ Square Pile Lateral Load Capacities – Profile 3B

Loading Case	Pile Head Condition	Pile Head Displacement (in)	Lateral Load at Pile Head (kips)	Maximum Moment in Pile (kip-ft)
1	Free	0.25	8	23
2	Free	0.5	12	39
3	Free	1.0	19	67
4	Fixed	0.25	17	58
5	Fixed	0.5	26	94
6	Fixed	1.0	37	146

Where competent subgrade soils exist, a soil passive resistance equal to an equivalent fluid weighing 350 pcf (pounds per cubic foot), which acts against the vertical face of the pile cap and grade beam (assumes a deflection of approximately 1/2 inch), can also be used in conjunction with the above estimated pile shaft lateral load capacities. A higher soil passive resistance equal to an equivalent fluid weighing 450 pcf can be used for the portion of the surficial fills that is properly over-excavated and re-compacted as engineered fills. The upper 12 inches of soils should be neglected in passive resistance design unless they are confined by a pavement or slab. This value can be used without reduction if the pile shaft lateral load capacity is also based on a compatible 1/2 inch pile head displacement. Any portion of the pile cap, grade beam and shaft located above an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent utility trenches should be ignored in the passive resistance design.

For closely spaced piles, the shear planes in the soil overlap and the lateral resistance for a pile within the group is less than that of a single pile. We note that the leading piles are generally less impacted by group effects and tend to draw higher loads. To account for the reduction of soil resistance because of group effects, we recommend multiplying the lateral loads by the reduction factors provided in the table below**.** Reduction factors, or p-multipliers, are a function of center-to-center spacing where D is the pile diameter. P-multipliers should be applied to trailing piles in the direction of loading.

As an example, a 1 by 6 pile row with a center-to-center spacing of 6 diameters and loaded in the direction parallel to the pile row would use a p-multiplier of 1.0 for the lead pile and 0.7 for all trailing piles. The same group loaded perpendicular to the pile row would use a p-multiplier of 1.0 for all piles. Linear interpolation may be used for other pile spacing.

Table 7.13: Reduction Factors for Pier Lateral Load Capacity

Center-to-Center Spacing	p-Multiplier
8D	1.0
6D	0.7
4D	0.4
3D	0.3

7.3.3 Pile Construction

We recommend that the installation or excavation of all piles be performed under the direct observation of the project Geotechnical Engineer to confirm that the piles are founded in suitable materials and constructed in accordance with the recommendations presented herein. All piles should be installed or constructed vertically to their design tip elevations at the specified locations to develop adequate vertical pile capacities.

The pile driving hammer and the methods of handling, picking, and setting the piles should be properly selected by the contractor and reviewed by both the project Structural Engineer and Geotechnical Engineer. It is possible for a very large or very small hammer to cause damage to the pile it is driving. The pile driving criteria should be stablished by the Contractor in conjunction with the project Geotechnical Engineer by performing a wave equation analysis (WEAP) after selections of type and size of pile and pile hammer have been finalized, and prior to pile installation.

In addition, we recommend an indicator pile program be performed for the project, which consists at least 5 indicator piles and Pile Dynamic Analyzer (PDA) tests. The indicator piles should be performed in close proximity to the exploratory borings and CPTs to determine the lengths for production piles and driving resistance of the piles, as well as to verify the pile capacities and the anticipated soil profile across the site. The indicator piles should be at least 10 feet longer than the anticipated design length of the production piles. The indicator pile program should be conducted using the same equipment and same installation methods that will be used for installing the production piles. Due to the potential for encountering hard driving within dense sands below the Young Bay Mud layer, we recommend that the moment resisting reinforcement in the indicator piles be deepened 10 to 20 feet in anticipation of possible pile cutoffs.

The project Geotechnical Engineer should observe the driving of all indicator and production piles and in no case should driving be terminated without the approval of the project Geotechnical Engineer. The project Geotechnical Engineer should evaluate the allowable capacity of any piles driven shorter than their anticipated lengths.

We recommend predrilling through the existing fill layer be performed at driven pile locations to avoid obstructions and potential damage to the piles. The pre-drilled holes should have a diameter less than the 3/4 the diagonal width of the piles.

7.3.4 Deep Mixing Method (DMM) Ground Improvement

Several alternatives were considered for mitigating the lateral spread hazard at the planned building site, including installation of a retaining wall and the deep mixing method (DMM) beneath the building footprint. Considering the high seismic demand, presence of shallow liquefiable soils and soft Young Bay Mud, proximity to the Lake Merritt Channel, and constraints from the PG&E easement on the north side of the planned building, it is our experience and opinion that continuous grids of deep mixed shear walls are the most suitable, robust, and cost-effective technique to mitigate the lateral spread hazard at the planned building site. The grids of deep mixed shear walls will provide support for shallow foundation systems for seismic loading and transfer bearing loads deeper to the medium dense to very dense sands and stiff to hard clays, reducing total and differential building settlements. In addition, we recommend using structural slabs to span between DMM deep mixed shear walls, assuming that the untreated soils within the grid walls may still develop post-liquefaction reconsolidation settlements below slabs. The deep mixed shear walls will also affect the composite ground response to horizontal ground motions. This section presents a brief overview of the deep mixing method (DMM), our design approach, DMM design properties, and results of our evaluation process, including results of seismic stability analyses. Seismic design parameters incorporating the composite response of the deep mixed zone are presented in **Supplement I and J**, herein.

7.3.5 Building Ground Interior Slab

The interior ground slab should consist structural slabs that are designed to span between pile foundations. The slab should be underlain by an at least 12 inches of properly compacted engineered fills that extend at least 3 feet beyond the foundation footprints.

If migration of water vapor through interior slab is undesirable, we recommend a vapor retarder and an underlying 4-inch layer of ¾-inch, clean, crushed, uniformly graded gravel/drain rock be placed between the bottom of the slab and the recommended engineered fill layer. The gravel/drain rock layer can be considered as part of the non-expansive engineered fill layer. We recommend the vapor retarder consist of a single layer of Stego Wrap Vapor Barrier 15 mil or equivalent provided the equivalent satisfies the following criteria: a permeance less than 0.01 perms as guided by ACI 302.2R, Class A strength as determined by ASTM E1745, and a thickness of at least 15 mils. Installation of the vapor retarder, including protrusions where pipes or conduit penetrate the membrane, should conform to ASTM E1643 and the manufacturer's requirements. Care must be taken to protect the membrane from tears and punctures during construction. We do not recommend placing sand or gravel over the membrane. The subgrade below the slab should be property prepared, firm, and non-yielding. All foundation excavations should be kept moist and free of loose soils and standing water prior to concrete placement.

Concrete slabs retain moisture and often take many months to dry; construction water added during the concrete pour further increases the curing time. If the slabs are not allowed to completely cure prior to constructing the super-structure, the concrete slabs will expel water vapor and the vapor will be trapped under impermeable flooring. A proper water/cement ratio should be determined by the foundation designers for the slabs to reduce vapor transmitting if need. We recommend the foundation designer determine if corrosion protection is needed for the foundation concrete and reinforcing steel. The corrosivity test results of onsite soil samples and a brief evaluation report by others are included in **Supplement B**; the foundation designer should determine if additional testing is needed. In addition, the foundation designers should provide recommendations to reduce the potential for differential concrete curing if necessary.

7.4 Retaining Walls

Retaining walls can be supported on spread footing or pile foundations. Fill placed behind walls should conform to the engineered fill materials, and fill placement and compaction recommendations. If heavy compaction equipment is used behind the walls, the walls should be appropriately designed to withstand loads exerted by the heavy equipment and/or temporarily braced.

For retaining walls not to be supported on piles, a "zero net load" approach should be used for the wall design and construction to reduce the soil consolidation settlement below the walls. Detailed descriptions of the approach are provided in **Section 6.3**. It should be noted that walls located within the area of potential ground lateral spreading/slope instability (east of the dashed line) may potentially experience large vertical and lateral movements during major earthquake events.

7.4.1 Lateral Loads

Any walls that retain soils should be designed to resist both lateral earth pressures and any additional lateral loads caused by roadway surcharging, earthquake loading, and hydrostatic pressure if the walls are located below groundwater table. Considerations should be given to applying waterproofing to backside of the wall to reduce water/vapor transmission and efflorescence forming on the front wall face.

We recommend that any undrained unrestrained walls are free to deflect or rotate be designed to resist an equivalent fluid pressure of 85 pounds per cubic foot (pcf). Undrained restrained walls should be designed to resist an equivalent fluid pressure of 100 pcf. This assumes walls with level backfills. Walls with inclined backfill should be designed for an additional equivalent fluid pressure of 1 pound per cubic foot for every 2 degrees of slope inclination. Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to 1/3 the anticipated surcharge load for unrestrained walls, and 1/2 the anticipated surcharge load for restrained walls.

If back-drainage is provided behind the walls, we recommend that drained unrestrained walls be designed to resist an equivalent fluid pressure of 50 pounds per cubic foot (pcf). Drained restrained walls should be designed to resist an equivalent fluid pressure of 75 pcf. These recommended drained lateral pressures assume walls are fully-back drained to prevent the build-up of hydrostatic pressures. This can be accomplished by using ½ to ¾ inch crushed, uniformly graded gravel entirely wrapped in filter fabric, such as Mirafi 140N or equal (an overlap of at least 12 inches should be provided at all fabric joints). The gravel and fabric should be at least 8 inches wide and extend from the base of the wall to within 12 inches of the finished grade at the top (Caltrans Class 2 permeable material (Section 68) may be used in lieu of gravel and filter fabric). A 4-inch diameter, perforated pipe should be installed at the base and centered within the gravel. The perforated pipe should be connected to a solid collector pipe that transmits the water directly to suitable discharge facilities. If weep holes are used in the wall, the perforated pipe within the gravel is not necessary provided the weep holes are kept free of animals and debris, are located no higher than approximately 6 inches from the lowest adjacent grade and are able to function properly. As an alternative to using gravel, pre-fabricated drainage panels (such as AWD SITEDRAIN Sheet 94 for walls or equal) may be used behind the walls in conjunction with perforated pipe (connected to solid collector pipe), weep holes, or strip drains (such as SITEDRAIN Strip 6000 or equal).

For walls that are higher than 6 feet, we recommend the walls also be designed to resist a uniform lateral pressure of 38H pcf for both unrestrained and restrained wall conditions based on the ground acceleration from a design basis earthquake (Seed and Whitman, 1970; Atik and Sitar, 2007), where H is the height of the retaining portion of the walls. This seismic induced earth pressure is in addition to the pressures noted above. Due to the transient nature of the seismic loading, a factor of safety of at least 1.1 can be used in the design of the walls when they resist seismic lateral loads.

7.4.2 Wall Footing Foundation

Retaining walls can be supported by conventional spreading footings that are designed for "zero net load" and bear on competent onsite fills. Over-excavation and re-compaction of any weak fills below the footings may be required due to the heterogenous nature of the onsite existing fills. The bottom of the footings should be at least 12 inches wide and founded at least 24 inches below lowest adjacent finished grade. Deeper embedment will be required for footings that are located adjacent to or near top of slopes. Portion of the footings located within 10 feet (as measured laterally) of the slope face should be ignored in both vertical and passive resistance design.

Footings located adjacent to other footings or utility trenches should also bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent footings or utility trenches. Alternatively, the foundation reinforcing could be increased to span the area defined above assuming no soil support is provided. Our recommended allowable spread footing bearing pressures are provided below. These allowable bearing pressures are net values; therefore, the weight of the footing can be neglected for design purposes.

Table 7.14: Allowable Wall Spread Footing Bearing Pressures

Load Condition	Allowable Bearing Pressure (psf)	Factor of Safety
Dead Load	"Zero Net Load"	-
Dead plus Live Loads	"Zero Net Load"	-
Total Loads (including Wind or Seismic)	3,000	1.5

Resistance to lateral loads can be provided by friction along the base of footings and by passive pressures acting on the sides of footings. An allowable friction coefficient of 0.3 times the dead load (a factor of safety of 1.5) may be used to evaluate the allowable frictional resistance along the bottom of footings. Where the footing is poured neat against competent subgrade soils, a passive pressure equal to an equivalent fluid pressure of 350 pounds per cubic foot (pcf) can be used for lateral load resistance against the sides of footings perpendicular to the direction of loading. The upper 12 inches of soils should be ignored, unless they are confined by pavement or slab. This passive resistance should be considered as an ultimate value (a factor of safety of 1.0) and assumes a deflection of approximately 0.5 inch to fully mobilize the passive resistance.

7.5 Additional Geotechnical Services

Fugro should review geotechnical aspects of the plans and specifications to check for conformance with the intent of our recommendations. We recommend that Fugro be also retained to provide geotechnical services during earthwork operation and foundation installation to observe compliance with the design concepts, specifications, and recommendations presented in this report. Our presence will also allow us to modify design if unanticipated subsurface conditions are encountered.

8. Limitations

The opinions, conclusions, and recommendations presented in this report are based on our reviews of available geologic and geotechnical data, maps, reports, our site subsurface exploration and laboratory testing results, our engineering analysis results, and information provided by others. Our opinions, conclusions, and recommendations are solely professional opinions and were made in accordance with generally accepted local and current geotechnical engineering principles and practices. We make no warranty, either express or implied.

Site exploration and testing characterizes subsurface conditions only at the locations where the explorations or tests are performed and at the time when services were conducted; actual subsurface conditions between explorations or tests may be different than those described in this report. Variations of subsurface conditions from those analyzed or characterized in this report are not uncommon and may become evident during construction. In addition, changes in the condition of the site can occur over time as a result of either natural processes (such as earthquakes, flooding, or changes in ground water levels) or human activity (such as construction adjacent to the site, dumping of fill, or excavating). If changes to the site's surface or subsurface conditions occur since the performance of the field work described in this report, or if differing subsurface conditions are encountered, we should be contacted immediately to evaluate the differing conditions to assess if the opinions, conclusions, and recommendations provided in this report are still applicable or should be amended.

This report has been prepared for the exclusive use of Peralta Community College District and their consultants for specific application to the proposed Laney College Library Learning Resource Center in Oakland, California as described herein. If there are any changes in the nature, design or location of the project, as described in this report, or if any future additions are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the project changes are reviewed by us, and the conclusions and recommendations presented in this report are modified or verified in writing.

Reliance on this report by others must be at their risk unless we are consulted on the use or limitations. We cannot be responsible for the impacts of any changes in geotechnical standards, practices, or regulations subsequent to performance of services without our further consultation. We can neither vouch for the accuracy of information supplied by others, nor accept consequences for use of segregated portions of this report without our prior consultation.

9. References

Aagaard, B., Blair, Boatwright, J., Garcia, Harris, Michael, A., Schwartz, D., and DiLeo, 2016. Earthquake Outlook for the San Francisco Bay Region 2014–2043, USGS Fact Sheet 2016–3020, Revised August 2016 (ver. 1.1).

American Society of Civil Engineers (ASCE), 2016, ASCE/SEI 7-16 Standard: Minimum Design Loads for Buildings and Other Structures. ASCE and Structural Engineers Institute (SEI), Reston, Virginia.

Association of Bay Area Governments Resilience Program, ABAG Liquefaction Susceptibility Map, http://gis.abag.ca.gov.

Al Atik, L. and Sitar, N., 2007, Development of Improved Procedures for Seismic Design of Buried and Partially Buried Structures, Pacific Earthquake Engineering Research Center.

Atwater, T.M.,1970. Implications of Plate Tectonics for the Cenozoic Evolution of Western North America, GSA Bulletin v. 81, p. 3513-3536.

Atwater, T.M. and Stock, J.M., 1998. Pacific-North America Plate Tectonics of the Neogene Southwestern United States: An Update. International Geology Review v. 40, p. 375-402.

Boatwright, J., and Bundock, H., 2005, Modified Mercalli Intensity Maps for the 1906 San Francisco Earthquake Plotted in ShakeMap Format: U.S. Geological Survey Open-File Report 2005-1135.

Boulanger, R., and Idriss, I., 2014. CPT and SPT Based Liquefaction Triggering Procedures, UC Davis Center for Geotechnical Modeling, Report No. UCD/CGM-14/01, April 2014.

California Building Standards Commission, 2019, 2019 California Building Code (CBC), California Code of Regulations Title 24, Part 2, Volume 2 of 2; Published by the International Codes Commission.

California Geological Survey (CGS), 1982. Earthquake Fault Zones, *in* Earthquake zones of required investigation, Oakland West Quadrangle, Revised Official Map, Released: January 1, 1982.

CGS, 2002, California Geomorphic Provinces: CGS Note 36, https://www.conservation.ca.gov/cgs/Documents/Publications/CGS-Notes/CGS-Note-36.pdf.

CGS, 2003. Seismic hazard zones, *in* Earthquake zones of required investigation, Oakland West Quadrangle Official Map, Released: February 14, 2003.

CGS, 2003. Seismic Hazard Zone Report for the Oakland West 7.5-Minute Quadrangle, Alameda County, California.

CGS, 2008. Guidelines for Evaluating and Mitigating Seismic Hazards in California, CGS Special Publication 117A.

CGS, 2009. Tsunami Inundation Map for Emergency Planning, Oakland West Quadrangle, July 31.

CGS, 2019. Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings: CGS Note 48.

City of Oakland Community and Economic Development Agency, 2004. Safety Element, City of Oakland Safety Plan, November 2004.

CSW/Stuber-Stroeh Engineering Group, Inc., 2019. CAD file, received in April 2019.

Federal Emergency Management Agency, 2018. Flood Insurance Rate Map (FIRM), Panel 06001C0067H (12/21/18).

Felzer, K.R., and Cao, TQ, 2008, WGCEP Historical California earthquake catalog, Supplement H in The Uniform California Earthquake Rupture Forecast, version 2 (UCERF 2): U.S. Geological Survey Open-File Report 2007-1437H and California Geological Survey Special Report 203H, 127 p. [http://pubs.usgs.gov/of/2007/1437/h/].

Fugro, 2002. Geotechnical Investigation, New Art Building at Laney College, Fugro No. 1430.001, March 27.

Fugro, 2005. Geotechnical Study and Geologic Hazard Evaluation, Laney College Art Building, Fugro No. 1430.005, March 29.

Fugro, 2006. Geologic Hazards Evaluation, Laney College Building A Renovation, Fugro No. 1430.008, August 25.

Fugro, 2008. Geotechnical Review, Proposed New Laney College Library Site Study, Fugro No. 1813.002, June 10.

Geotechnical Engineering, Inc., 2006. Additions to Building A & Chiller Room Adjacent to Building B, Laney College, GEI No. 41357, March 20.

Harza Kaldveer, 1993. Geotechnical Investigation for Proposed Pool Replacement, Laney College, Harza No. K1329, October 22.

Helley, E., and Graymer, R., 1997. Quaternary Geology of Alameda County, and Parts of Contra Costa, Santa Clara, San Mateo, San Francisco, Stanislaus, and San Joaquin Counties, California: A Digital Database, USGS Open File Report 97-97.

Holzer, T.,1998. The Loma Prieta, California, Earthquake of October 17, 1989 - Liquefaction, USGS Professional Paper 1551-B.

Holzer, T., Bennett, M., Noce, T., Padovani, A., and Tinsley, J., 2002. Liquefaction Hazard and Shaking Amplification Maps of Alameda, Berkeley, Emeryville, Oakland, and Piedmont, California: A Digital Database, USGS Open File Report 2002-02-296, revised 2010.

Idriss, I., and Boulanger, R., 2008. Soil Liquefaction During Earthquakes, EERI Monograph MNO-12.

Ishihara, K., and Yoshimine, M.,1992. Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes, Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineer, Volume 32 Issue 1, p. 173-188.

Kaldveer Associates, 1991. Feasibility Foundation Investigation, Proposed Pool Improvements, Laney College, Kaldveer No. K1329-1-863, September 9.

Knudsen, K.L., Sowers, J.M., Witter, R.C., Wentworth, C.M., and Helley, E.J., 2000, Preliminary Maps of Quaternary Deposits and Liquefaction Susceptibility, Nine-County San Francisco Bay Region, California: A Digital Database: U.S. Geological Survey Open-File Report 00-444.

Lawson, A.C., 1908, The California earthquake of April 18, 1906, Report of the State Earthquake Investigation Commission, two volumes and atlas: Carnegie Institution of Washington, Washington, D.C., reprinted 1969.

McNutt, S.R., and Toppozada, T.R., 1990, Seismological aspects of the 17 October, 1989 Earthquake, in McNutt S.R., and Sydnor, R.H., eds., The Loma Prieta (Santa Cruz Mountains), California Earthquake of 17 October, 1989: California Division of Mines and Geology Special Publication 104, pp. 11-27.

Miller, C.D.,1989. Potential hazards from future volcanic eruptions in California, USGS Bulletin 1847, 17 p.

National Atlas of United States, 2005, Significant United States Earthquakes, 1568-2009, USGS dataset, Available at: http://purl.stanford.edu/sy174vb2193.

Noll and Tam Architects and Planners, September 10, 2008. Laney College Learning Resource Center, Sheets A1.00 to A1.7.

Northern California Earthquake Data Center (NCEDC), 2018. http://www.quake.geo.berkeley.edu/anss/catalog-search.html, website accessed on March 30, 2018.

Radbruch, D., 1969. Areal and Engineering Geology of the Oakland East Quadrangle, California, USGS Map GQ-79, Scale 1:24,000.

Robertson, D., and Shao, L., 2010. Estimation of Seismic Compression in Dry Soils Using the CPT, Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Paper No. 4.05a.

Rogers, J.D., and Figuers, S., 1991. Engineering Geologic Site Characterization of the Greater Oakland-Alameda Area, Alameda and San Francisco Counties, California, NSF Grant No. BCS-9003785, December 30.

Seed, H.B., 1979. Considerations in the Earthquake-Resistant Design of Earth and Rockfill Dams, Geotechnique, V. 29 (3), p. 215-263.

Seed, H.B., and Whitman, R.V.,1970, Design of Earth Retaining Structures for Dynamic Loads, *in* ASCE Specialty Conf.-Lateral Stress in the Ground and Design of Earth Retaining Structures, p 103-147.

Sloan, D., 2006, Geology of the San Francisco Bay Region, University of California Press, 360 p.

South California Earthquake Center, 2002. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California, June.

Terraphase Engineering, 2012. Geotechnical Design Report, Proposed Laney College Building Efficiency for a Sustainable Tomorrow (BEST), Terraphase No. 0034-001-003, May 31.

Tinsley, J., Egan, J., Kayen, R., Bennett, M., Kropp, A., and Holzer, T., 1998. Supplement: Maps and Descriptions of Liquefaction and Associated Effects, The Loma Prieta, California, Earthquake of October 17, 1989 - Liquefaction, USGS Professional Paper 1551-B.

Tokimatsu, K., and Seed, H.B., 1987. Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Geotechnical Engineering, Vol. 113, Issue 8.

Toppozada, T.R. 2000, San Andreas M≥5.5 earthquakes from Parkfield to Fort Bragg, California, 1800 to 1999: Proceedings of the 3rd Conference on Tectonic Problems of the San Andreas Fault System, September 6-8, 2000, Stanford University, Palo Alto, California [http://pangea.stanford.edu/GP/sanandreas2000/]

Toppozada, T.R., and Parke, D.L., 1982, Area damaged by the 1868 Hayward earthquake and recurrence of damaging earthquakes near Hayward: Proceedings of the Conference on Earthquake Hazards in the Eastern San Francisco Bay Area: California Geological Survey Special Publication 62, p. 321-328.

U.S. Geological Survey (USGS), 2008. National Seismic Hazard Maps – Source Parameters website, https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query_main.cfm.

U.S. Geological Survey (USGS) Website, https://earthquake.usgs.gov/hazards/designmaps/usdesign.php.

Wallace, R.E. ed., 1990, The San Andreas fault system, California. U.S. Geological Survey Professional Paper 1515.

Witter, R., Knudsen, K., Sowers, J., Wentworth, C., Koehler, R., and Randolph, C., 2006. Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California, US Geological Survey Open-File Report 2006-1037.

Woodward-Clyde-Sherard and Associates, 1966. Soil Investigation for the Proposed Peralta Junior College Civic Center Site, Phase 1 – Preliminary Studies, WCS No. S10312, March 9.

Woodward-Clyde-Sherard and Associates, 1967. Peralta College – Chinatown General Neighborhood Renewal Area (GNRA), WCS No. 11032, May 1.

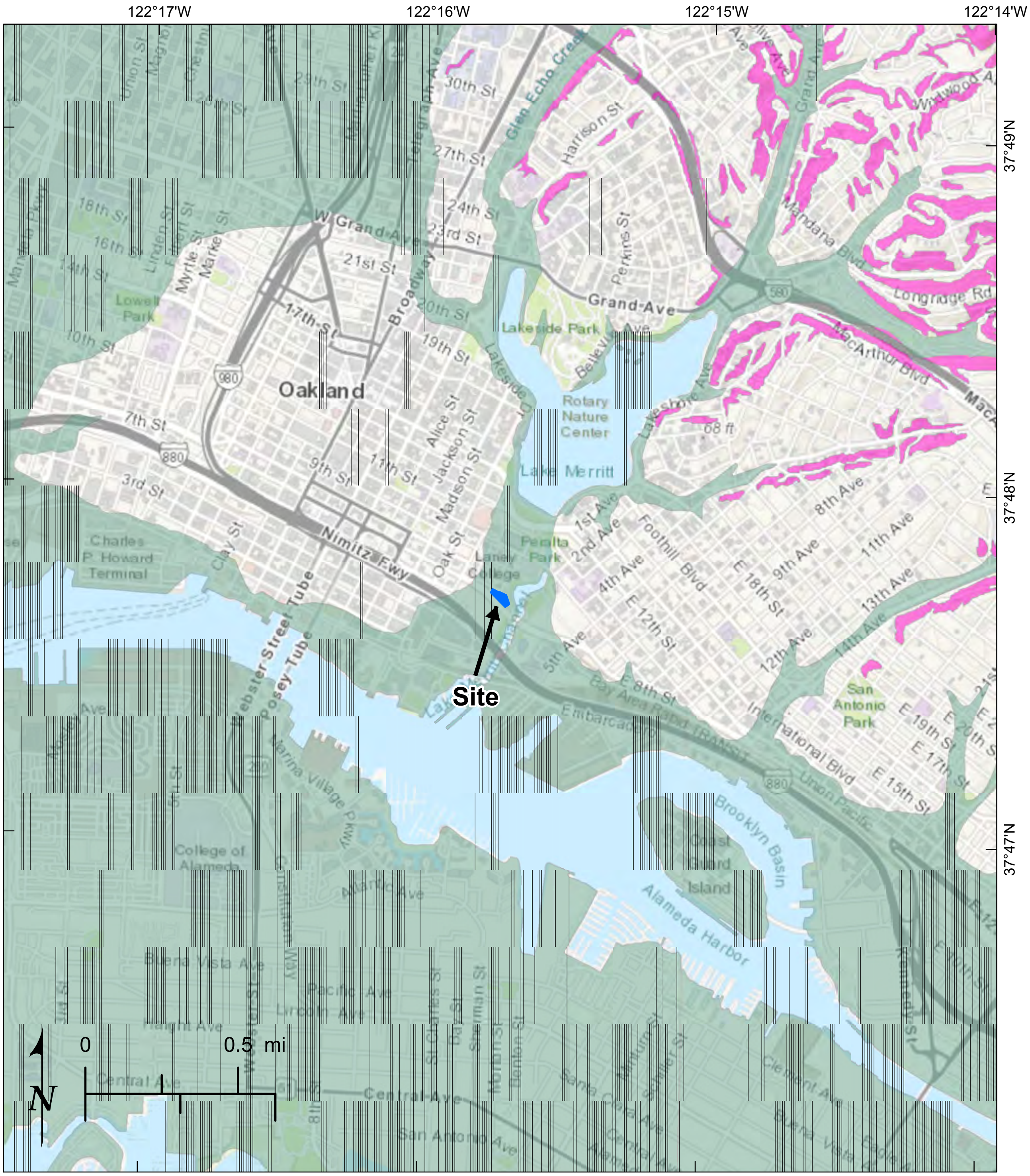
Youd, L., and Hoose, S.,1978. Historical Ground Failures in Northern California Triggered by Earthquakes, USGS Professional Paper 993.

Youd, L., and Idriss, I., 2001. Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, ASCE Journal of Geotechnical and Geoenvironmental Engineering, April 2001, pp. 297-813.

Youd, L., Hansen, C., and Bartlett, S., 2002. Revised Multilinear Regression Equations for Predication of Lateral Spread, Journal of Geotechnical and Geoenvironmental Engineering, p. 1007-1017, December.

Zhang, G., Robertson, P.K. and Brachman, R.W.I., 2004. Estimating Liquefaction-induced Lateral Displacement Using the Standard Penetration Test or Cone Penetration Test, Journal of Geotechnical and Geoenviromental Engineering, ASCE Vol. 130, Issue 8, August.

List of Plates



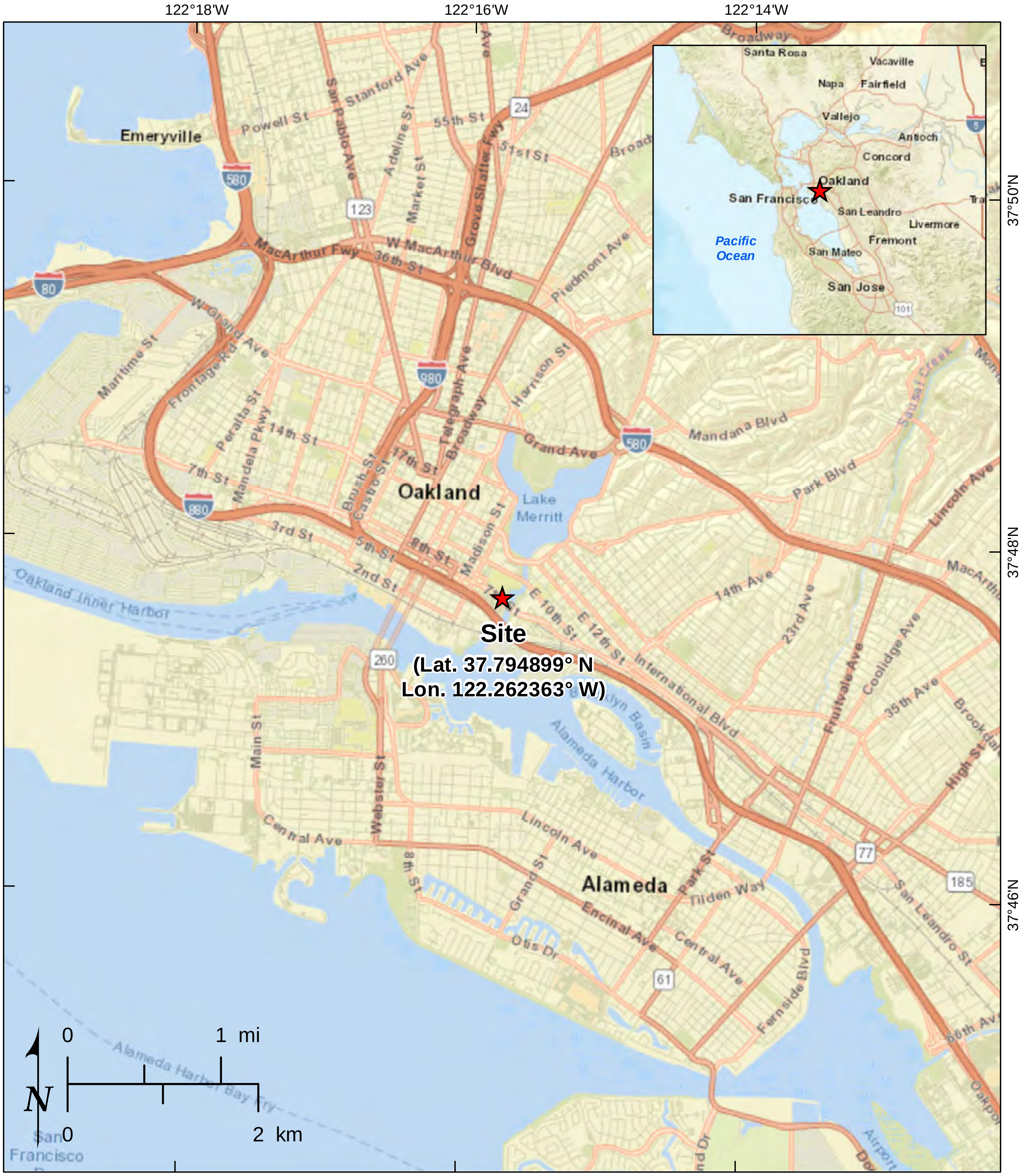
Base map from ESRI, 2023

Legend

Earthquake-induced landslide zone (CGS, 2003)

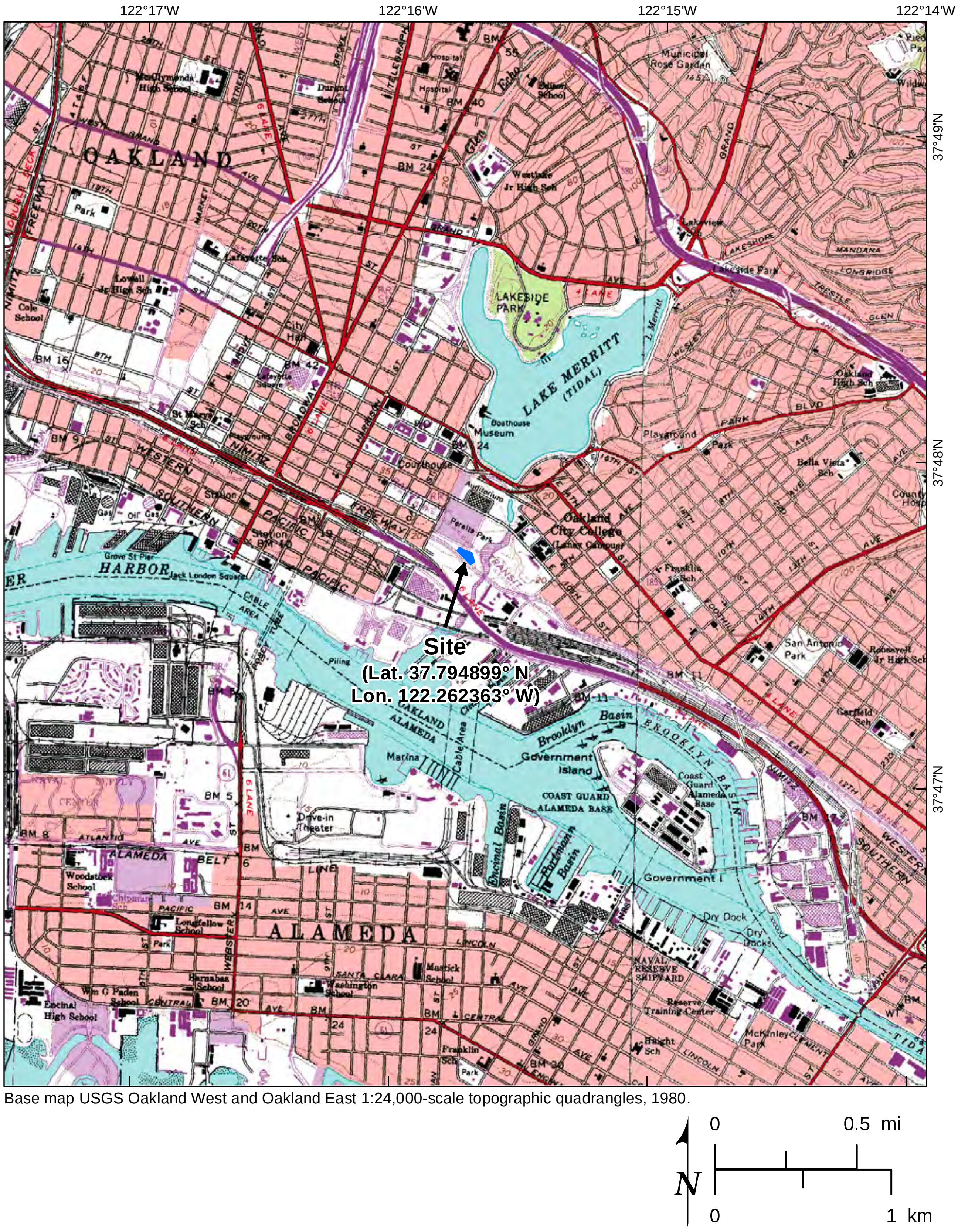
Liquefaction zone (CGS, 2003)

Plate-6: CGS Seismic Hazard Zone Map



Base map from Esri, 2023.

Plate-1: Vicinity Map

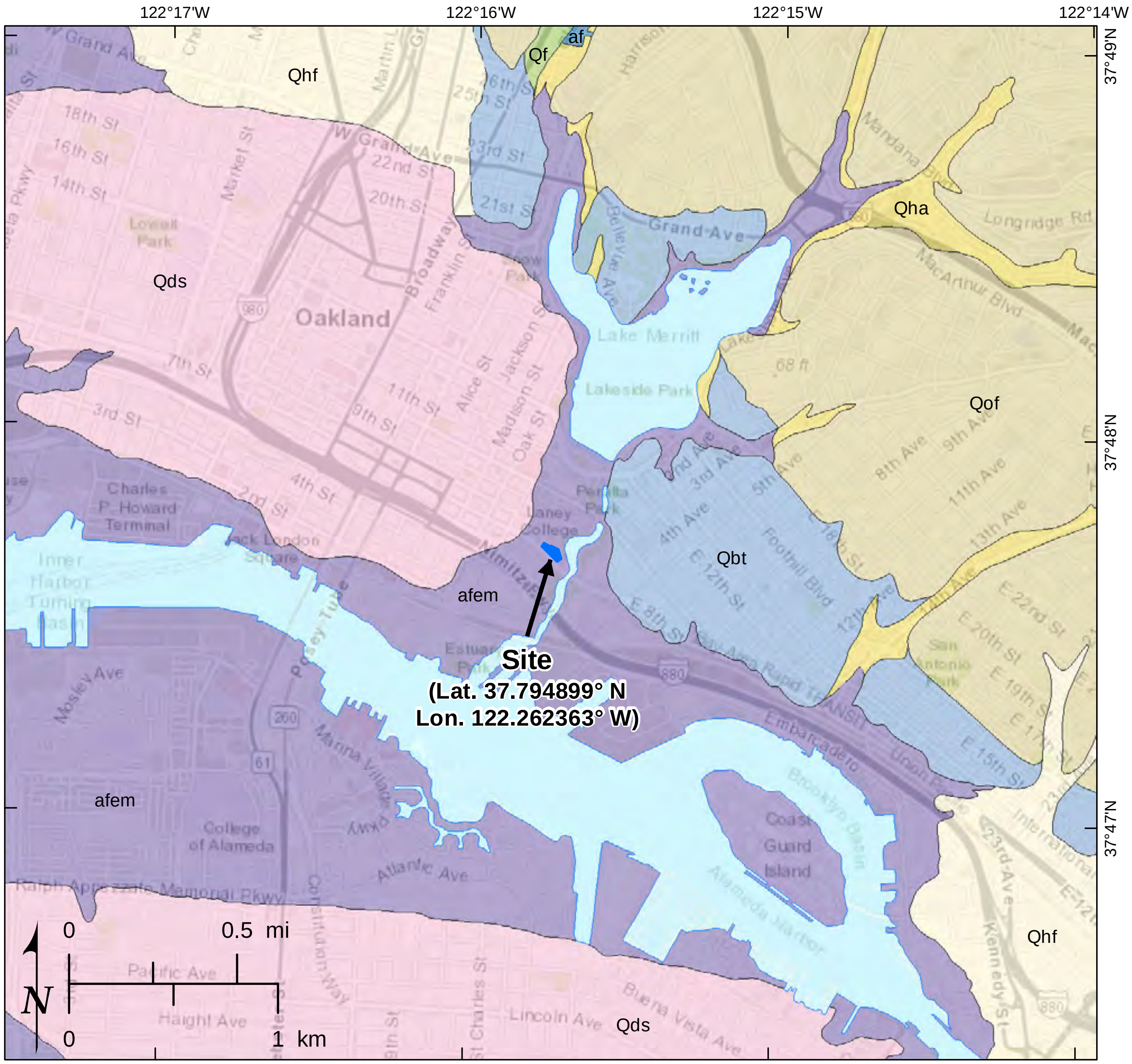


Base map USGS Oakland West and Oakland East 1:24,000-scale topographic quadrangles, 1980.

Plate-2: Topographic Site Map

Aerial imagery from Bing Maps. Topo contours provided by CSW/Stuber-Stroeh, April 2019. Proposed building location provided by Noll and Tam Architects, January 2020.

Figure 3: Site Plan



Base map USGS Oakland West and Oakland East 1:24,000-scale topographic quadrangles.Geologic map: Witter et al, 2006.

Legend

HISTORICAL

- af — Artificial fill
- afem — Artificial fill over estuarine mud

HOLOCENE TO LATEST PLEISTOCENE

- Qds — Dune sand
- Qf — Alluvial fan deposits

EARLY TO LATE PLEISTOCENE

- Qof — Alluvial fan deposits

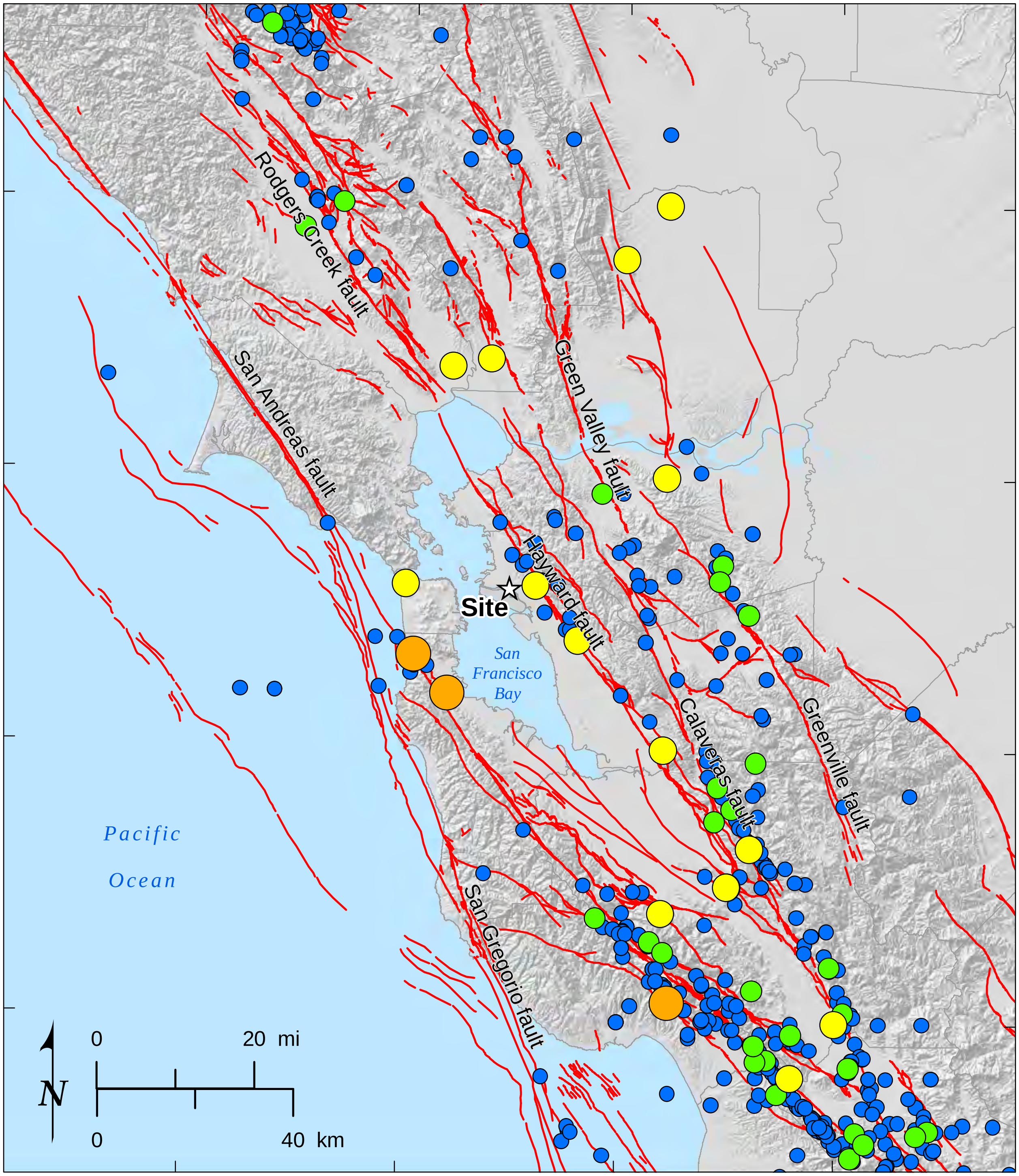
HOLOCENE

- Qhf — Alluvial fan deposits
- Qha — Alluvial deposits, undifferentiated

PLEISTOCENE

- Qbt — Bay terrace deposits

Plate-4: Quaternary Geologic Map



Legend

●	4.0 - 4.9
●	5.0 - 5.9
●	6.0 - 6.9
●	>=7.0

Seismicity by Magnitude (1910-2018 Earthquake Catalog from NCEDC and 1568-2009 Significant Earthquake data from USGS)

Faults (USGS, Quaternary Fault and Fold Database, 2017)

REGIONAL FAULT AND SEISMICITY MAP

PLATE 5

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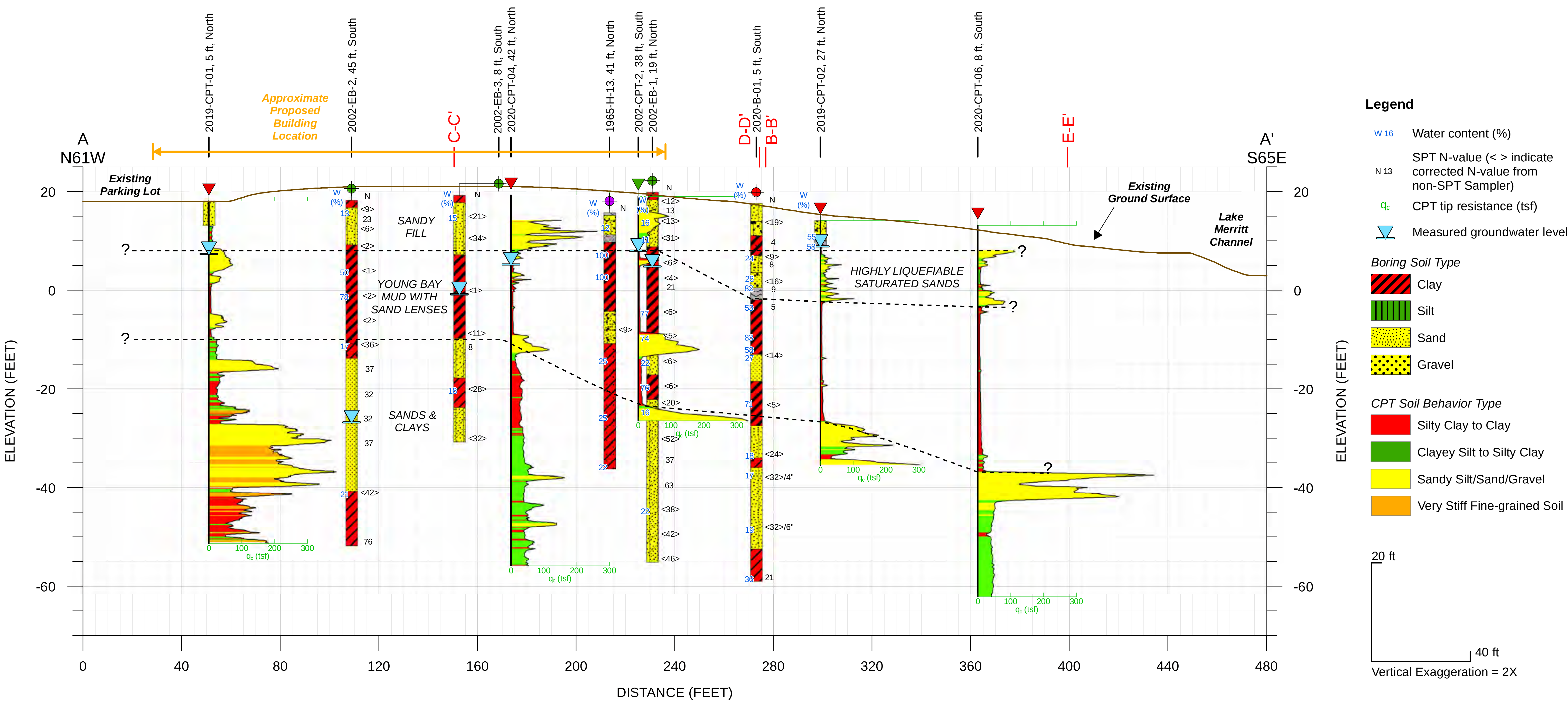

CROSS SECTION A-A'

PLATE 7

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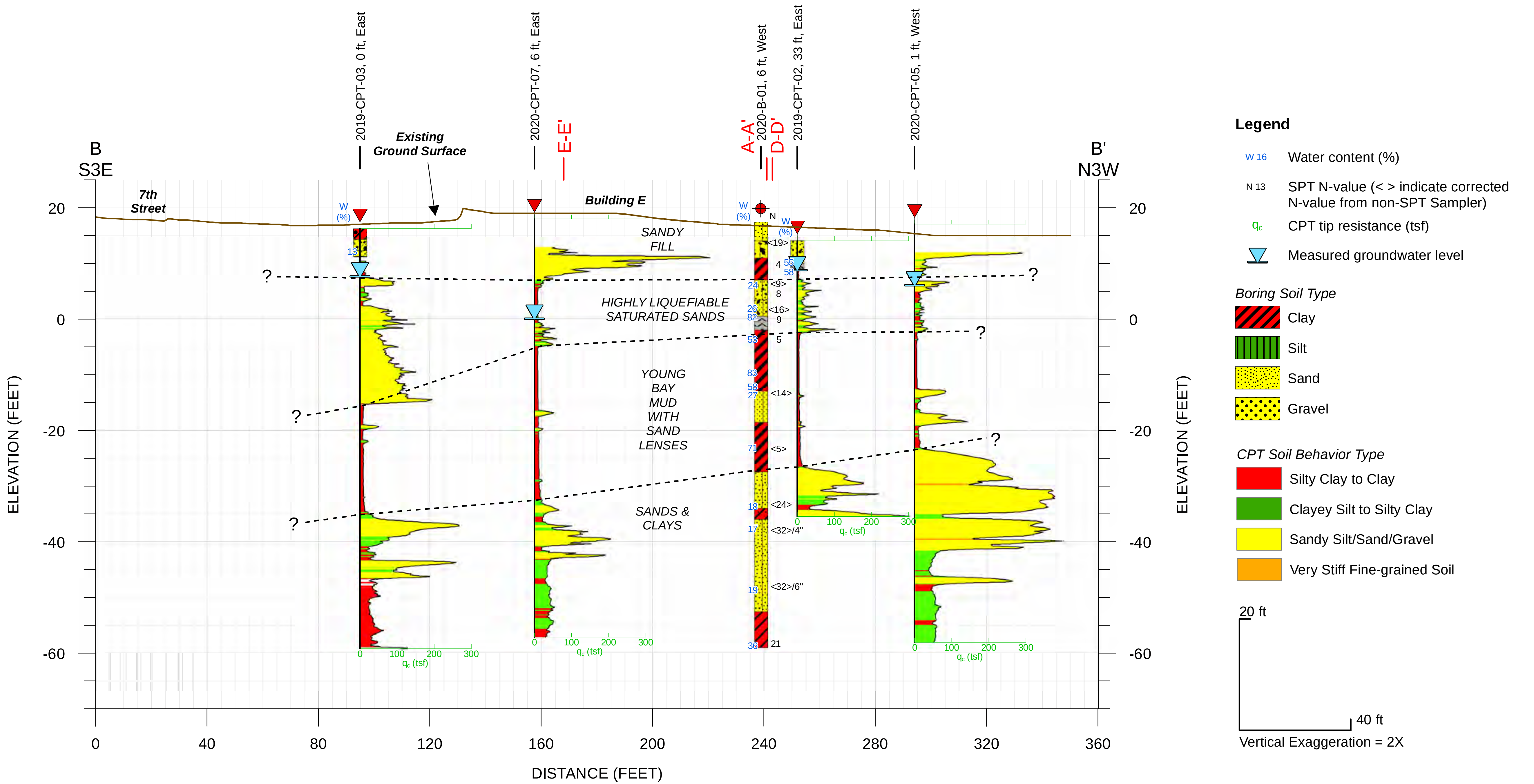

CROSS SECTION B-B'

PLATE 8

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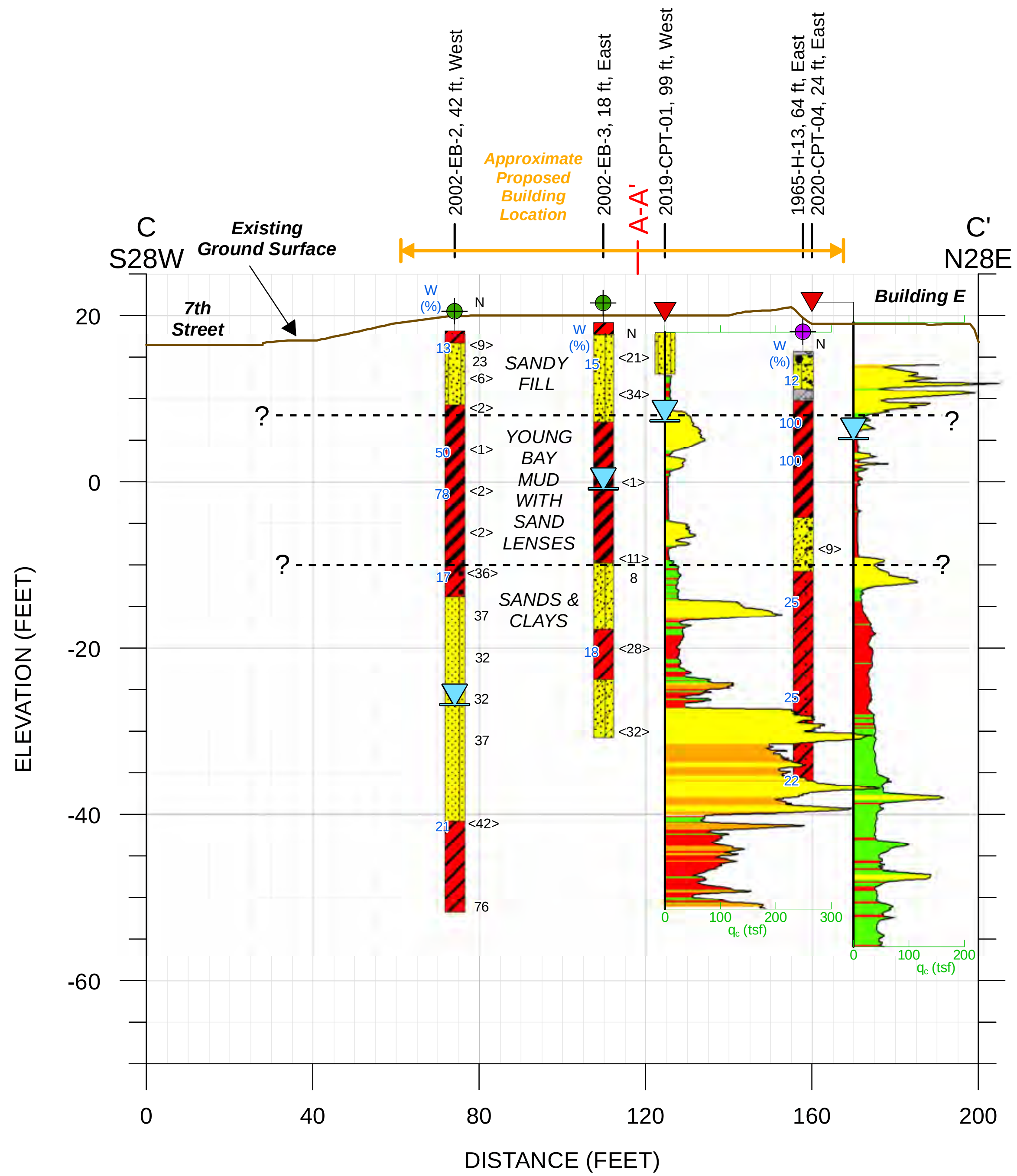

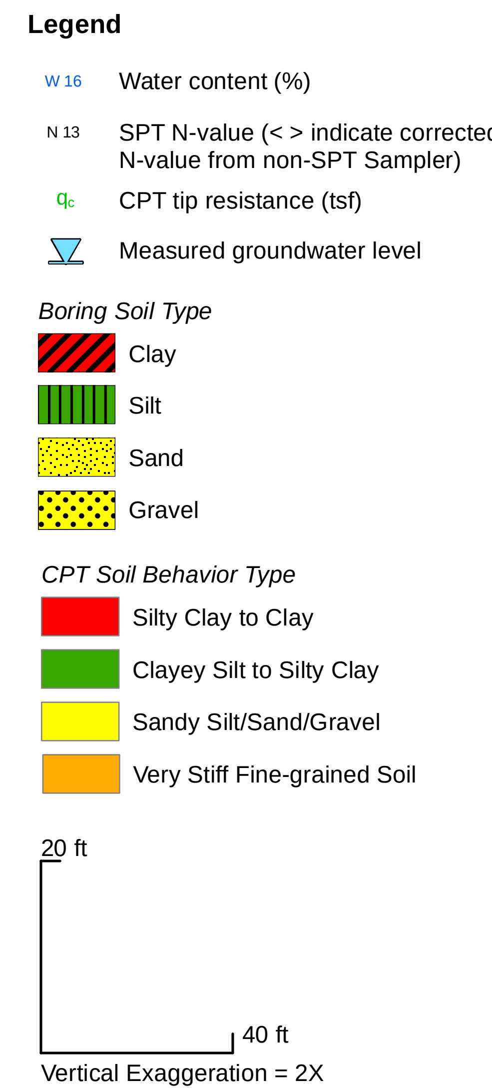

CROSS SECTION C-C'

PLATE 9

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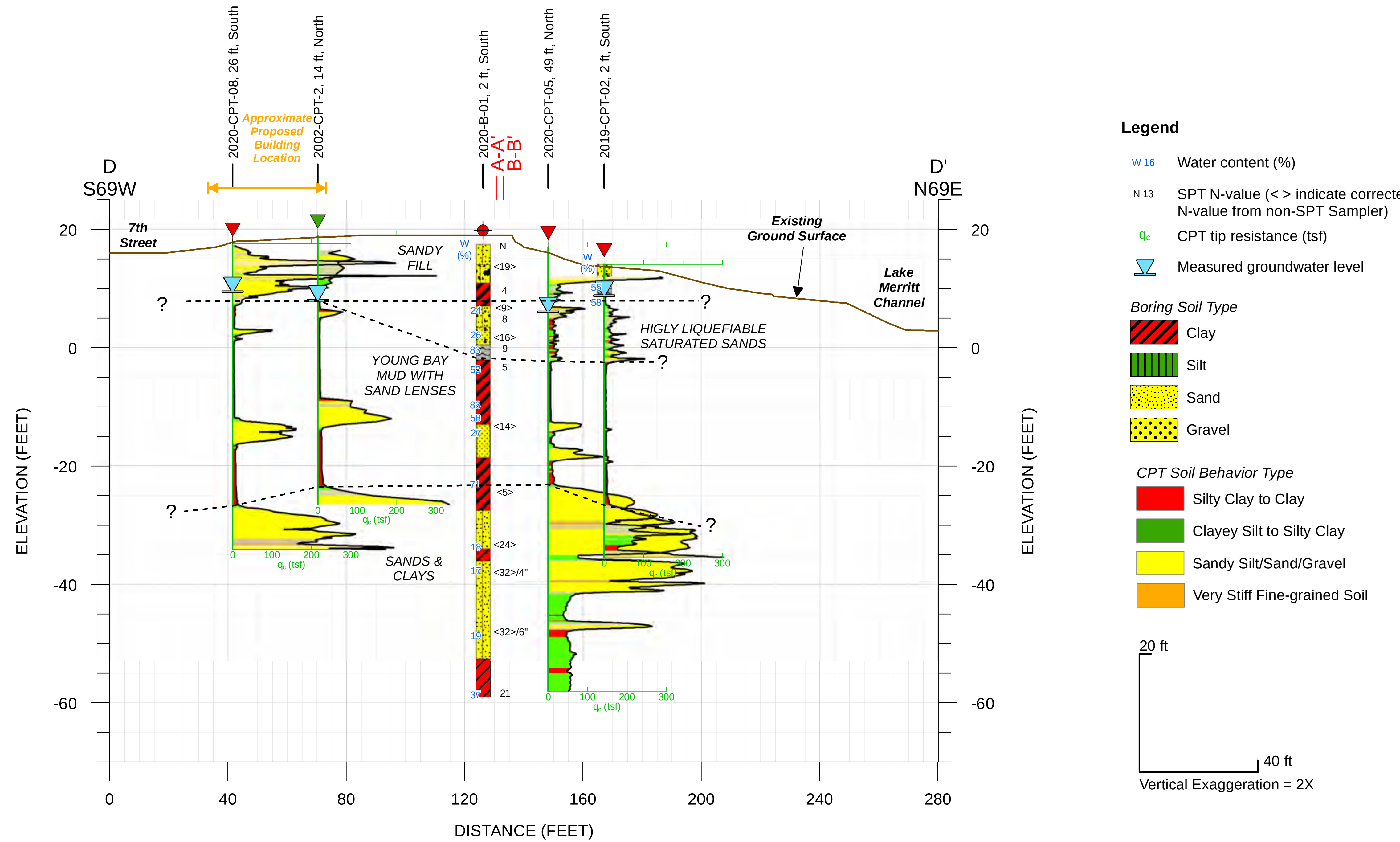
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER
OAKLAND, CALIFORNIA

FUGRO



CROSS SECTION D-D'

PLATE 10

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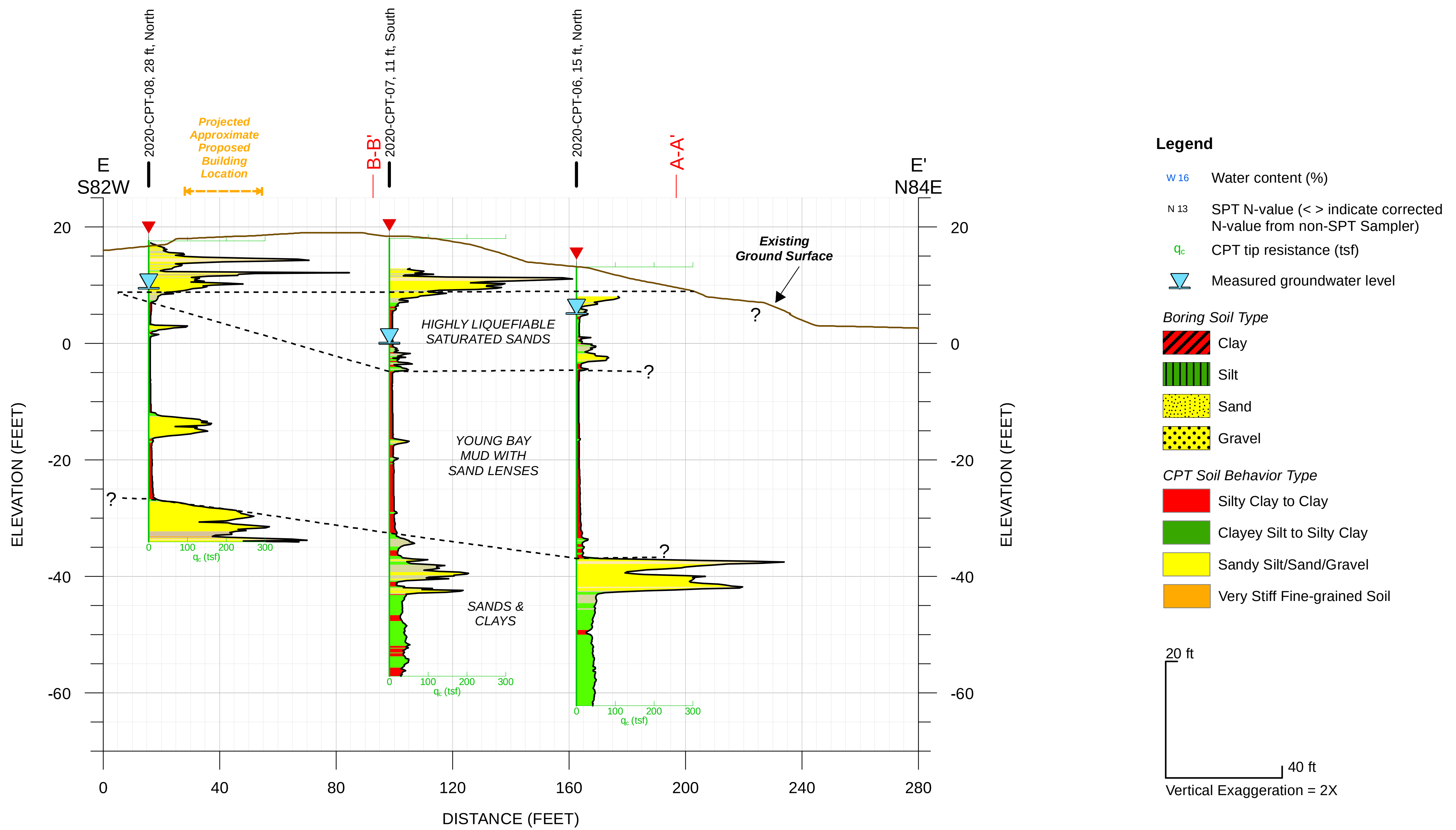
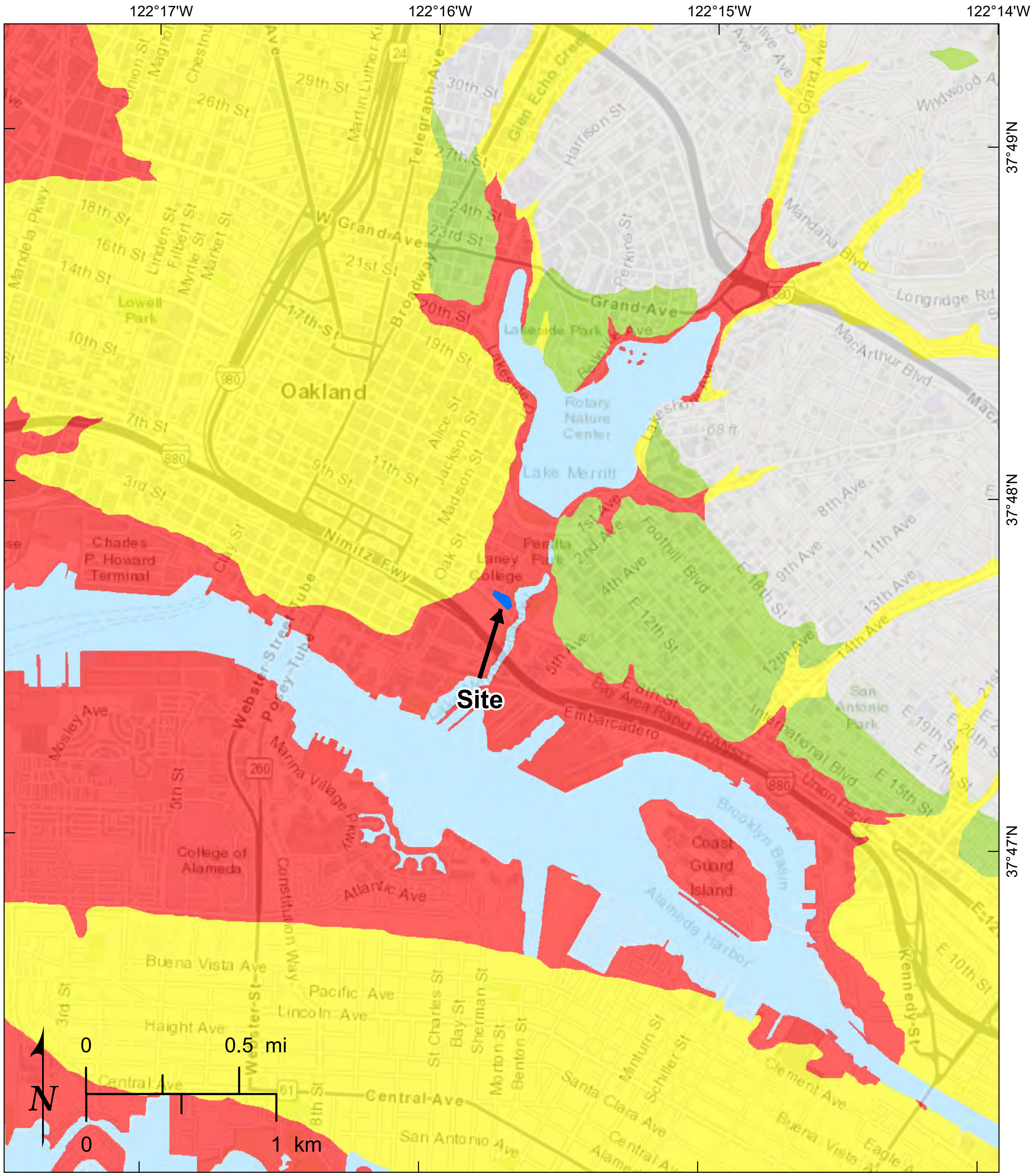
Fugro Document No. 04.72190021



CROSS SECTION E-E'

PLATE 11



Legend

Liquefaction Susceptibility (Witter et al., 2006)

- Very high
- High
- Moderate
- Low
- Very low

LIQUEFACTION SUSCEPTIBILITY

PLATE 12

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\04_Outputs\2019_05_01_GeotechReport\MXD\10_Liquefaction_Susceptibility.mxd; A.Ramirez; 6/14/2019

Legend

Historical Ground Failures (Knudsen et al., 2000)

⊠	Miscellaneous effects
✦	Ground settlement
◁▶	Lateral Spread
O	Sand boil
+	Pipeline break
✕	Cracks in streets or ground
x	Location of multiple ground effects (See corresponding symbols)
●	Geotechnical borings used in liquefaction evaluation
○	Groundwater level data
174	Number assigned to ground failure site - adapted from Youd and Hoose (1978), Tinsley and others (1998), and by Knudsen and others (2000)
—10—	Depth to historically high groundwater, in feet

HISTORICAL LIQUEFACTION SITES AND HISTORICALLY HIGH GROUNDWATER TABLE

PLATE 13

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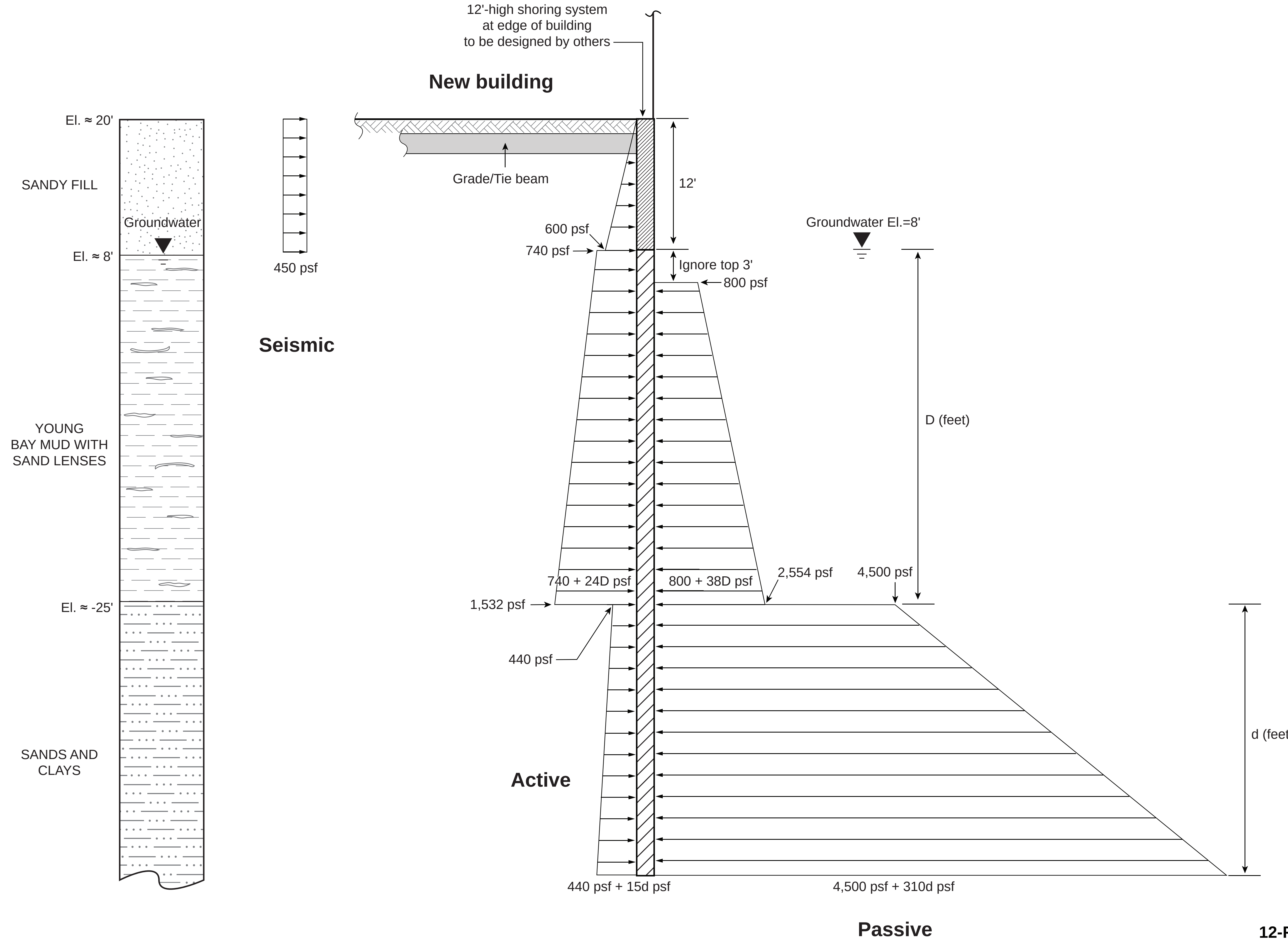
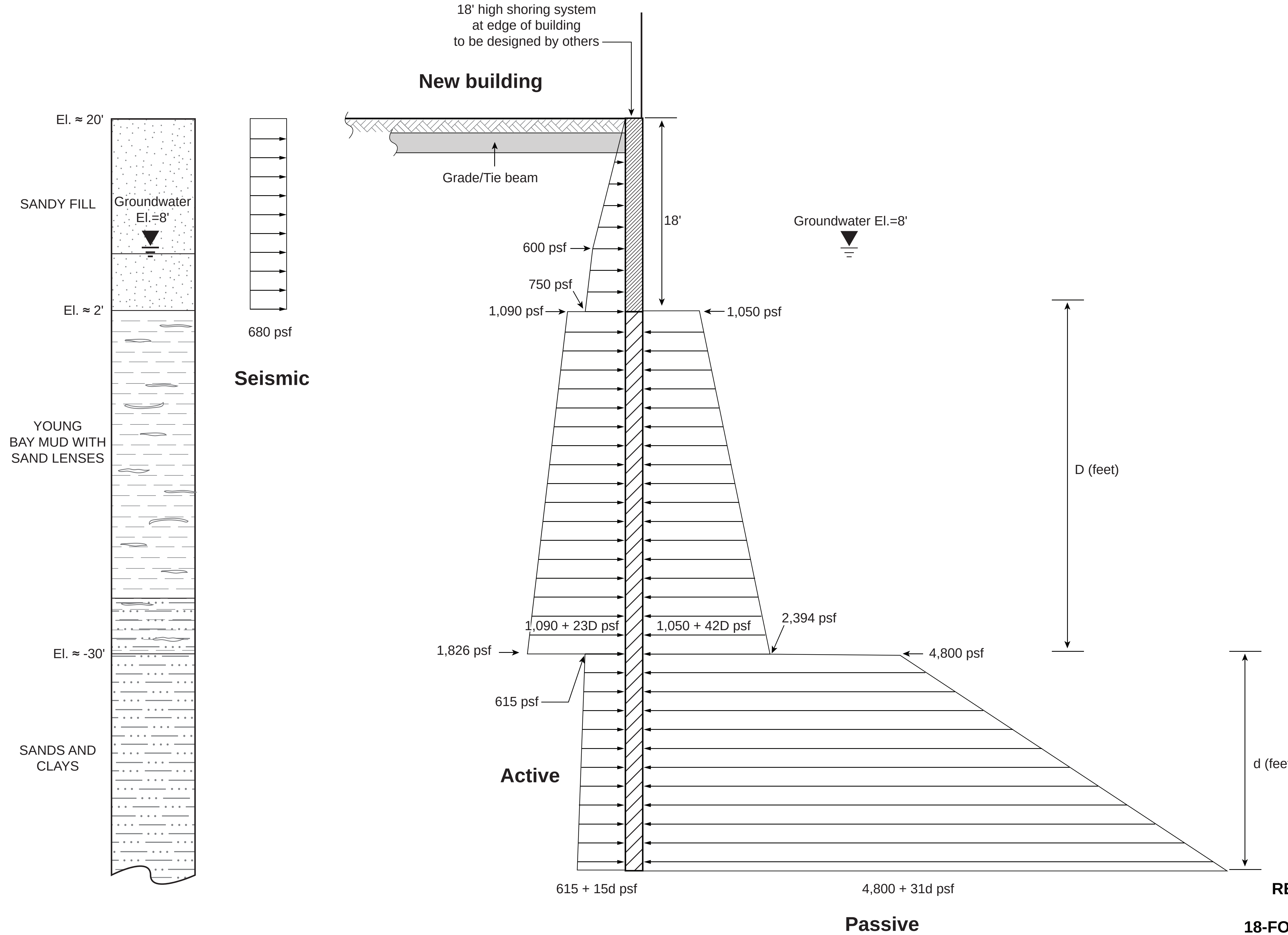


RECOMMENDED LATERAL PRESSURES FOR 12-FOOT HIGH SHORING SYSTEM

PLATE 14



RECOMMENDED LATERAL PRESSURES FOR 18-FOOT HIGH SHORING SYSTEM

PLATE 15

Supplement A

Field Explorations

CLASSIFICATION AND MATERIAL SYMBOLS

MAJOR DIVISIONS PER ASTM D2488-06			MAJOR GROUP NAMES AND MATERIAL SYMBOLS	
COARSE-GRAINED SOILS More than 50% retained on the No. 200 sieve	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	Clean gravels less than 5% fines	GW	Well-Graded GRAVEL
			GP	Poorly Graded GRAVEL
		Gravels with more than 12% fines	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SANDS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	Clean sand less than 5% fines	SW	Well-Graded SAND
			SP	Poorly Graded SAND
		Sands with more than 12% fines	SM	SILTY SAND
			SC	CLAYEY SAND
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	SILTS AND CLAYS Liquid Limit Less than 50%		ML	SILT
			CL	Lean CLAY
			OL	ORGANIC SILT
	SILTS AND CLAYS Liquid Limit Greater than 50%		MH	Elastic SILT
			CH	Fat CLAY
			OH	ORGANIC CLAY
HIGHLY ORGANIC SOILS			PT	Peat or Highly Organic Soils

Notes:
Classification of soils on the boring logs is in general accordance with ASTM D2488, or D2487 if appropriate laboratory data are available.
The geologic formation is noted in bold font at the top of interpreted interval on the boring logs.

OTHER MATERIAL SYMBOLS

- Debris or Mixed Fill
- Pavement with Aggregate Base

SAMPLER TYPE

SPT (Driven) 1-3/8" ID 2" OD	Modified California (Driven) 2-3/8" ID 3" OD	Modified California (Driven) 1-7/8" ID 2-1/2" OD
Shelby Tube (Pushed) 2-7/8" ID 3" OD	Pitcher Barrel (Rotary-cut) 2-7/8" ID	Osterberg (Piston) 2-7/8" ID
101 Geobarrel (Rotary-cut) 2-7/8" ID	Rock Core (Rotary-cut) See log for size	Vibracore (Vibrated) See log for size
Push-core (Pushed) See log for size	Collected from Auger	Other See log for details

Note: Refer to text of report for additional details or other sampler types.

BLOW COUNT

Number of blows required to drive sampler each of three 6-in. intervals, as measured in the field (uncorrected). An SPT hammer (140 lb., falling 30-in.) was used unless otherwise noted on the boring log. For example:

Blow Count	Description
5 7 8	5, 7, and 8 blows for first, second, and third interval, respectively.
35 50/3"	35 blows for the first interval. 50 blows for the first 3 inches of the second interval. Lack of third value implies that driving was stopped 3 inches into the second interval.
WOH WOH 5	"WOH" indicates that the weight of the hammer was sufficient to advance the sampler over the first two intervals. 5 blows were required to advance the sampler over the third interval.

N-VALUE

The N-Value represents the blowcount for the last 12 inches of the sample drive if three 6-inch intervals were driven. N-value presented is independant of impact energy. If 50 hammer blows were insufficient to drive through either the second or the third interval, the total number of blows and total length driven are reported (excluding the first interval). "ref" (refusal) indicates that 50 blows were insufficient to drive through the first 6-inch interval.

Parenthesis indicate that an approximate correction has been applied for non-SPT drive samplers. For example, a factor of 0.63 is commonly used to adjust blow counts obtained using a 3-inch outside diameter modified California sampler to correspond to Standard Peneteration Test.

UNDRAINED SHEAR STRENGTH

A value of undrained shear strength is reported. The value is followed by a letter code indicating the type of test that was performed, as follows:

U - Unconfined Compression
Q - Unconsolidated Undrained Triaxial
T - Torvane
P - Pocket Penetrometer
M - Miniature Vane
F - Field Vane
R - R-value

OTHER TESTS

Field or laboratory tests without a dedicated column on the boring log are reported in the Other Tests column. A letter code is used to indicate the type of test. For certain tests, a value representing the test result is also provided. Typical letter codes are as follows. Additional codes may be used. Refer to the report text and the laboratory testing results for additional information.

k - Permeability (cm/s)
Consol - Consolidation
Gs - Specific Gravity
MA - Particle Size Analysis
EI - Expansion Index
OVM - Organic Vapor Meter

WATER LEVEL SYMBOLS

- Initial water level
- Final water level
- Seepage encountered

INCREASING MOISTURE CONTENT

Dry
Moist
Wet

CONSISTENCY OF COHESIVE SOIL

CONSISTENCY	UNDRAINED SHEAR STRENGTH (KIPS PER SQUARE FOOT)
Very Soft	< 0.25
Soft	0.25 to 0.50
Medium Stiff	0.50 to 1.0
Stiff	1.0 to 2.0
Very Stiff	2.0 to 4.0
Hard	> 4.0

Note: In absence of test data, consistency has been estimated based on manual observation.

APPARENT DENSITY OF COHESIONLESS SOIL

APPARENT DENSITY	N-VALUE
Very Loose	0 to 4
Loose	5 to 9
Medium Dense	10 to 29
Dense	30 to 49
Very Dense	> 49

TERMS AND SYMBOLS USED ON BORING LOGS PLATE A-1

Peralta Community College District
Project No. 04.72190021

FUGRO

DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	LOCATION: N 37.795163+/- E 122.262754+/- WGS84 SURFACE EL: 18.0 ft +/- (rel. NAVD88 datum) MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
						FILL: 0 TO 6 FEET SILTY SAND (SM): loose to medium dense, light brown, dry, fine-grained, silty							
5						Change color to mottled gray brown , trace coarse-grained, few gravel (fine, subangular to subrounded), few brick fragments and organics							
						NOTES: 1. Terms and symbols defined on Plate A-1.							

BORING DEPTH: 6.0 ft
BACKFILL: Cement Grout
DEPTH TO WATER: Not Encountered
FIELDWORK DATE: March 29, 2019
DRILLING METHOD: 3-in dia Hand Auger

HAMMER TYPE: N/A
RIG TYPE: N/A
DRILLED BY: Fugro
LOGGED BY: F De Paola
CHECKED BY: T Chen

LOG OF BORING NO. 2019-CPT-01
Laney College Library Learning Resource Center
Oakland, California

LOCATION:

N 37.794900+/- E 122.261959+/- WGS84
SURFACE EL: 14.1 ft +/- (rel. NAVD88 datum)

DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
						FILL: 0 TO 6 FEET SILTY SAND with GRAVEL (SM): medium dense, light gray, dry, fine- to medium-grained, trace coarse-grained, silty, with gravel (fine to coarse, subangular to subrounded)							
5						PEAT (PT): very soft to soft, black, dry, with organic odor.		55					
						Fat CLAY (CH): soft, gray, moist, trace sand (fine-grained), trace small shell fragments, few organics, with strong organic odor		58					

NOTES:
1. Terms and symbols defined on Plate A-1.

BORING DEPTH: 6.0 ft
BACKFILL: Cement Grout
DEPTH TO WATER: Not Encountered
FIELDWORK DATE: March 29, 2019
DRILLING METHOD: 3-in dia Hand Auger

HAMMER TYPE: N/A
RIG TYPE: N/A
DRILLED BY: Fugro
LOGGED BY: F De Paola
CHECKED BY: T Chen

LOG OF BORING NO. 2019-CPT-02
Laney College Library Learning Resource Center
Oakland, California

Peralta Community College District
Project No. 04.72190021

FUGRO

LOCATION:

N 37.794463+/- E 122.262030+/- WGS84
SURFACE EL: 16.3 ft +/- (rel. NAVD88 datum)

DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
						FILL: 0 TO 6 FEET							
						Lean CLAY with GRAVEL (CL): soft to medium stiff, mottled gray brown, dry, with gravel (fine to coarse, subangular to rounded), few sand (fine- to coarse-grained)							
						CLAYEY GRAVEL with SAND (GC): loose, mottled gray brown, dry to moist, fine to coarse, subangular to rounded, clayey, with sand (fine- to coarse-grained)		13	20				
5						CLAYEY SAND (SC): loose to medium dense, dark brown, moist, fine- to coarse-grained, clayey , few gravel (fine, subangular to subrounded)							
						NOTES: 1. Terms and symbols defined on Plate A-1.							

BORING DEPTH: 5.0 ft
BACKFILL: Cement Grout
DEPTH TO WATER: Not Encountered
FIELDWORK DATE: March 29, 2019
DRILLING METHOD: 3-in dia Hand Auger

HAMMER TYPE: N/A
RIG TYPE: N/A
DRILLED BY: Fugro
LOGGED BY: F De Paola
CHECKED BY: T Chen

LOG OF BORING NO. 2019-CPT-03
Laney College Library Learning Resource Center
Oakland, California

Peralta Community College District
Project No. 04.72190021

FUGRO

LOCATION:

N 37.794856+/- E 122.262089+/- WGS88
SURFACE EL: 17.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
15							**FILL: 0 TO 19.5 FEET** SILTY SAND with GRAVEL (SM): loose to medium dense, brown, dry, fine- to medium-grained, trace coarse-grained, silty, with gravel (fine to coarse, angular to subangular)							
			S1	18 17 13	(19)	14"/18"	SILTY GRAVEL with SAND (GM): medium dense, mottled gray brown, dry, fine to coarse, angular to subrounded, sandy (fine- to coarse-grained), silty, trace clay							
	5		S3	50	ref	6"/6"	with rock fragments up to 2", dry to moist at 5'							
10			S4	2 2 2	4	18"/18"	Fat CLAY with SAND (CH): medium stiff, mottled black green dark gray, dry, with sand (fine- to coarse-grained), trace organics, trace glass fragments, with organic odor							
	10		S5	14 10 5	(9)	18"/18"	SILTY SAND with GRAVEL (SM): medium dense, mottled brown gray, dry, fine- to coarse-grained, silty, with gravel (fine to coarse, angular to subangular), a large brick fragment at 11'	91	24	21				MA
5			S6	3 3 5	8	16"/18"	with abundant wood chips at 12' to 13', trace glass fragments, moist below 12.5'							
	15		S7	5 6 19	(16)	18"/18"	Poorly-graded SAND with SILT and GRAVEL (SP-SM): medium dense, mottled brown gray, moist, fine- to coarse-grained, with silt, with abundant wood chips, with brick and glass fragments, trace clay chunks	95	26	6				MA Organic = 5%
0			S8	10 4 5	9	16"/18"	samll rock fragments at 16.5' to 17'		82					Organic = 21.2%
							ORGANIC CLAY with SAND (OH): soft to medium stiff, mottled brown dark gray, moist, with peat, with sand (fine- to coarse-grained), trace gravel (fine, angular to subangular), few wood chips							
	20		S9	3 3 2	5	10"/18"	**NATIVE: 19.5 TO 76.5 FEET** Fat CLAY (CH): medium stiff, gray, moist, trace wood chips		53					Organic = 6.6%
-5														
	25		S10	50 psi 100 psi		30"/30"	very soft to soft, trace wood chips	52	83					Consol
-10														
	30		S11	5 13 9	(14)	18"/18"	soft to medium stiff, trace sand (fine-grained), trace rootlets, a 2" rock fragment at 30'	69	58	93	73	43	0.5 Q	MA
							Poorly-graded SAND with SILT (SP-SM): medium dense, gray, wet, fine- to medium-grained, with silt, trace small shell fragments	94	27	8				MA
-15														
	35						3" rock fragment at 35'							
			S12	100 psi		30"/30"	Fat CLAY (CH): soft to medium stiff, gray, moist							
-20														

Continued

BORING DEPTH: 76.5 ft
BACKFILL: Cement Grout
DEPTH TO WATER: Not Estabilished
FIELDWORK DATE: January 7, 2020
DRILLING METHOD: 4-in. dia. Solid Stem Auger/Rotary Wash

HAMMER TYPE: Automatic Trip
RIG TYPE: CME 75 Track
DRILLED BY: Geo-Ex
LOGGED BY: T Chen
CHECKED BY: A Johan

LOG OF BORING NO. 2020-B-01
Laney College Library Learning Resource Center
Oakland, California

LOCATION:

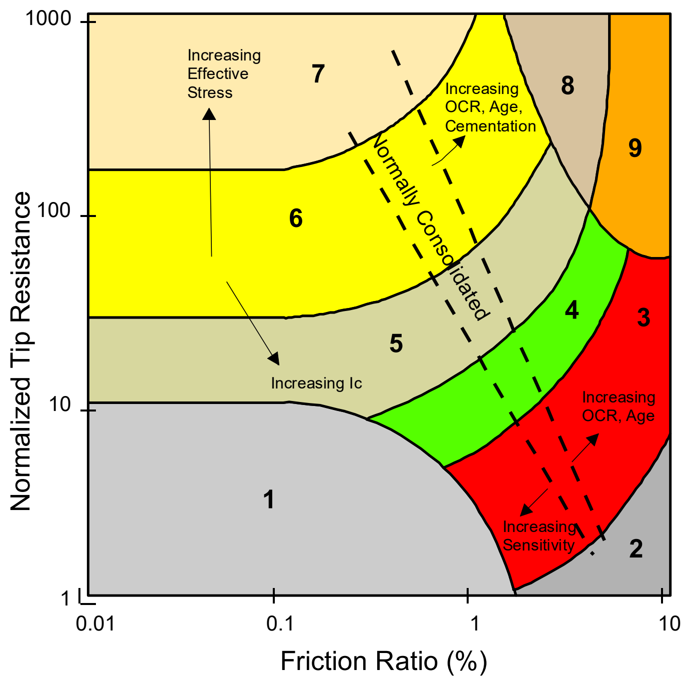
N 37.794856+/- E 122.262089+/- WGS84
SURFACE EL: 17.5 ft +/- (rel. NAVD88 datum)

ELEVATION, ft	DEPTH, ft	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u, ksf	OTHER TESTS
		S13	0 2 6	(5)	18"/18"	medium stiff	59	71				0.7 Q	
-25													
	45	S14	200 psi 650 psi		0"/15"	SILTY SAND (SM): medium dense to dense, gray, wet, fine- to medium-grained, silty							
-30													
	50	S15	26 24 14	(24)	18"/18"		112	18	16				MA
						SANDY Lean CLAY (CL): very stiff, mottled gray yellowish brown, moist, sandy (fine- to medium-grained)							
-35													
						SILTY SAND (SM): dense to very dense, gray, wet, fine- to medium-grained, silty, trace shell fragments							
	55	S16	49 50/4"	(32)/4"	10"/10"		116	17	17				MA
						very dense, fine- to medium-grained, with coarse-grained, with silt, few gravel (fine, angular to subangular)							
-40													
	60												
-45													
	65	S17	35 50	(32)/6"	12"/12"			19	19				
-50													
	70					Lean CLAY (CL): very stiff to hard, light brown, moist							
-55													
	75	S18	7 10 11	21	18"/18"			37					

NOTES:
1. Terms and symbols defined on Plate A-1.

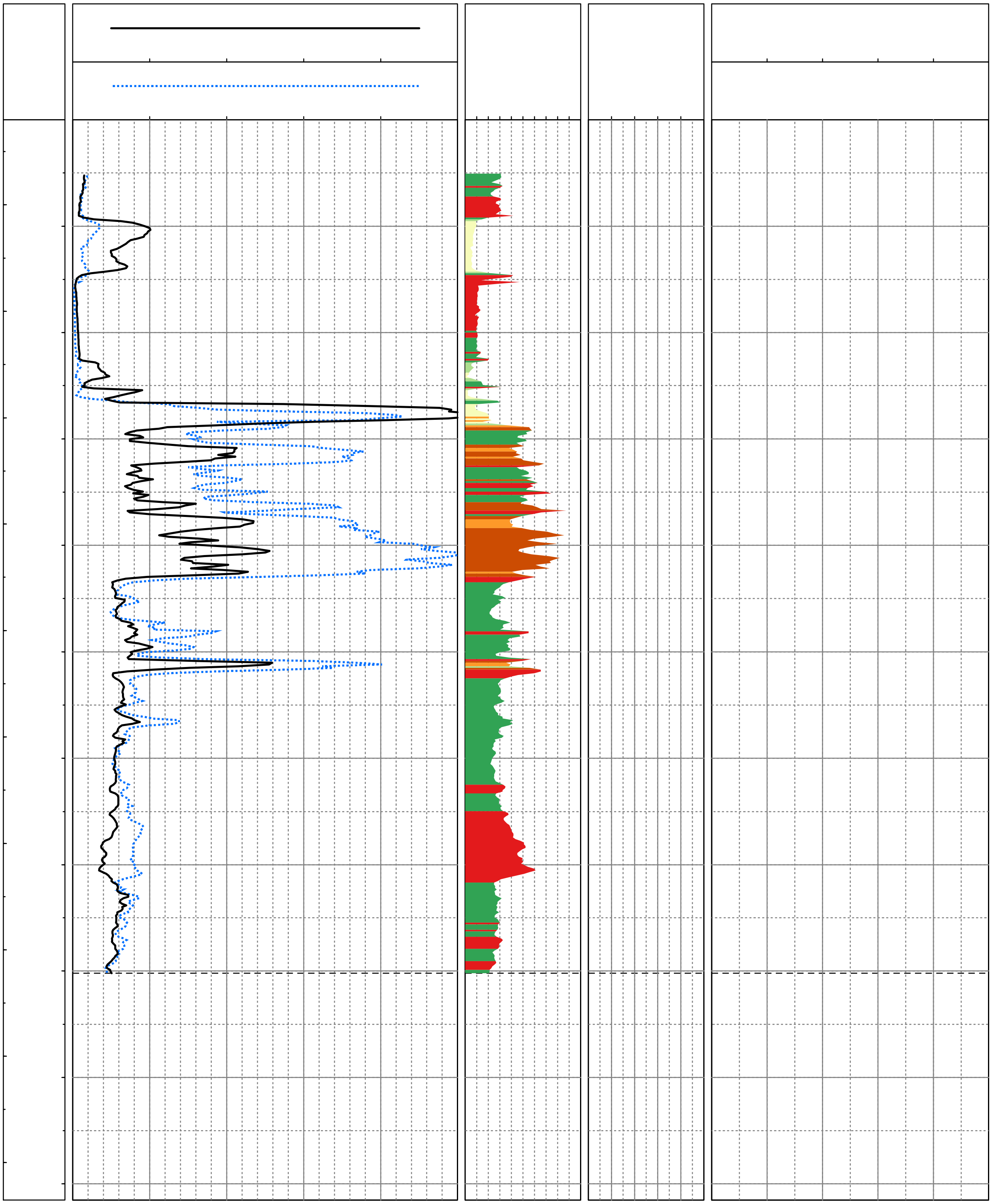
LOG OF BORING NO. 2020-B-01
Laney College Library Learning Resource Center
Oakland, California

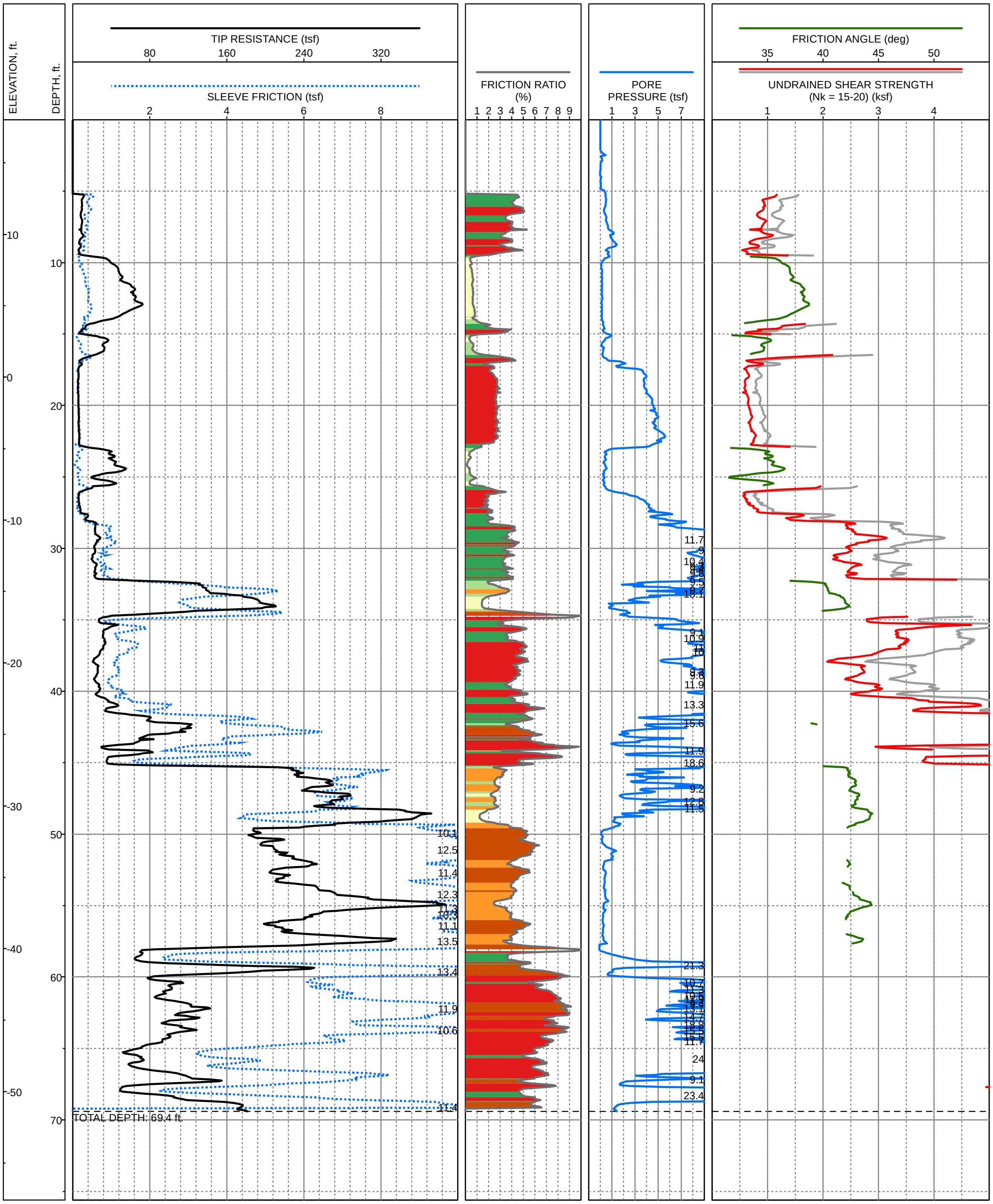
CPT CORRELATION CHART
(Robertson 1990)



Zone	Soil Behavior Type
1	Sensitive Fine-grained
2	Organic Soils, Peats
3	Clays - Clay to Silty Clay
4	Silt Mixtures - Clayey Silt to Silty Clay
5	Sand Mixtures - Silty Sand to Sandy Silt
6	Sands - Clean Sand to Silty Sand
7	Gravelly Sand to Sand
8	Very Stiff Sand to Clayey Sand
9	Very Stiff Fine-Grained

Plate A-7: Key to CPT Interpretation

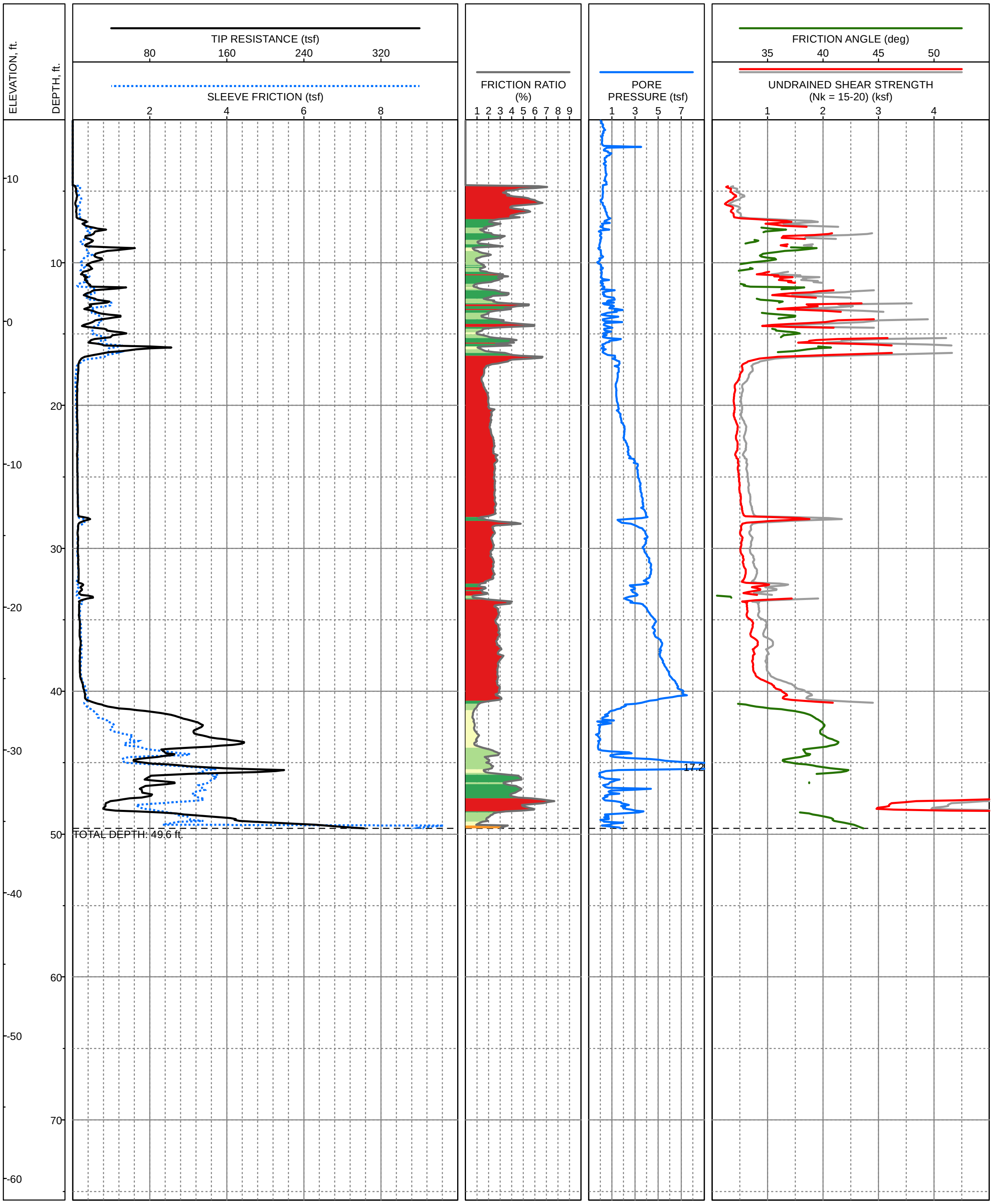




LOCATION: E6,052,365, N2,116,794, NAD83 SP CA Z3 FT
SURFACE EL: 18ft +/-
COMPLETION DEPTH: 69.4ft
TESTDATE: 3/29/2019

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.59

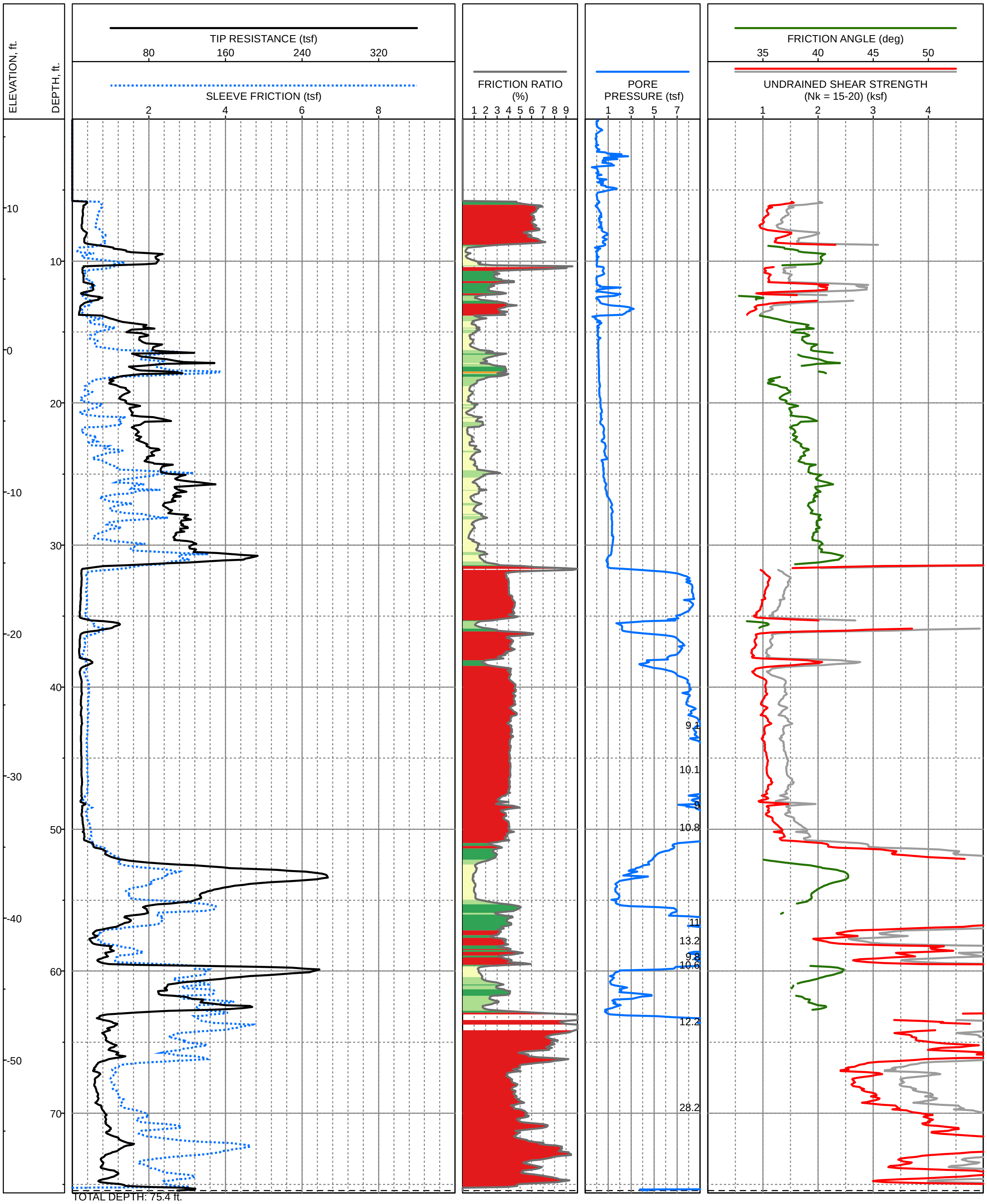
PLATE A-8: LOG OF 2019-CPT-01



LOCATION: E6,052,593, N2,116,694, NAD83 SP CA Z3 FT
SURFACE EL: 14ft +/-
COMPLETION DEPTH: 49.6ft
TESTDATE: 3/29/2019

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.59

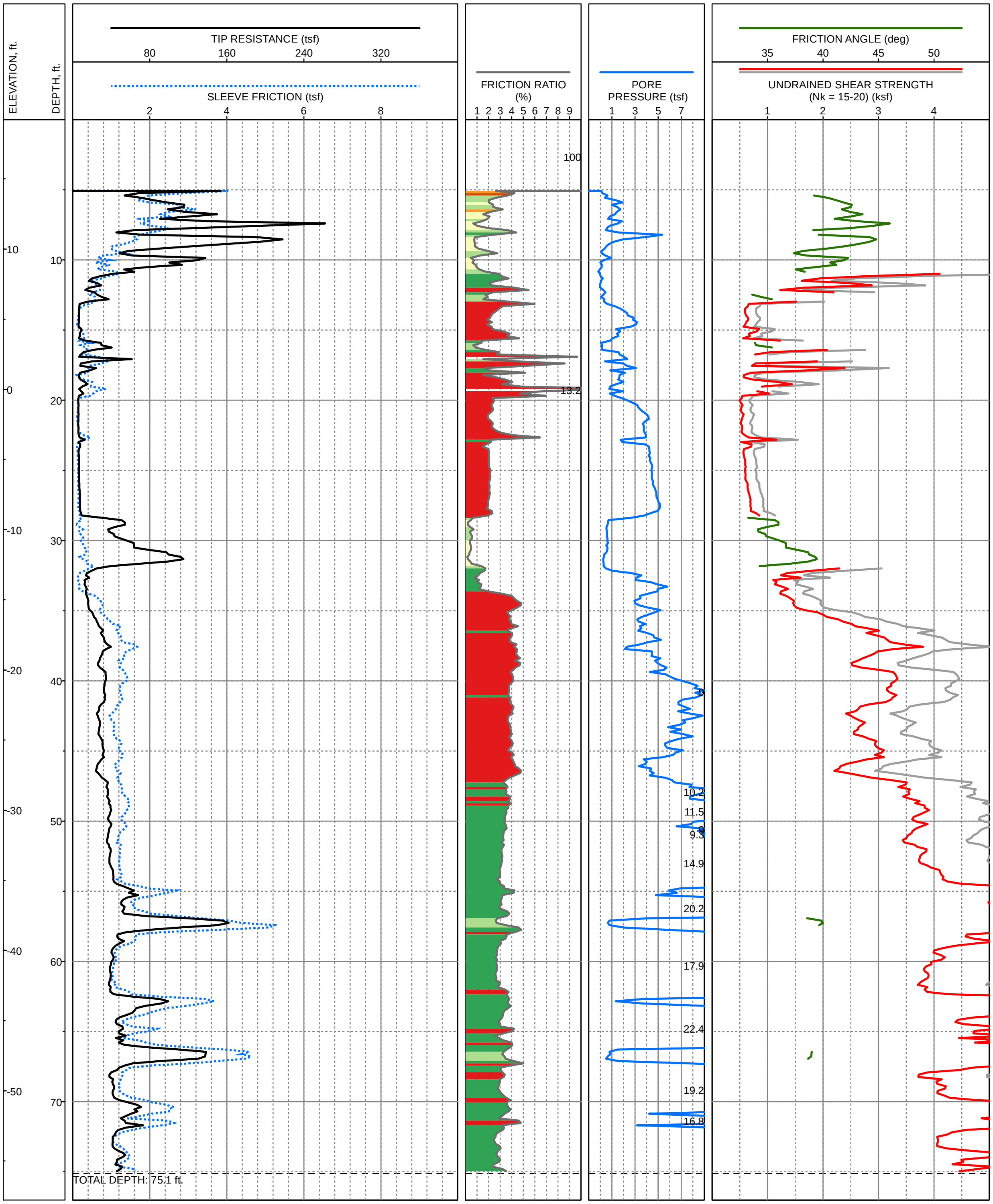
PLATE A-9: LOG OF 2019-CPT-02



LOCATION: E6,052,570, N2,116,535, NAD83 SP CA Z3 FT
SURFACE EL: 16ft +/-
COMPLETION DEPTH: 75.4ft
TESTDATE: 3/29/2019

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.59

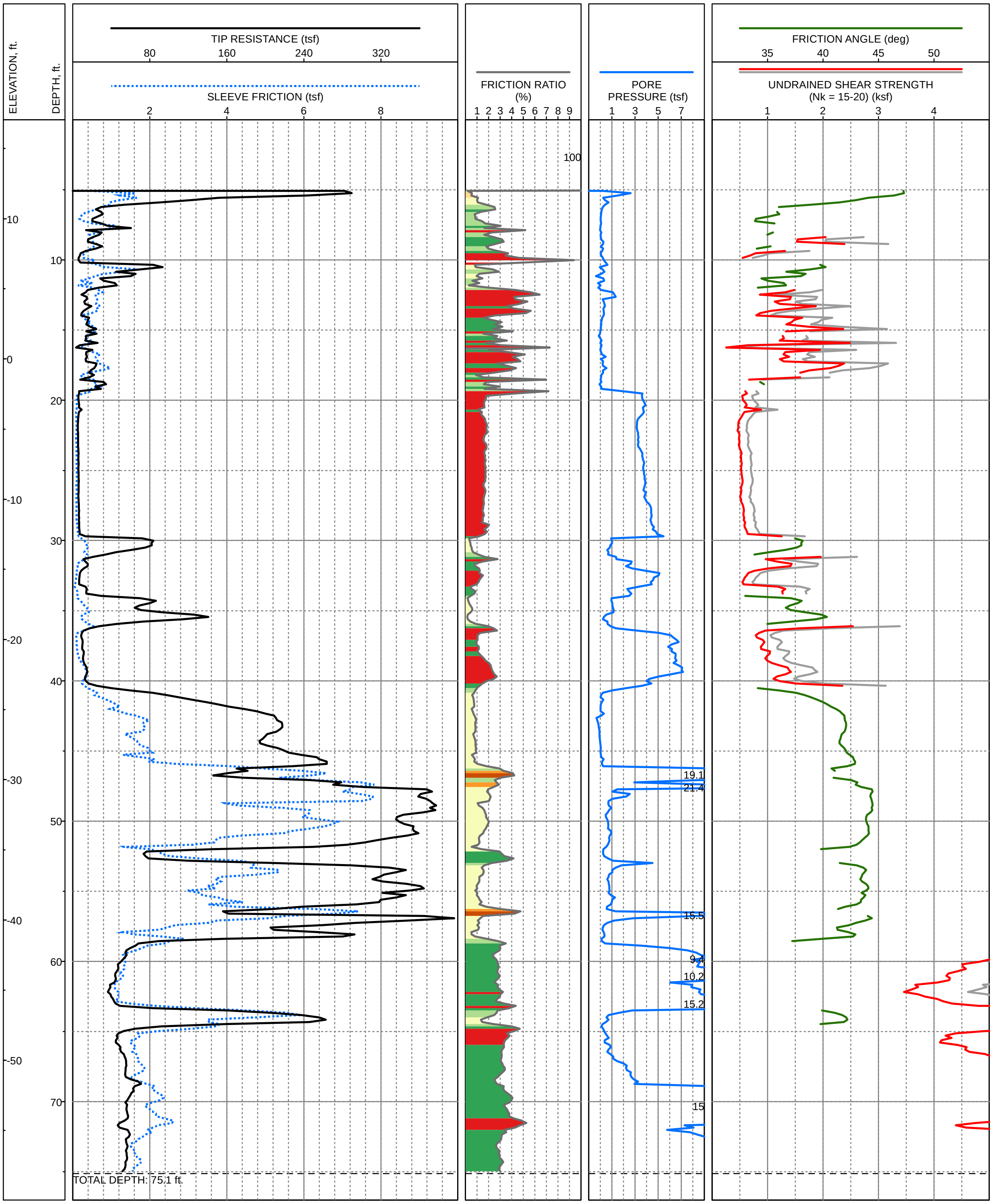
PLATE A-10: LOG OF 2019-CPT-03



LOCATION: E6,052,490, N2,116,767, NAD83 SP CA Z3 FT
SURFACE EL: 19.2ft
COMPLETION DEPTH: 75.1ft
TESTDATE: 1/3/2020

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.80

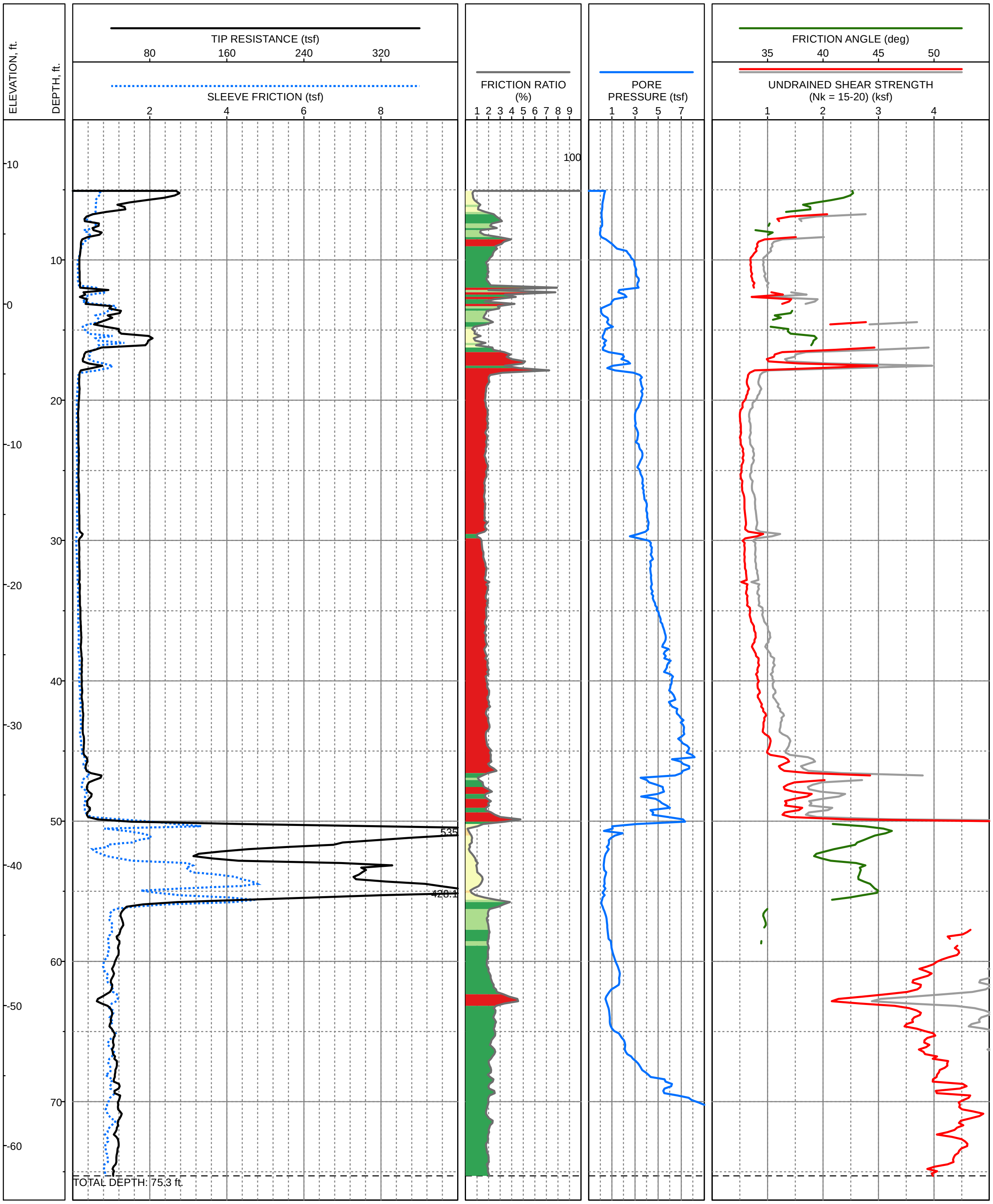
PLATE A-11: LOG OF 2020-CPT-04



LOCATION: E6,052,557, N2,116,734, NAD83 SP CA Z3 FT
SURFACE EL: 17.1ft
COMPLETION DEPTH: 75.1ft
TESTDATE: 1/3/2020

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.80

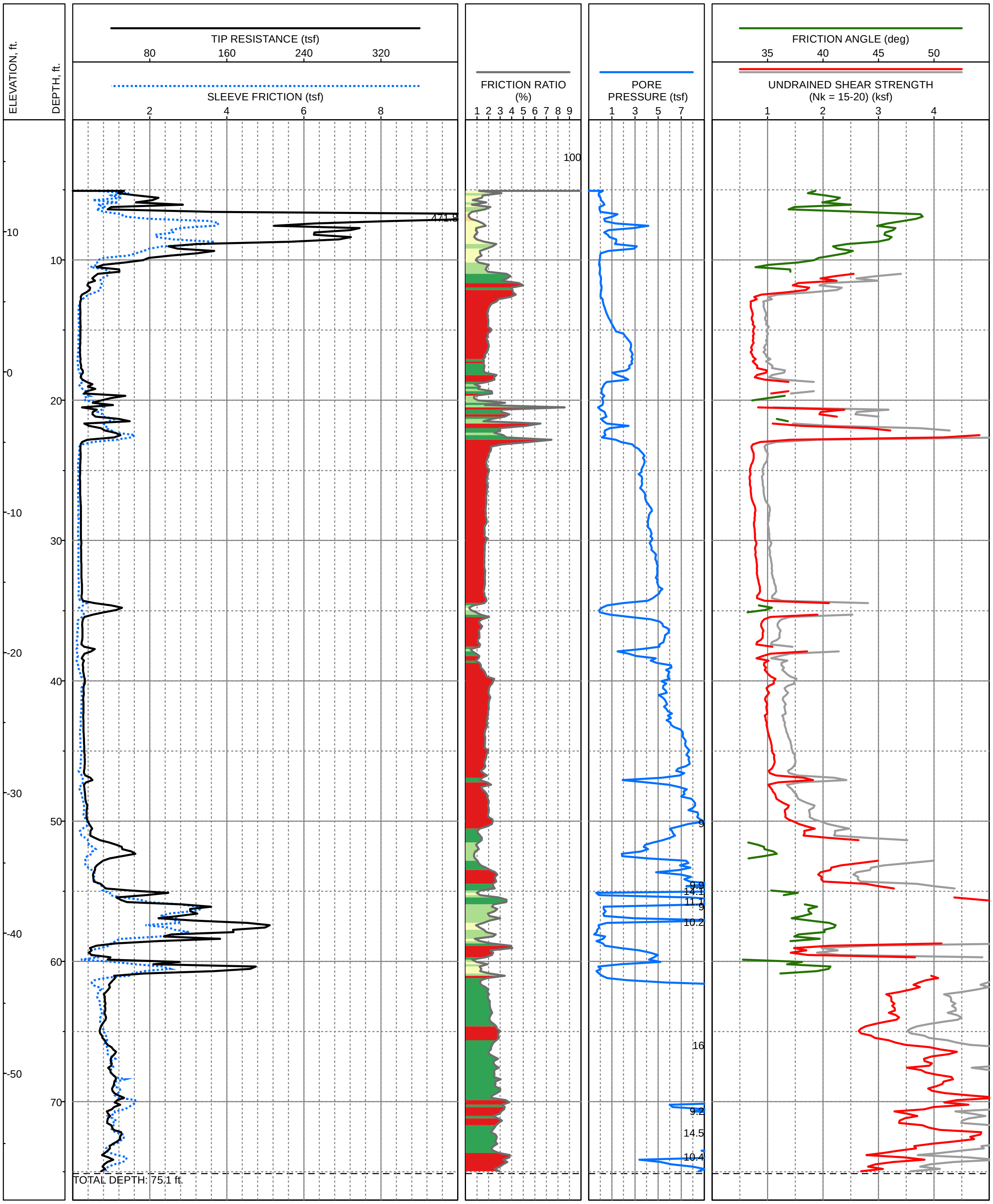
PLATE A-12: LOG OF 2020-CPT-05



LOCATION: E6,052,632, N2,116,632, NAD83 SP CA Z3 FT
SURFACE EL: 13.1ft
COMPLETION DEPTH: 75.3ft
TESTDATE: 1/3/2020

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.80

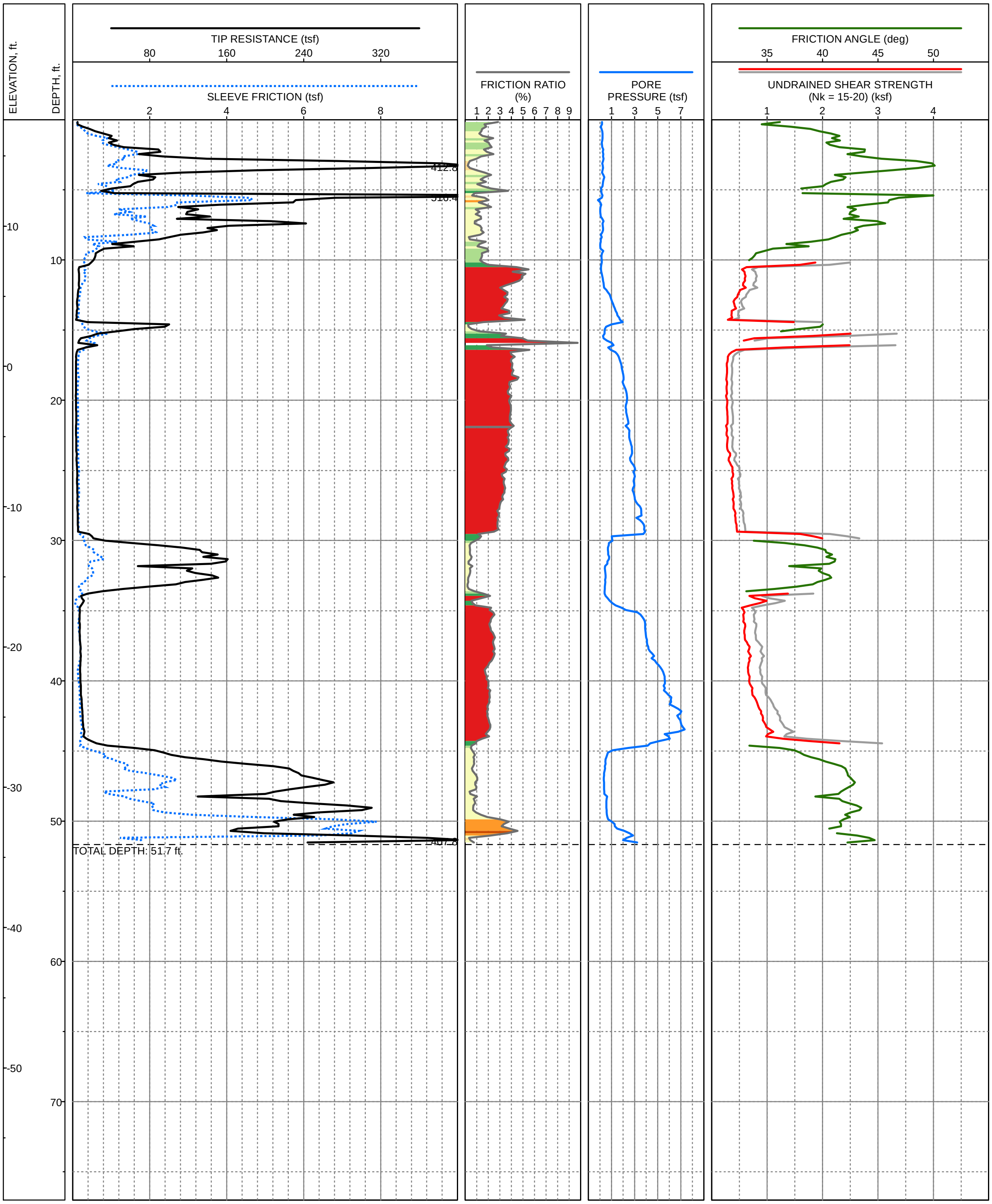
PLATE A-13: LOG OF 2020-CPT-06



LOCATION: E6,052,572, N2,116,598, NAD83 SP CA Z3 FT
SURFACE EL: 18.0ft
COMPLETION DEPTH: 75.1ft
TESTDATE: 1/3/2020

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.80

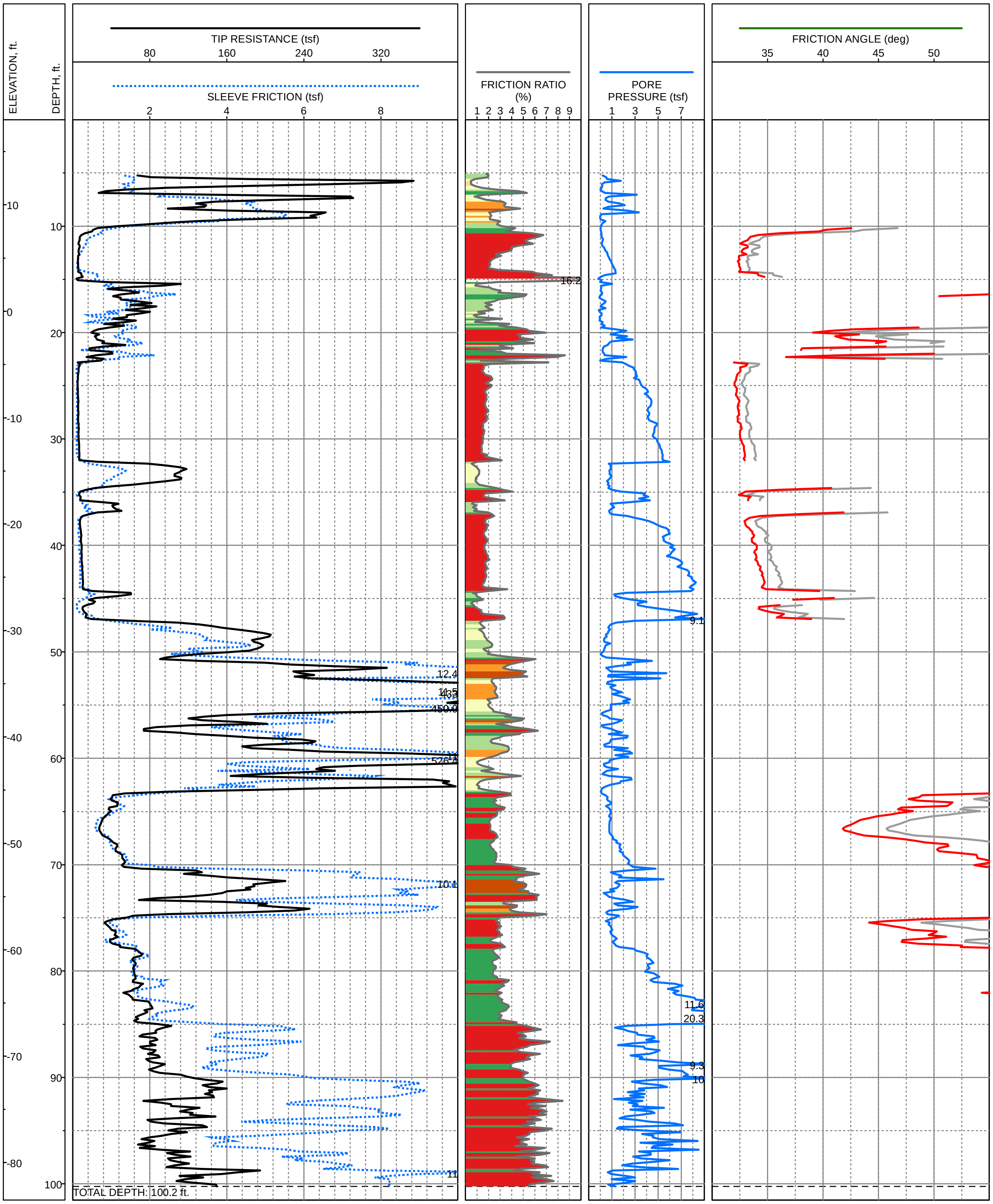
PLATE A-14: LOG OF 2020-SCPT-07



LOCATION: E6,052,485, N2,116,625, NAD83 SP CA Z3 FT
SURFACE EL: 17.6ft
COMPLETION DEPTH: 51.7ft
TESTDATE: 1/2/2020

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.80

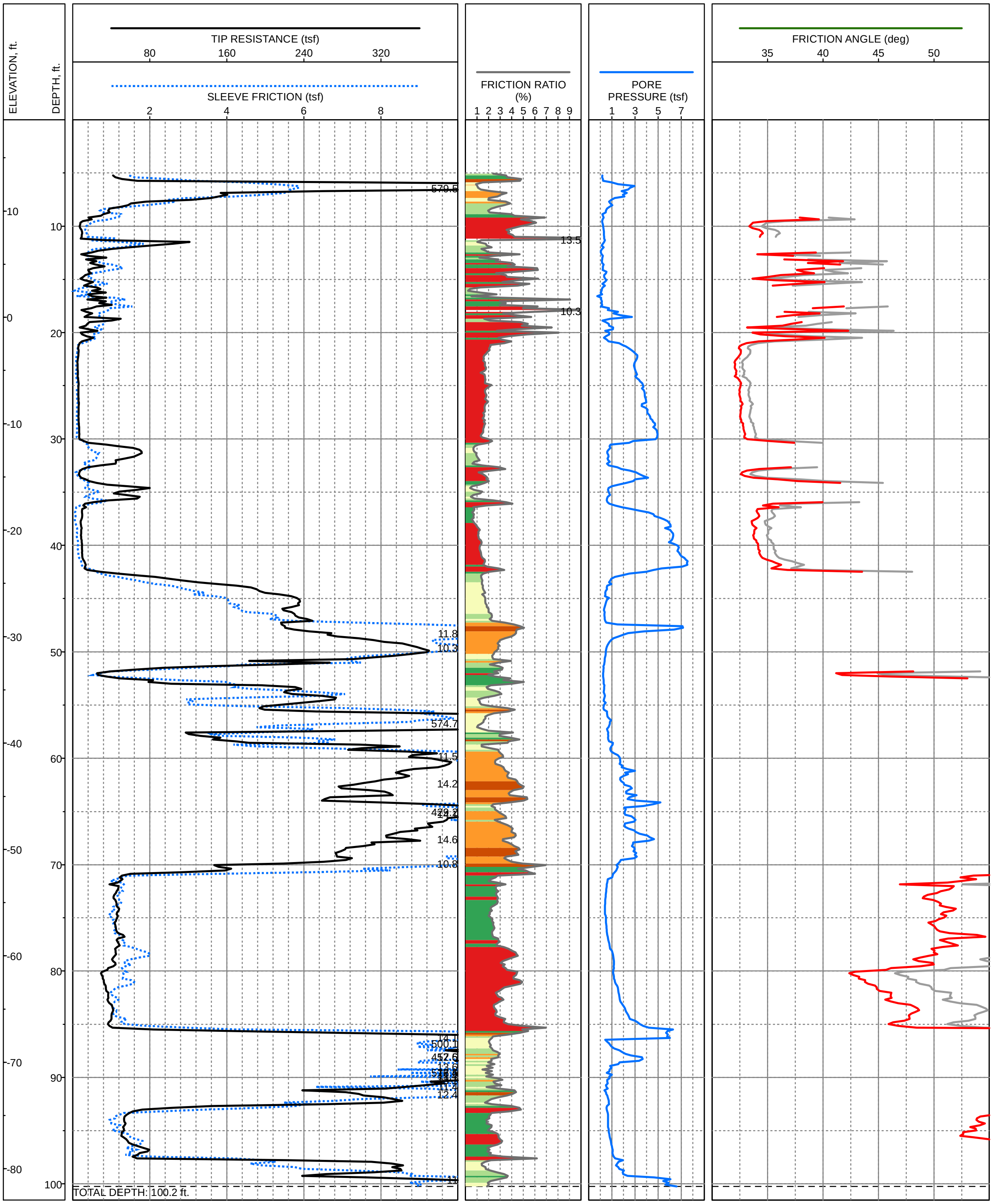
PLATE A-15: LOG OF 2020-CPT-08



LOCATION: E6,052,530, N2,116,637, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 18.0ft
COMPLETION DEPTH: 100.2ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

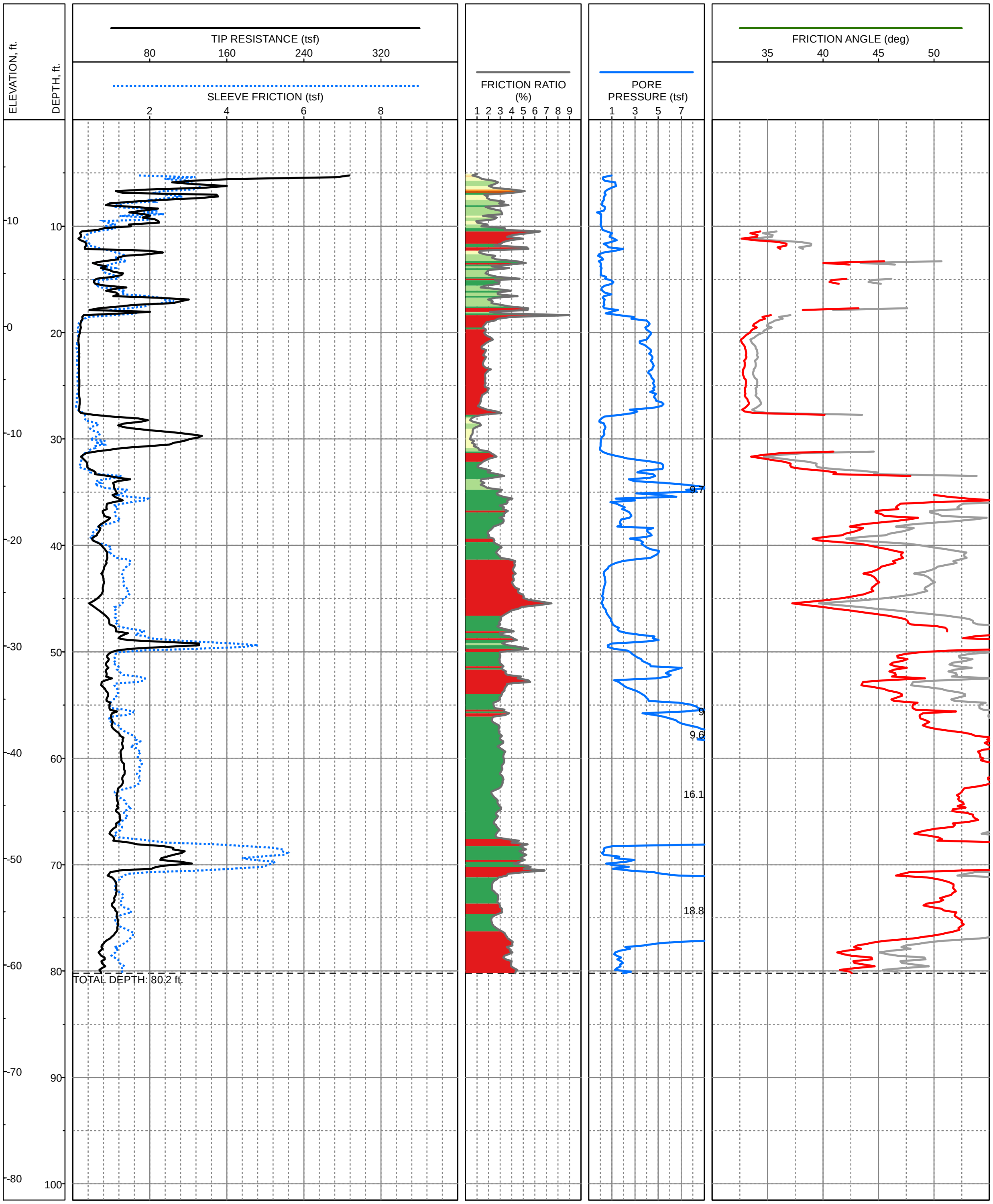
PLATE A-16: LOG OF 2022-CPT-16



LOCATION: E6,052,534, N2,116,701, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 18.6ft
COMPLETION DEPTH: 100.2ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

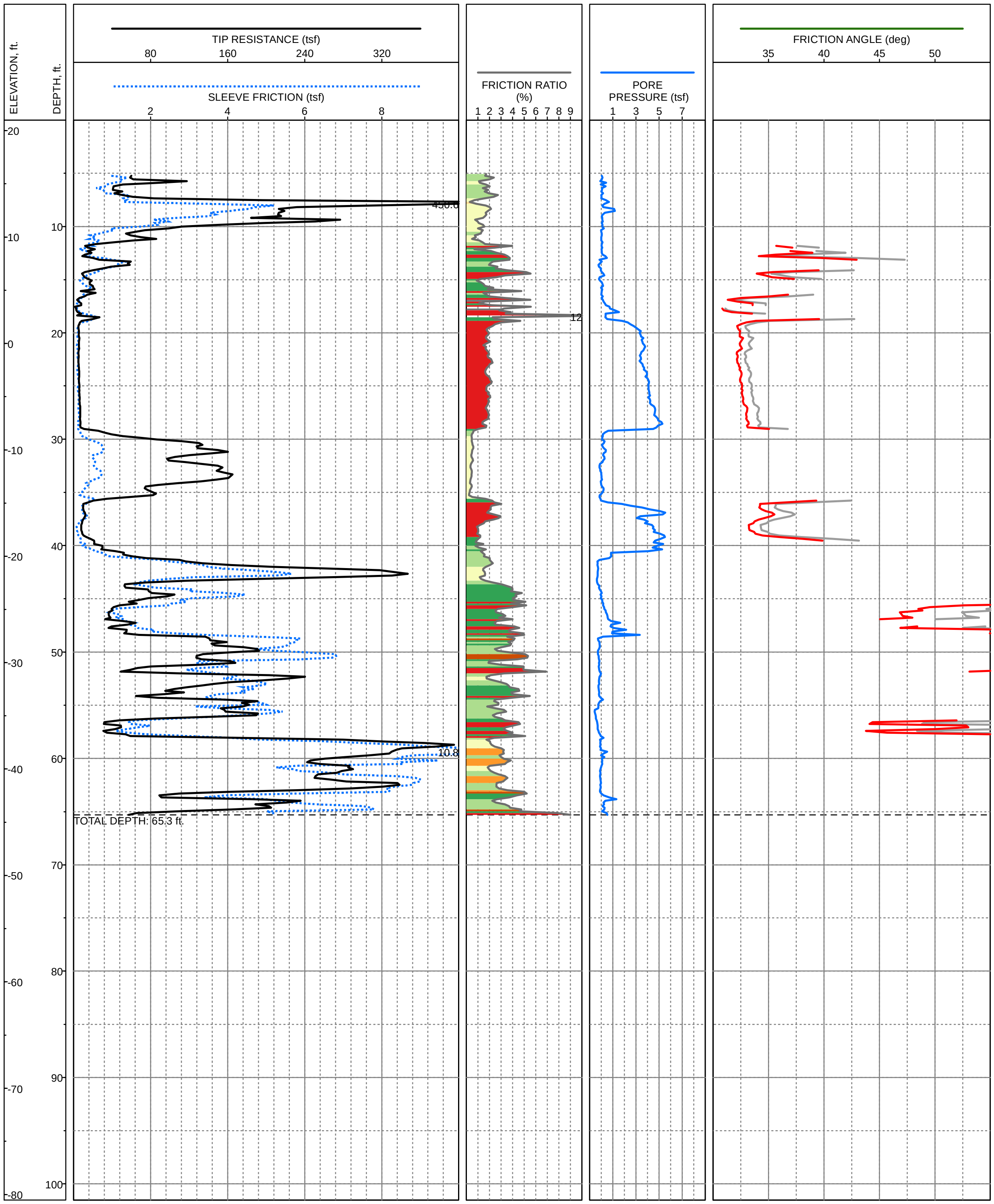
PLATE A-17: LOG OF 2022-CPT-17



LOCATION: E6,052,503, N2,116,793, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 19.4ft
COMPLETION DEPTH: 80.2ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

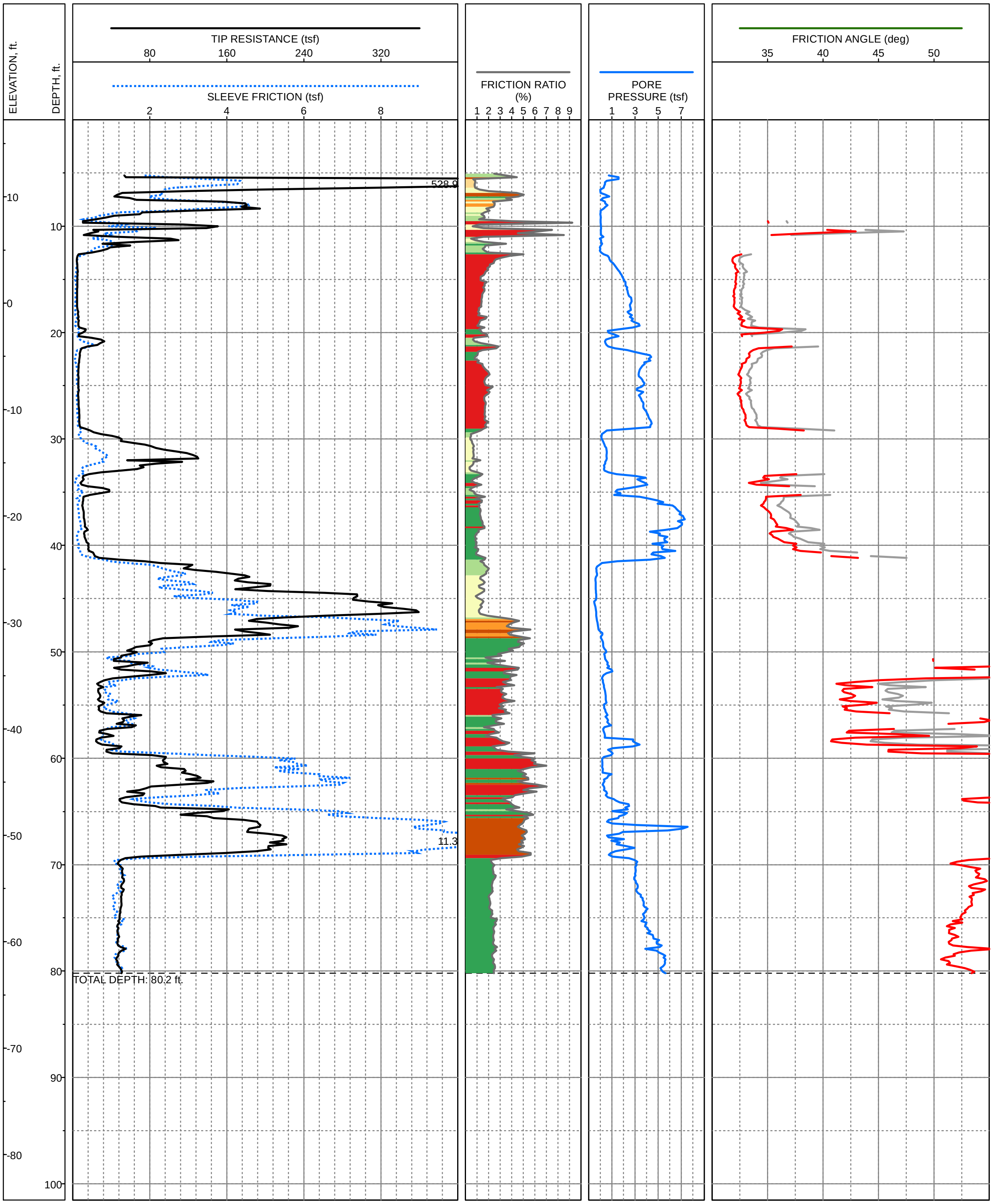
PLATE A-18: LOG OF 2022-CPT-18



LOCATION: E6,052,485, N2,116,730, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 21.0ft
COMPLETION DEPTH: 65.3ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

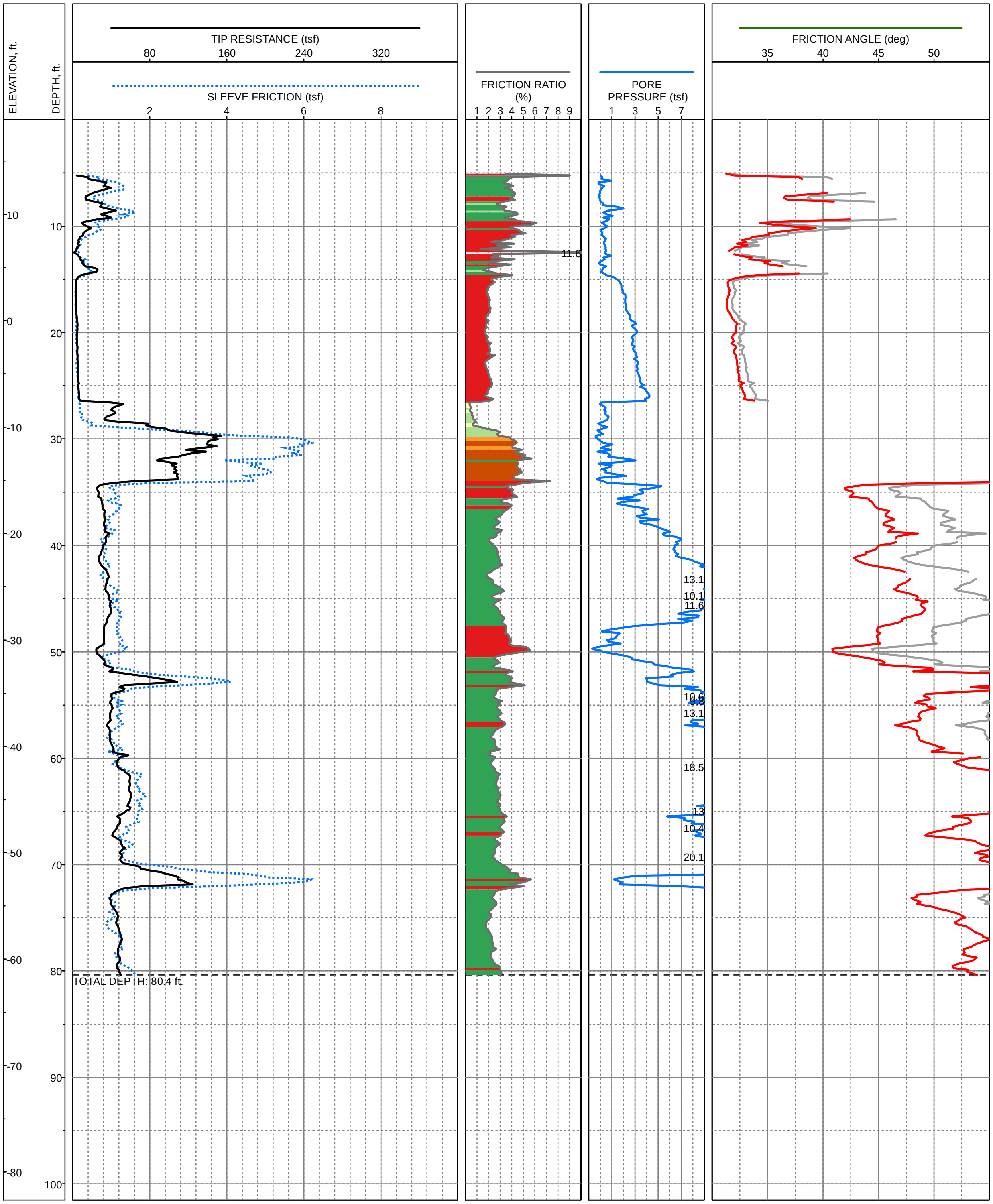
PLATE A-19: LOG OF 2022-CPT-19



LOCATION: E6,052,442, N2,116,671, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 17.2ft
COMPLETION DEPTH: 80.2ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

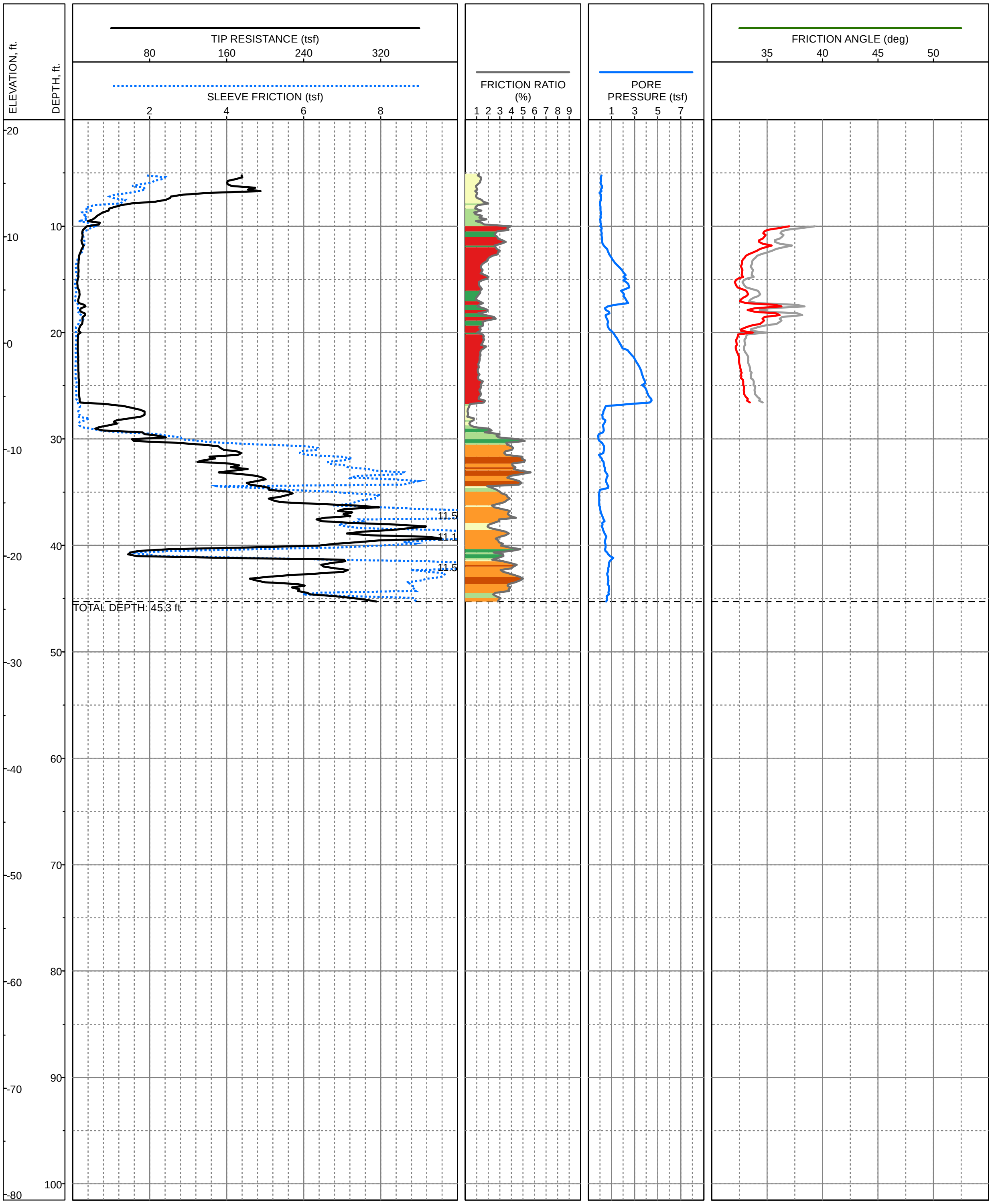
PLATE A-20: LOG OF 2022-CPT-20



LOCATION: E6,052,457, N2,116,815, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 18.9ft
COMPLETION DEPTH: 80.4ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

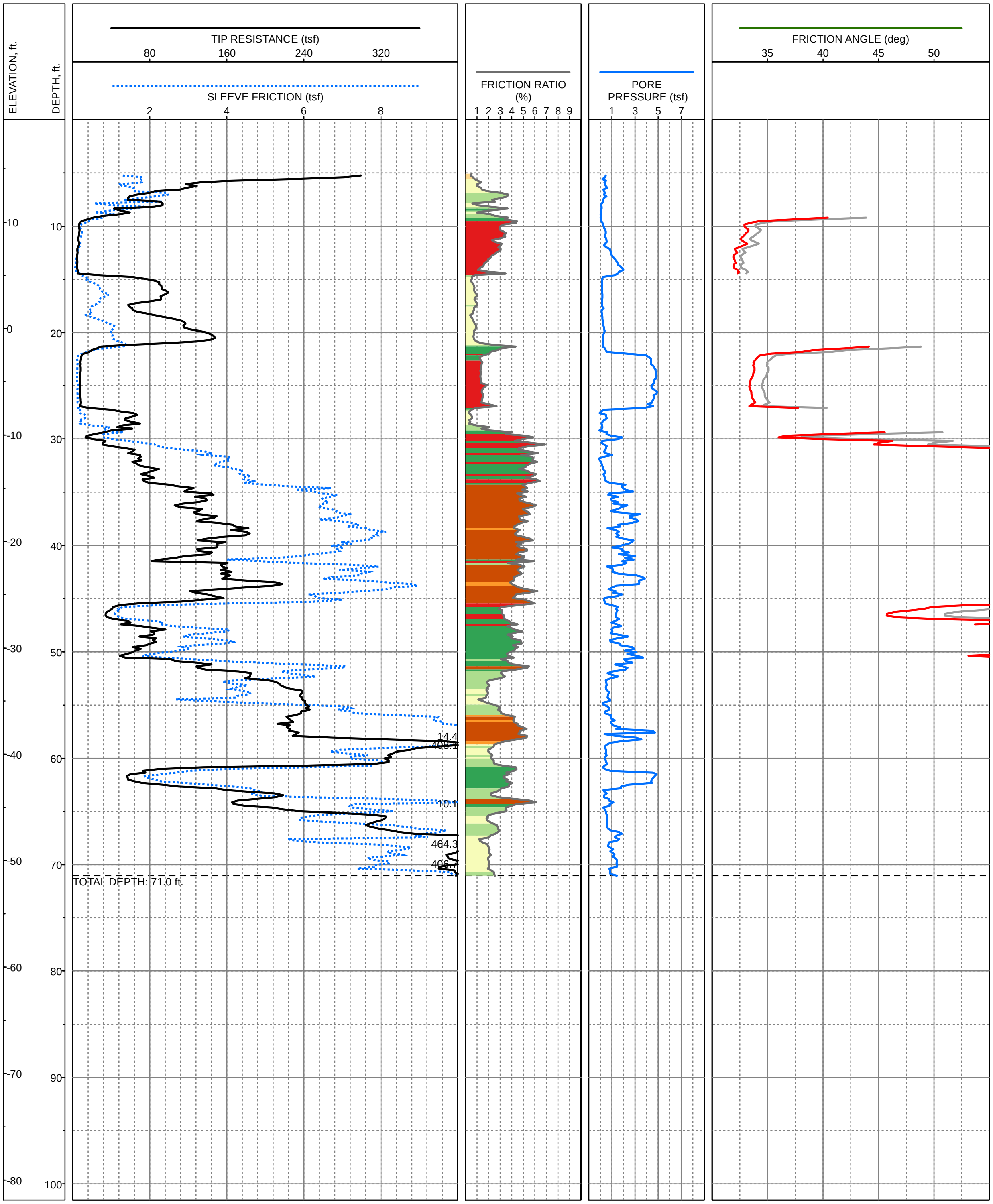
PLATE A-21: LOG OF 2022-CPT-21



LOCATION: E6,052,429, N2,116,770, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 21.0ft
COMPLETION DEPTH: 45.3ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

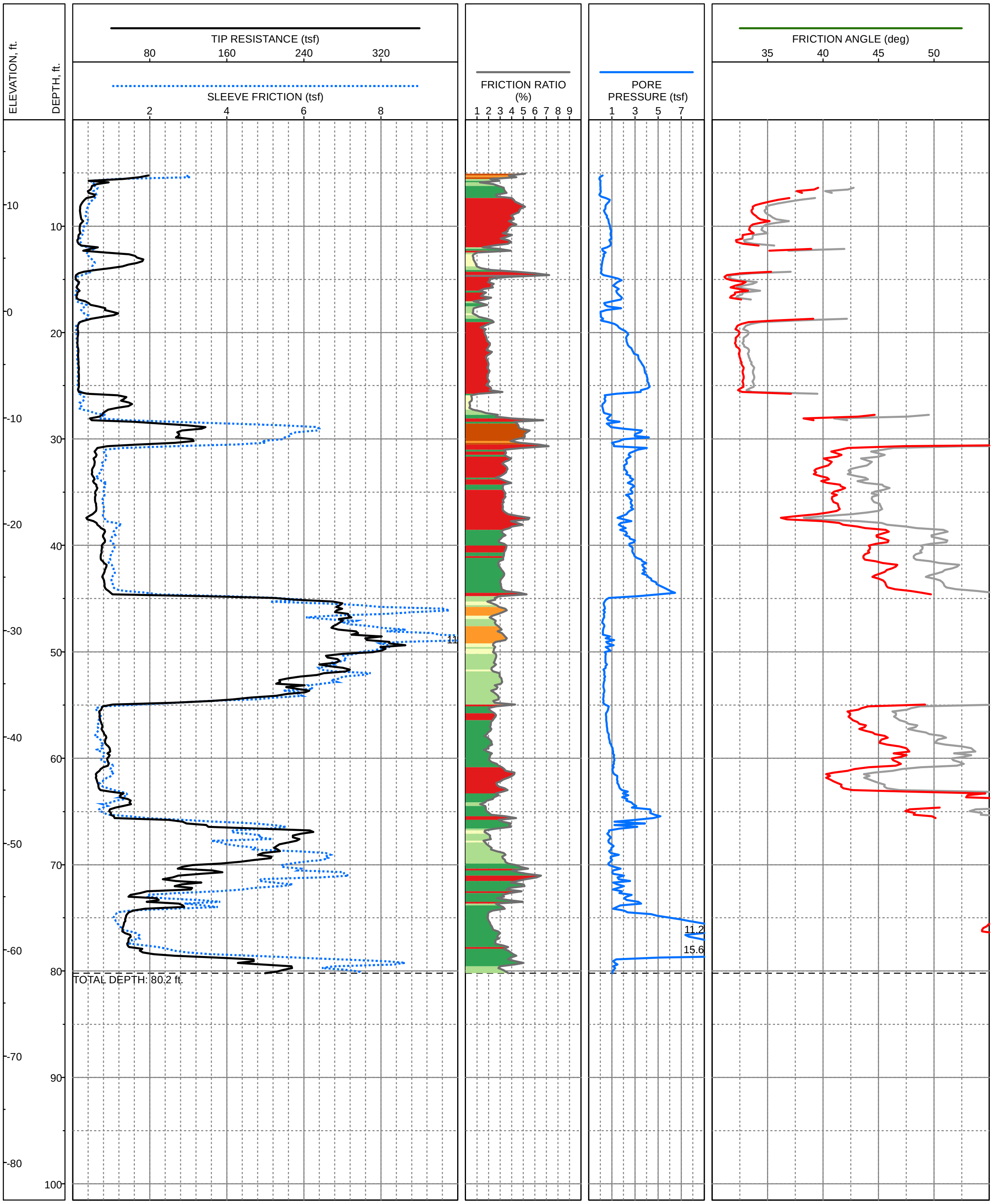
PLATE A-22: LOG OF 2022-CPT-22



LOCATION: E6,052,417, N2,116,734, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 19.6ft
COMPLETION DEPTH: 71.0ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

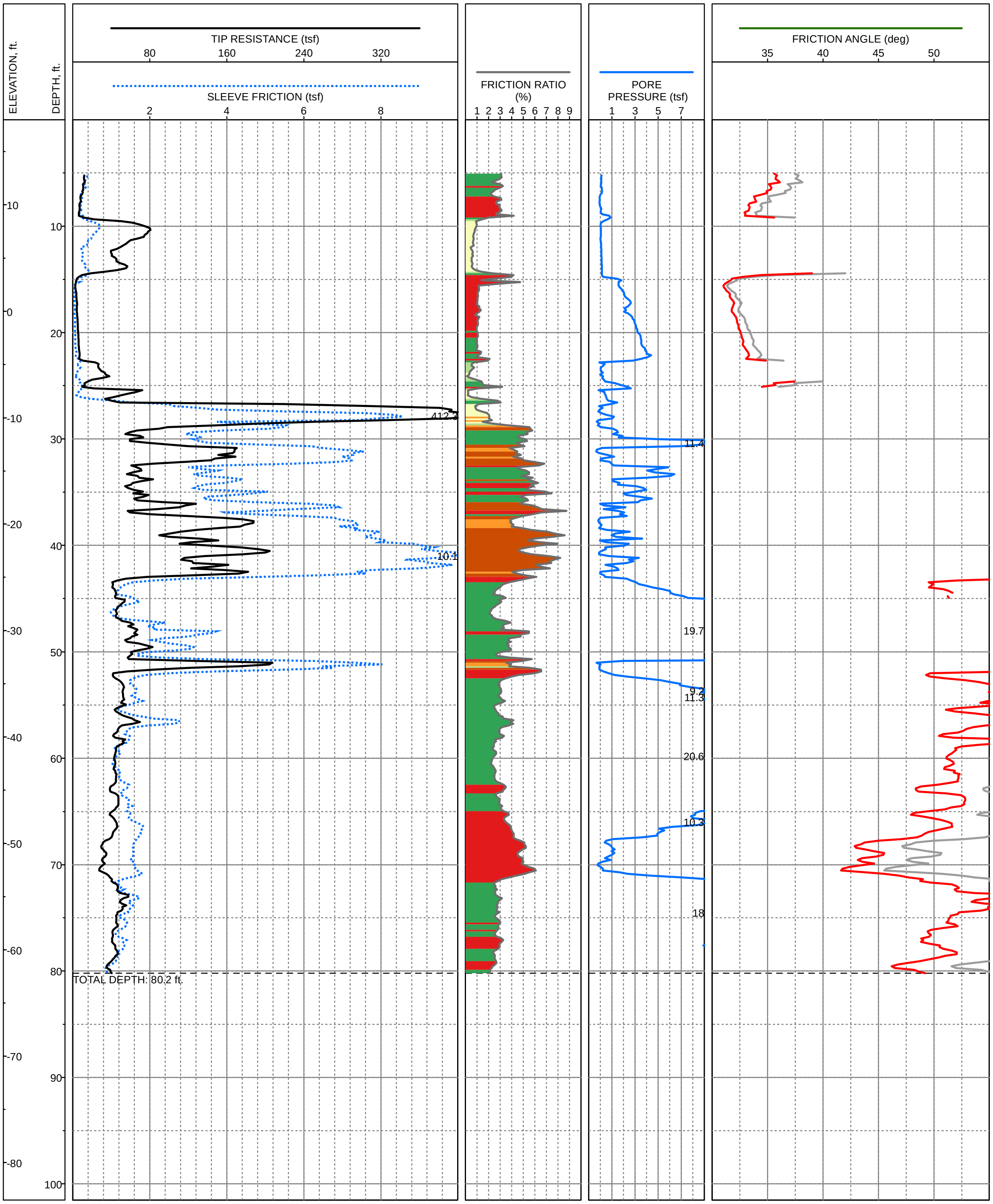
PLATE A-23: LOG OF 2022-CPT-23



LOCATION: E6,052,359, N2,116,755, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 18.0ft
COMPLETION DEPTH: 80.2ft
TESTDATE: 10/7/2022

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

PLATE A-24: LOG OF 2022-CPT-24



LOCATION: E6,052,339, N2,116,824, NAD 1983 State Plane CA, Zone 3
SURFACE EL: 18.0ft
COMPLETION DEPTH: 80.2ft
TESTDATE: 10/7/2022

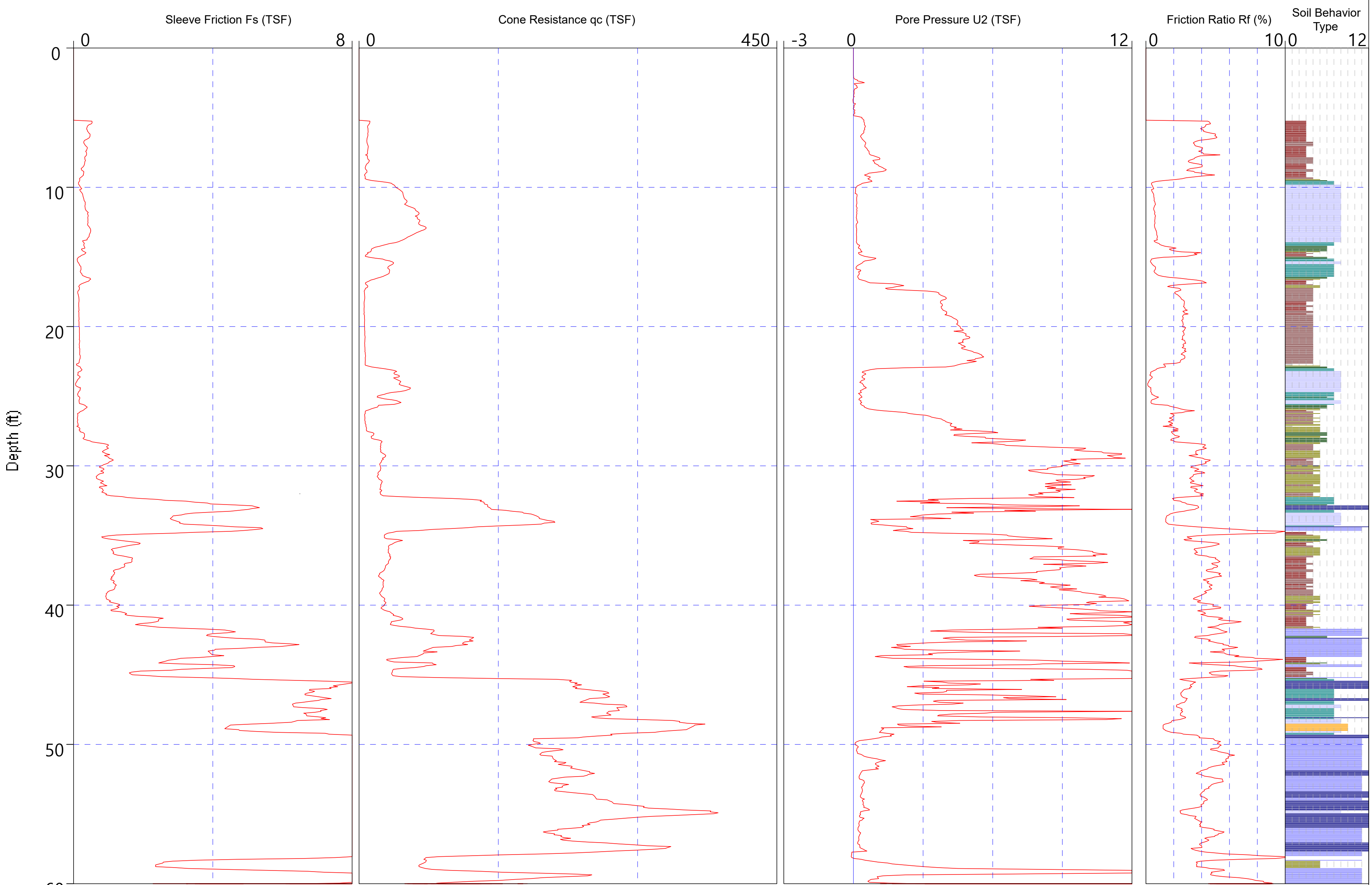
EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: R. Rahimnejad
CONE AREA RATIO: 0.85

PLATE A-25: LOG OF 2022-CPT-25

Job Number: 04.72190021
Operator: Daniel Garza
Location: Oakland, CA

CPT Number: CPT-01
Date: 29-Mar-2019
Elevation: 0.00

Coordinates: 37.795163 -122.262754
Cone Number: CP15-CF75PB7SN2-P1E1 2598



(1) sensitive fine grained (OL-CH)
(2) organic material (OL-OH)
(3) clay (CH)
(4) silty clay to clay (CL-CH)
(5) clayey silt to silty clay (MH-CL)
(6) sandy silt to clayey silt (ML-MH)
(7) silty sand to sandy silt (SM-ML)
(8) sand to silty sand (SM-SP)
(9) sand (SW-SP)
(10) gravel to gravelly sand (SW-GW)
(11) very stiff fine grained* (CH-CL)
(12) sand to clayey sand* (SC-SM)

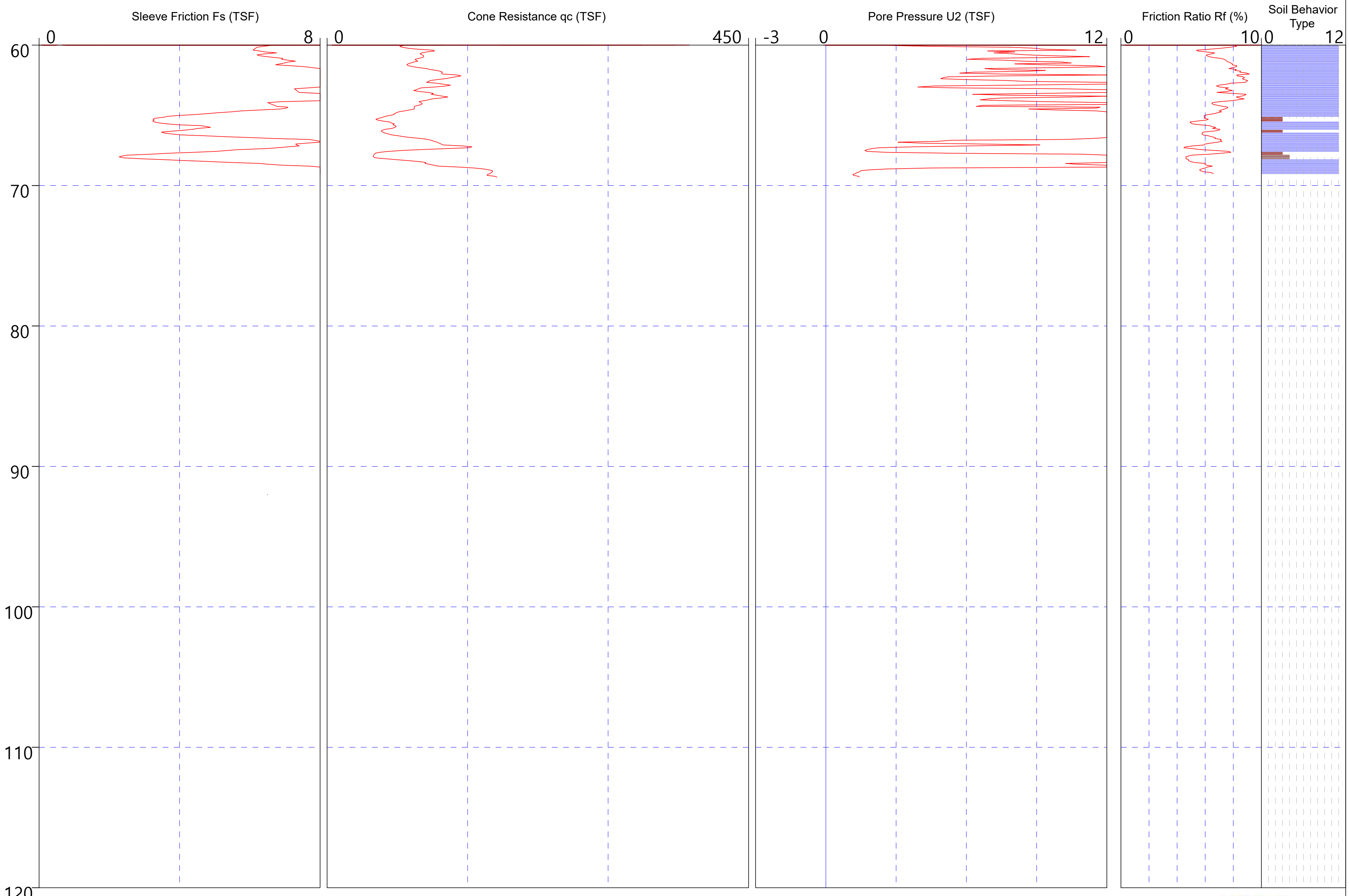
PLATE A-26

Robertson et al. 1986 *Overconsolidated or Cemented

Job Number: 04.72190021
Operator: Daniel Garza
Location: Oakland, CA

CPT Number: CPT-01
Date: 29-Mar-2019
Elevation: 0.00

Coordinates: 37.795163 -122.262754
Cone Number: CP15-CF75PB7SN2-P1E1 2598



(1) sensitive fine grained (OL-CH)
(2) organic material (OL-OH)
(3) clay (CH)
(4) silty clay to clay (CL-CH)
(5) clayey silt to silty clay (MH-CL)
(6) sandy silt to clayey silt (ML-MH)
(7) silty sand to sandy silt (SM-ML)
(8) sand to silty sand (SM-SP)
(9) sand (SW-SP)
(10) gravel to gravelly sand (SW-GW)
(11) very stiff fine grained* (CH-CL)
(12) sand to clayey sand* (SC-SM)

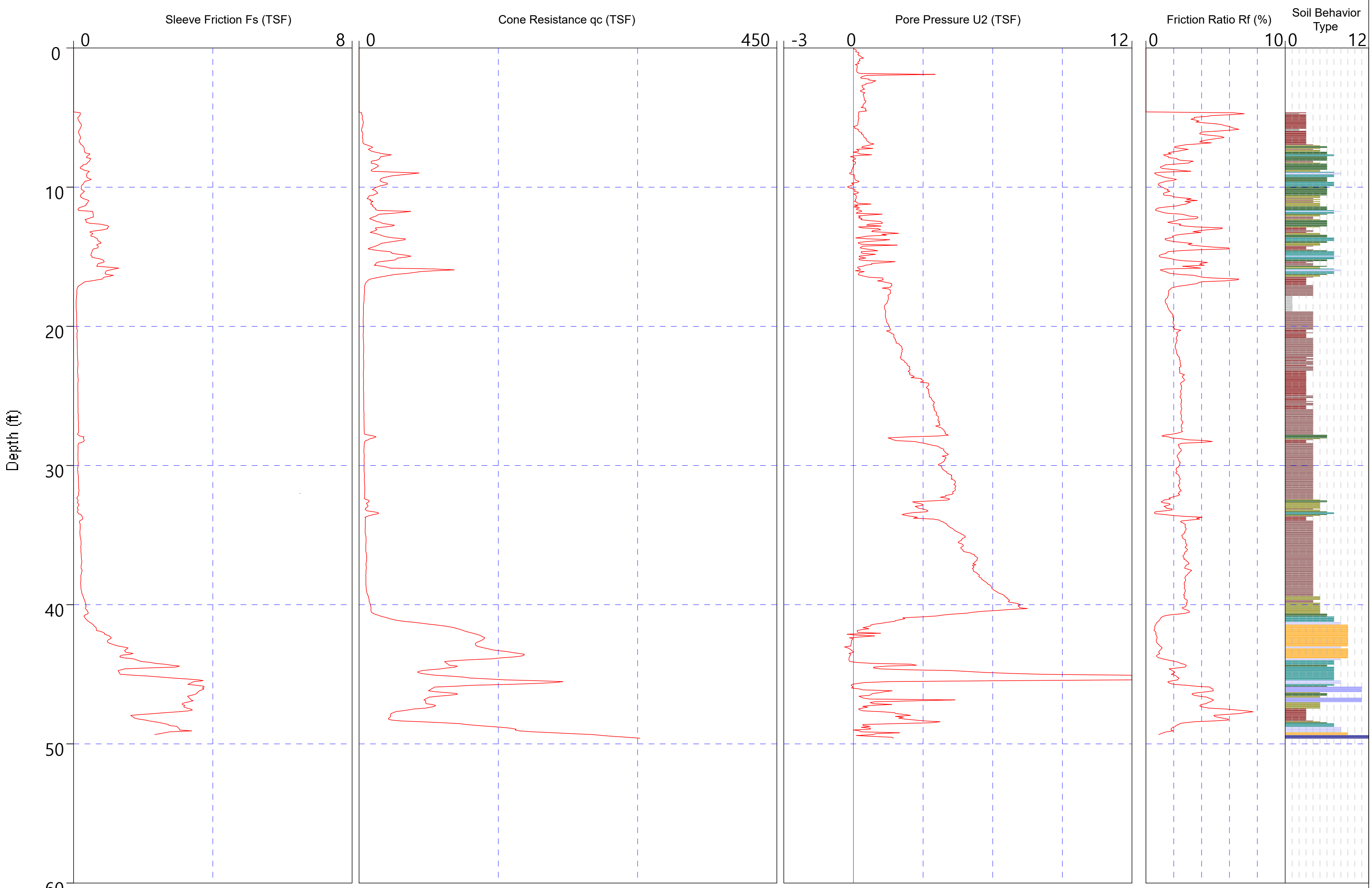
PLATE A-27

Robertson et al. 1986 *Overconsolidated or Cemented

Job Number: 04.72190021
Operator: Daniel Garza
Location: Oakland, CA

CPT Number: CPT-02
Date: 29-Mar-2019
Elevation: 0.00

Coordinates: 37.794900 -122.261959
Cone Number: CP15-CF75PB7SN2-P1E1 2598



(1) sensitive fine grained (OL-CH)
(2) organic material (OL-OH)
(3) clay (CH)
(4) silty clay to clay (CL-CH)
(5) clayey silt to silty clay (MH-CL)
(6) sandy silt to clayey silt (ML-MH)
(7) silty sand to sandy silt (SM-ML)
(8) sand to silty sand (SM-SP)
(9) sand (SW-SP)
(10) gravel to gravelly sand (SW-GW)
(11) very stiff fine grained* (CH-CL)
(12) sand to clayey sand* (SC-SM)

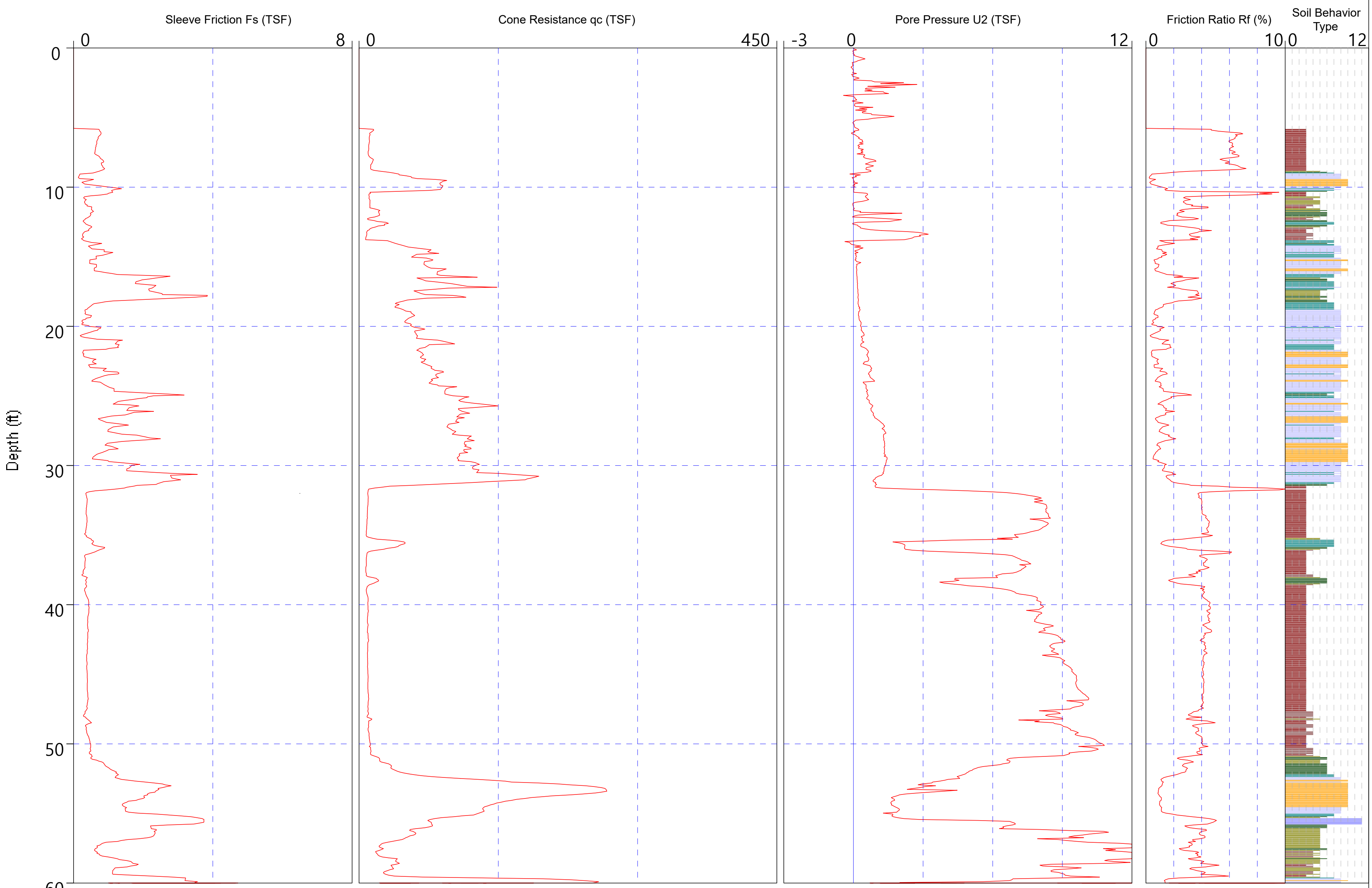
PLATE A-28

Robertson et al. 1986 *Overconsolidated or Cemented

Job Number: 04.72190021
Operator: Daniel Garza
Location: Oakland, CA

CPT Number: CPT-03
Date: 29-Mar-2019
Elevation: 0.00

Coordinates: 37.794463 -122.262030
Cone Number: CP15-CF75PB7SN2-P1E1 2598



(1) sensitive fine grained (OL-CH)
(2) organic material (OL-OH)
(3) clay (CH)
(4) silty clay to clay (CL-CH)
(5) clayey silt to silty clay (MH-CL)
(6) sandy silt to clayey silt (ML-MH)
(7) silty sand to sandy silt (SM-ML)
(8) sand to silty sand (SM-SP)
(9) sand (SW-SP)
(10) gravel to gravelly sand (SW-GW)
(11) very stiff fine grained* (CH-CL)
(12) sand to clayey sand* (SC-SM)

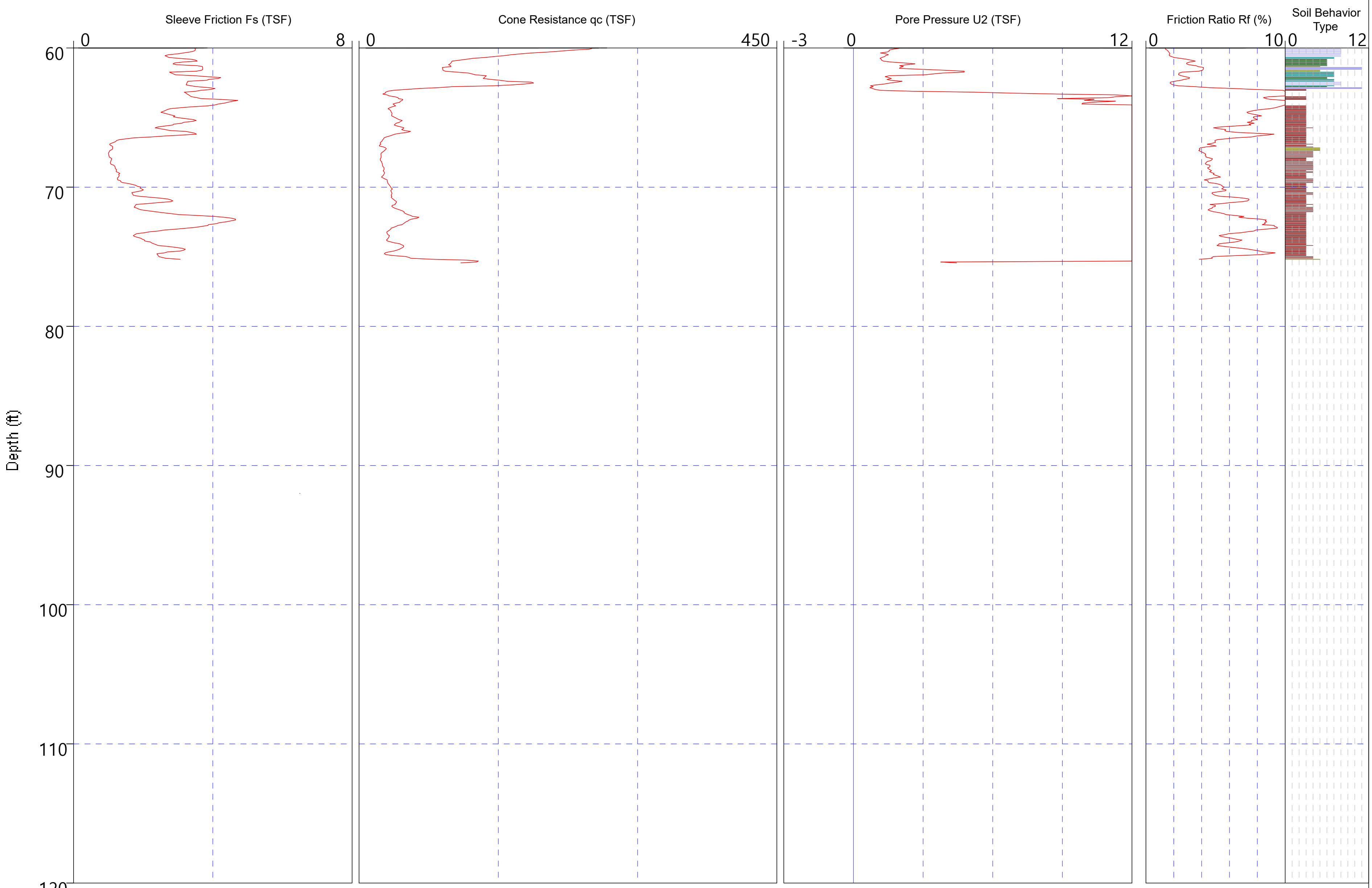
PLATE A-29

Robertson et al. 1986 *Overconsolidated or Cemented

Job Number: 04.72190021
Operator: Daniel Garza
Location: Oakland, CA

CPT Number: CPT-03
Date: 29-Mar-2019
Elevation: 0.00

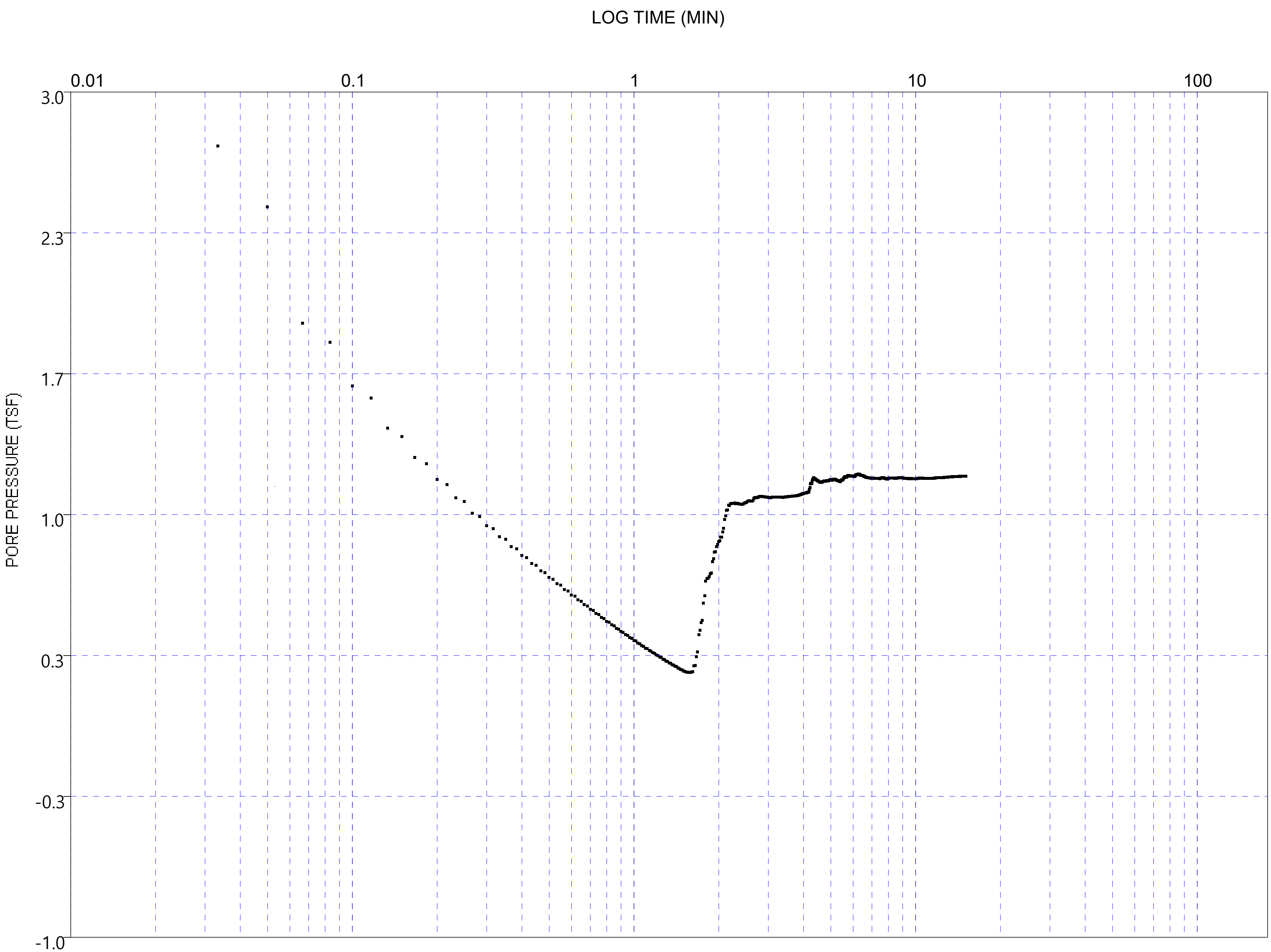
Coordinates: 37.794463 -122.262030
Cone Number: CP15-CF75PB7SN2-P1E1 2598



(1) sensitive fine grained (OL-CH)
(2) organic material (OL-OH)
(3) clay (CH)
(4) silty clay to clay (CL-CH)
(5) clayey silt to silty clay (MH-CL)
(6) sandy silt to clayey silt (ML-MH)
(7) silty sand to sandy silt (SM-ML)
(8) sand to silty sand (SM-SP)
(9) sand (SW-SP)
(10) gravel to gravelly sand (SW-GW)
(11) very stiff fine grained* (CH-CL)
(12) sand to clayey sand* (SC-SM)

PLATE A-30

Robertson et al. 1986 *Overconsolidated or Cemented



DISSIPATION TEST

CPT Number: CPT-01

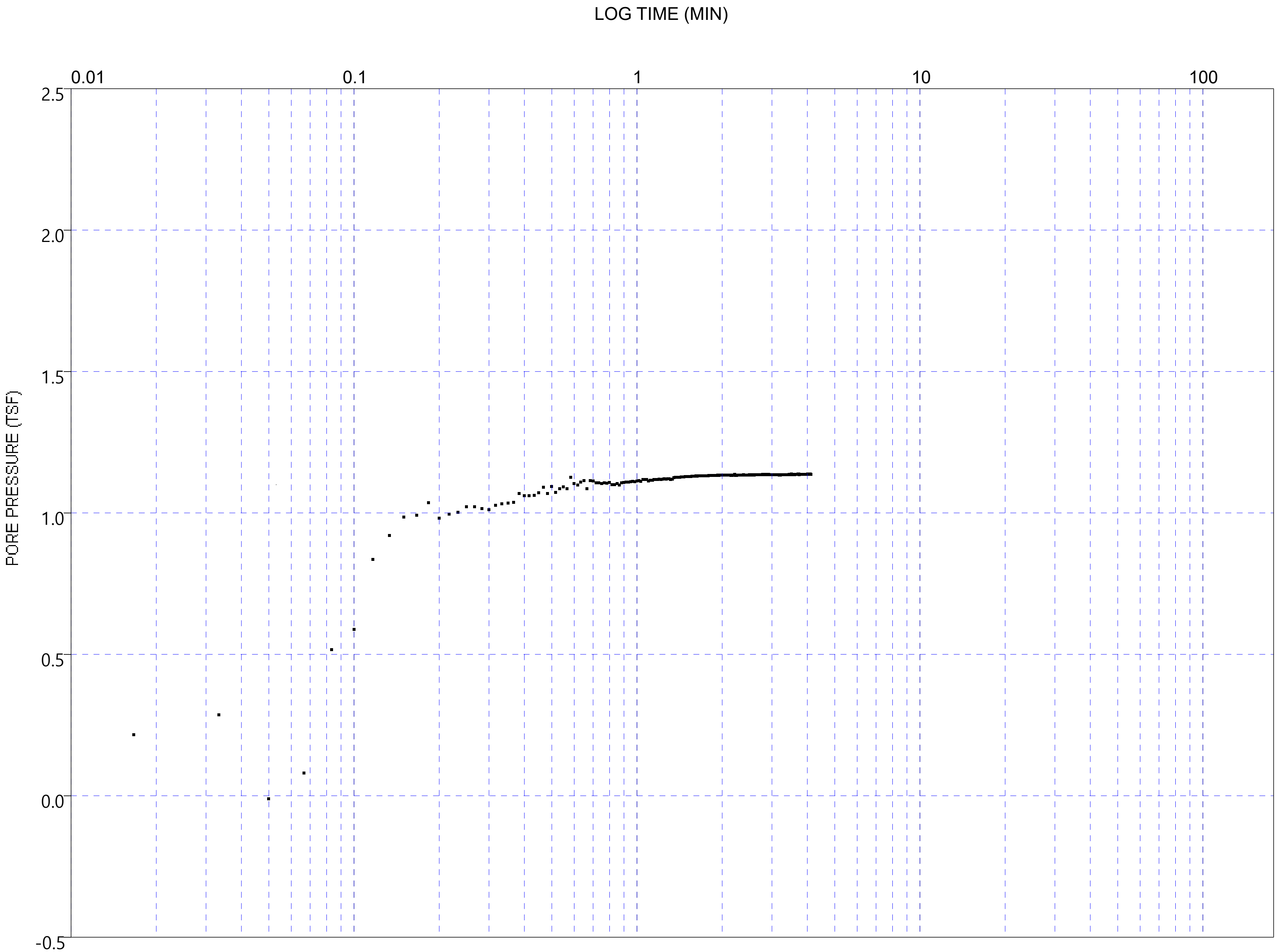
Job Number: 04.72190021

Depth: 48.77

Date: 29-Mar-2019

FUGRO Consultants, Inc.

PLATE A-31



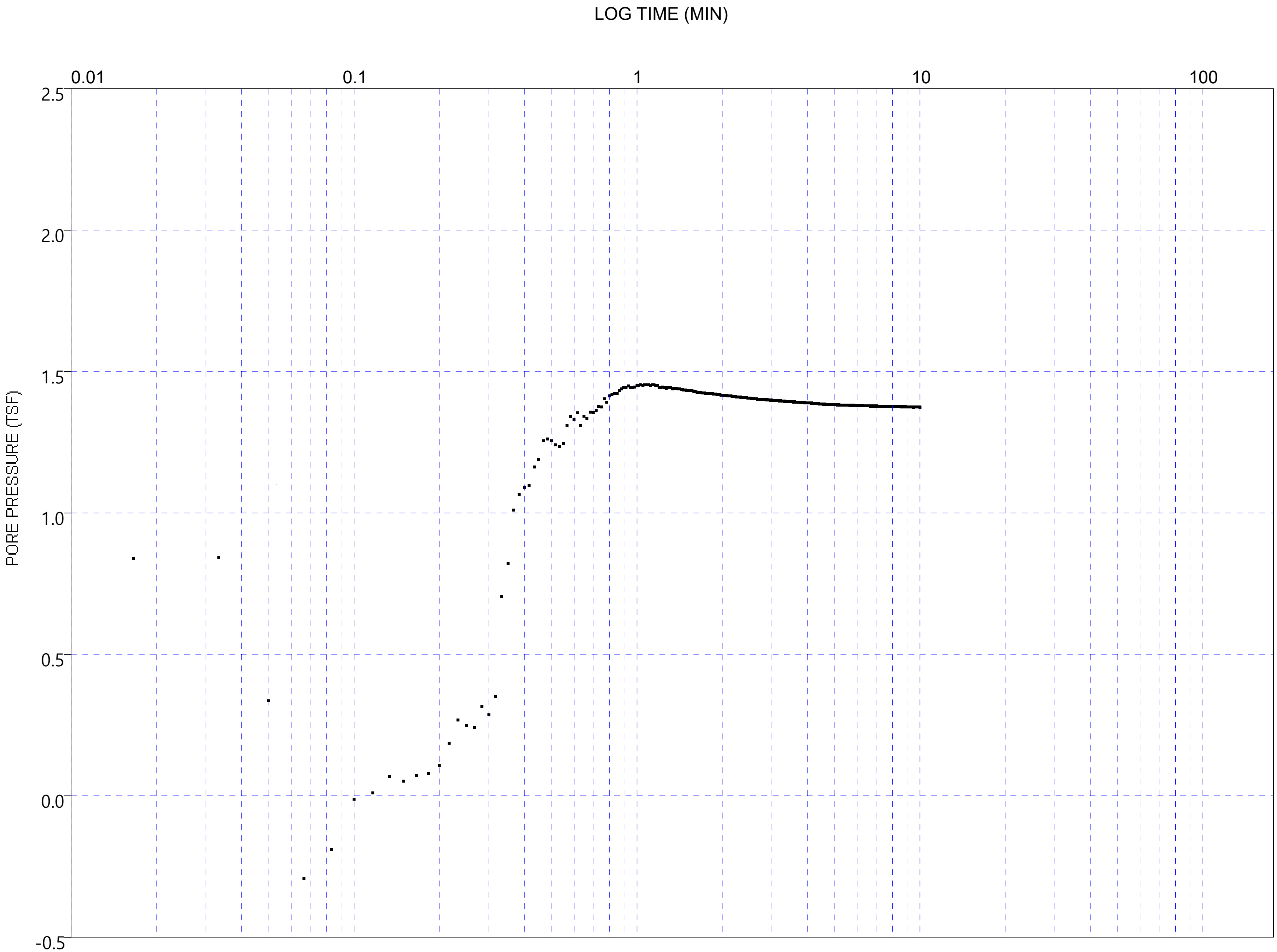
DISSIPATION TEST

CPT Number: CPT-02

Job Number: 04.72190021

Depth: 42.02

Date: 29-Mar-2019



DISSIPATION TEST

CPT Number: CPT-02

Job Number: 04.72190021

Depth: 49.33

Date: 29-Mar-2019

FUGRO Consultants, Inc.

PLATE A-33

Cone Penetration Test Sounding Summary

-Table 1-

CPT Sounding Identification	Date	Termination Depth (feet)	Depth of Groundwater Samples (feet)	Depth of Soil Samples (feet)	Depth of Pore Pressure Dissipation Tests (feet)
CPT-04	01/03/2020	75.13	-	-	31.3
CPT-05	01/03/2020	75.13	-	-	41.2
CPT-06	01/03/2020	75.30	-	-	-
SCPT-07	01/03/2020	75.13	-	-	57.6
CPT-08	01/02/2020	51.67	-	-	51.7

2726 Walnut Ave. • Signal Hill, California 90755 • (562) 427-6899 • FAX (562) 427-3314
950 Howe Road. • Martinez, California 94553 • (925) 313-5800 • FAX (925) 313-0302
www.greggdrilling.com

Gregg Drilling, LLC
www.greggdrilling.com

CPT: SCPT-07

CLIENT: FUGRO
SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD
Total depth: 75.13 ft, Date: 1/3/2020

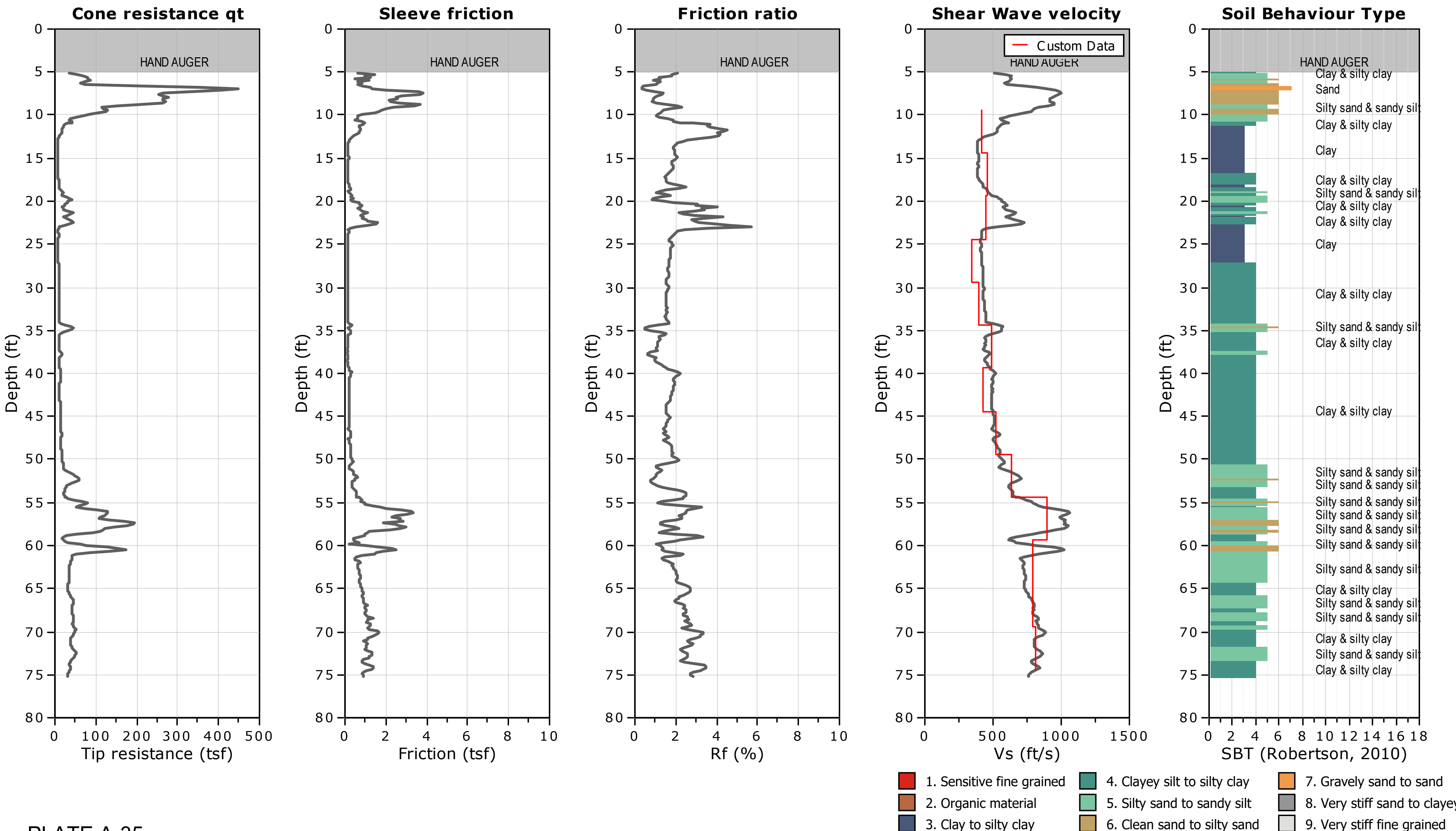


PLATE A-35

Gregg Drilling, LLC
www.greggdrilling.com

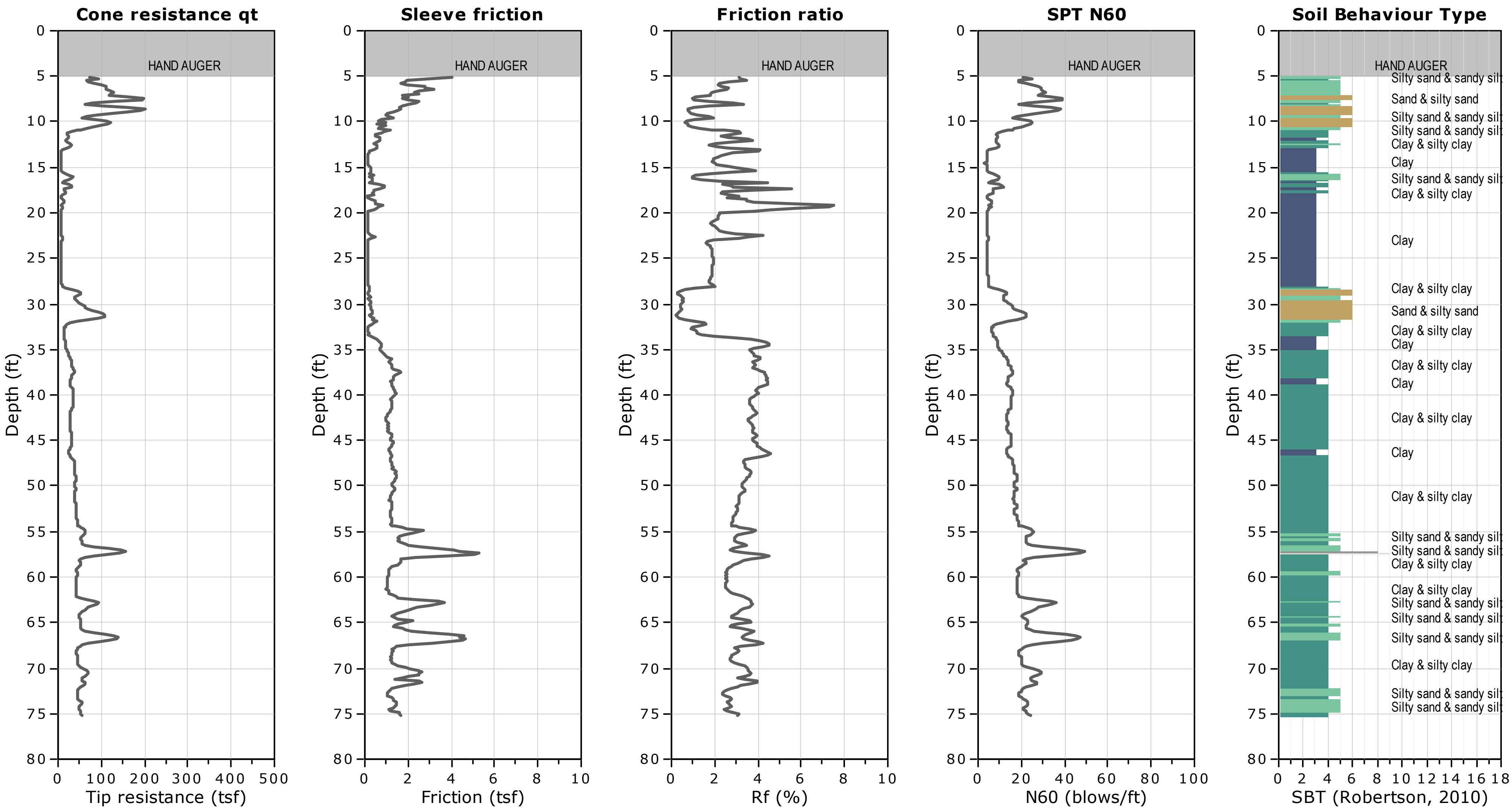
CPT: CPT-04

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

Total depth: 75.13 ft, Date: 1/3/2020



1. Sensitive fine grained	4. Clayey silt to silty clay	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to clayey
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

PLATE A-36

Gregg Drilling, LLC
www.greggdrilling.com

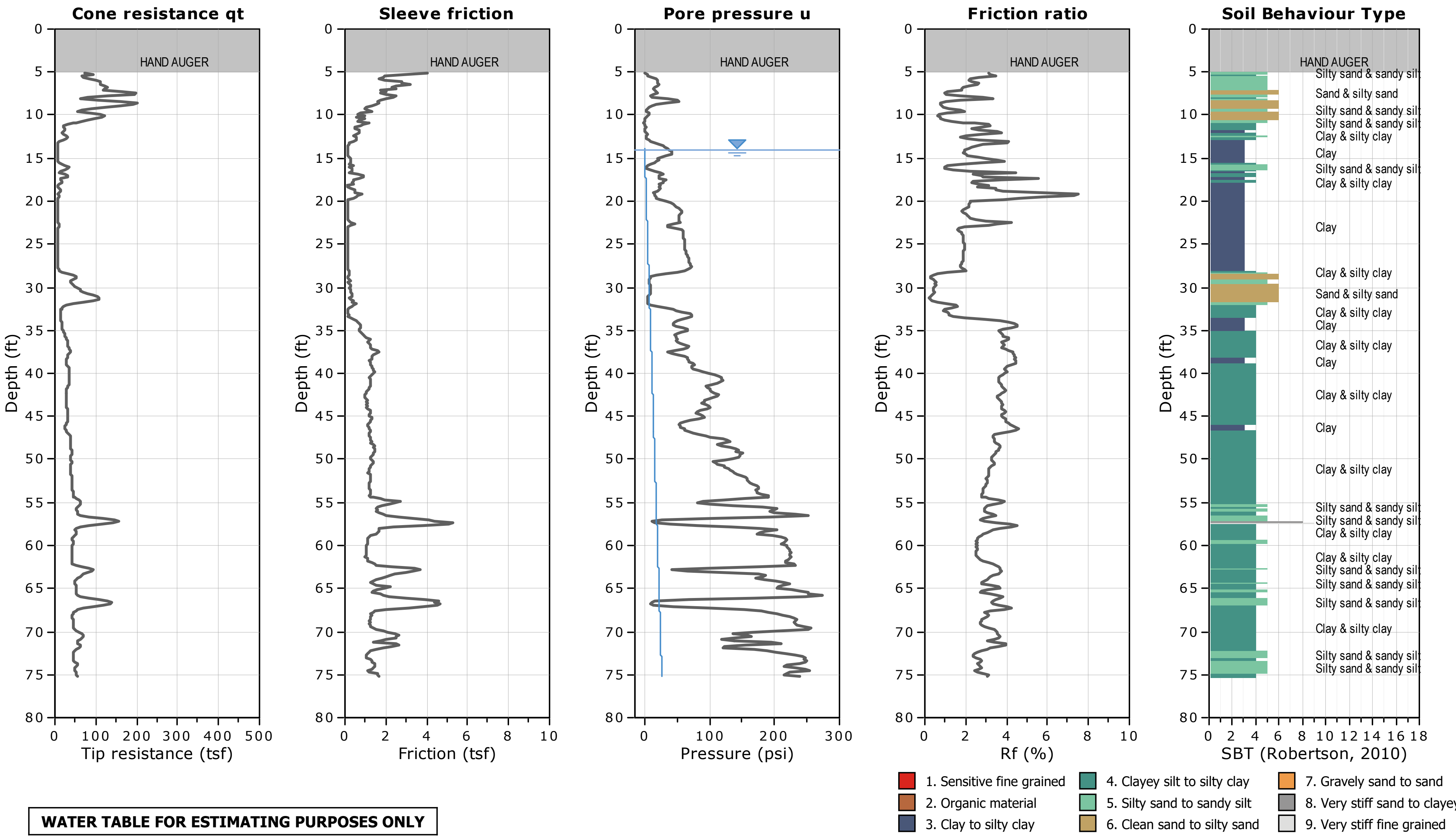
CPT: CPT-04

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

Total depth: 75.13 ft, Date: 1/3/2020



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-37

Gregg Drilling, LLC
www.greggdrilling.com

CPT: CPT-05

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

Total depth: 75.13 ft, Date: 1/3/2020

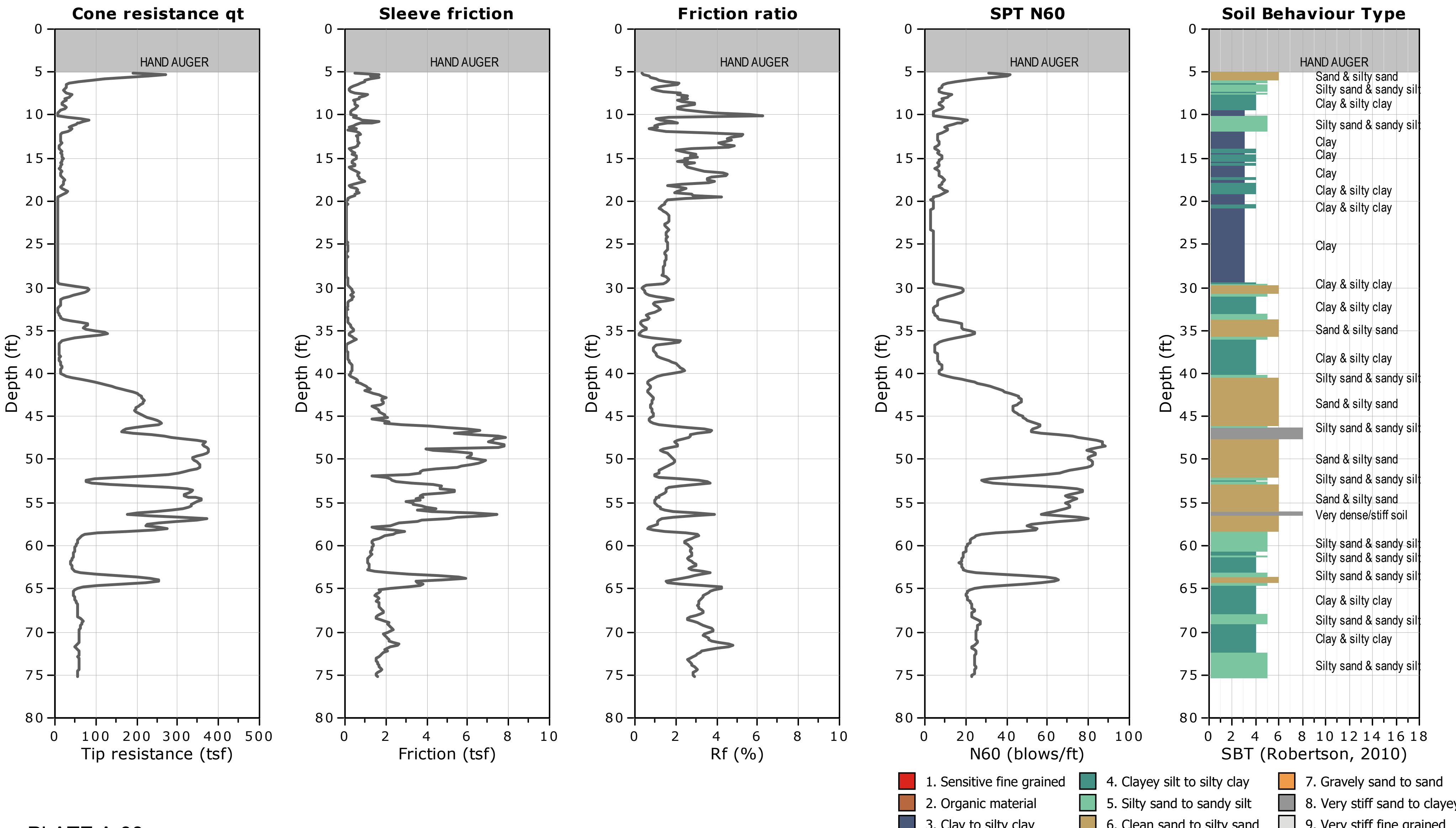


PLATE A-38

Gregg Drilling, LLC
www.greggdrilling.com

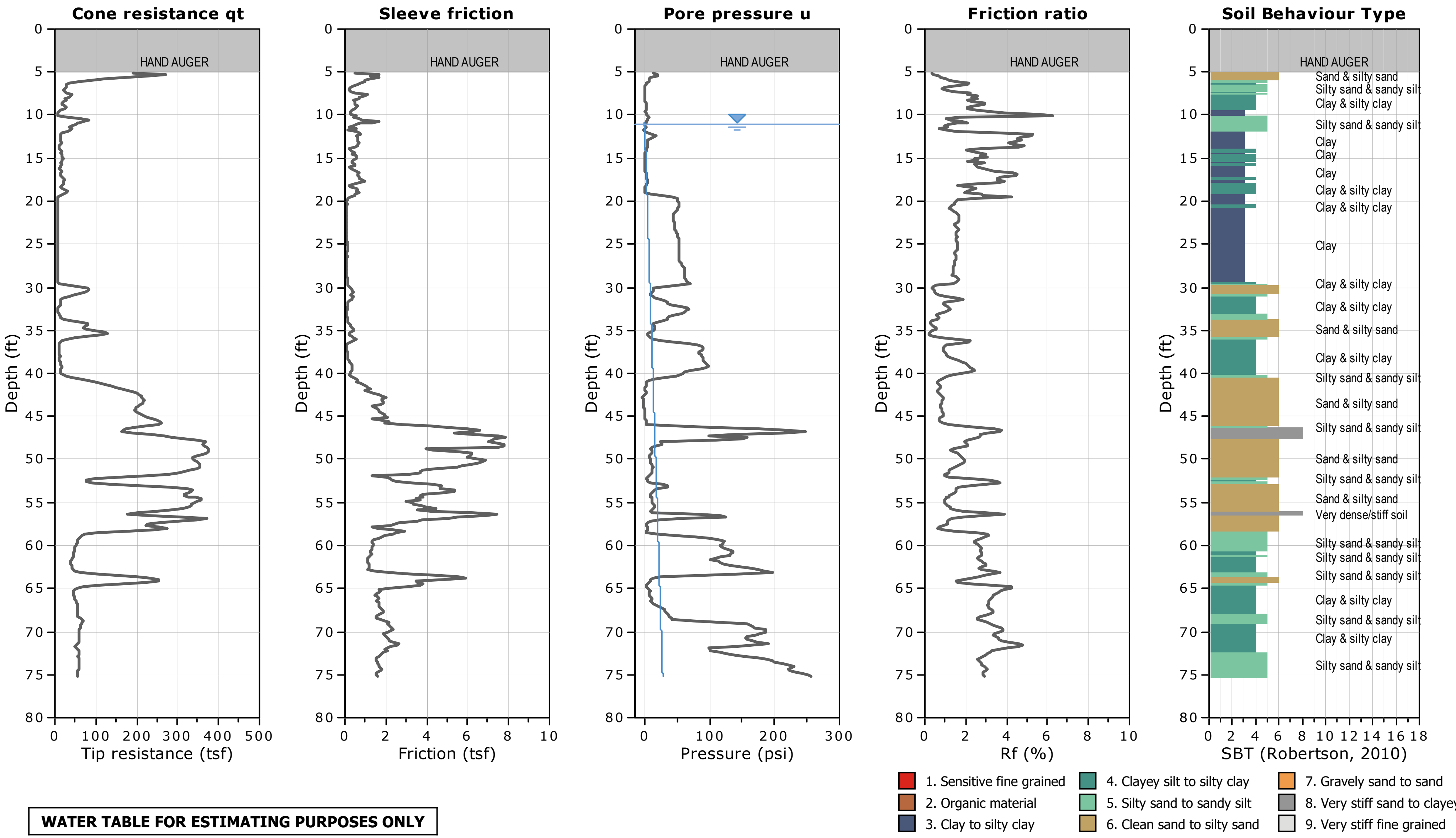
CPT: CPT-05

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

Total depth: 75.13 ft, Date: 1/3/2020



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-39

Gregg Drilling, LLC
www.greggdrilling.com

CPT: CPT-06

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

Total depth: 75.30 ft, Date: 1/3/2020

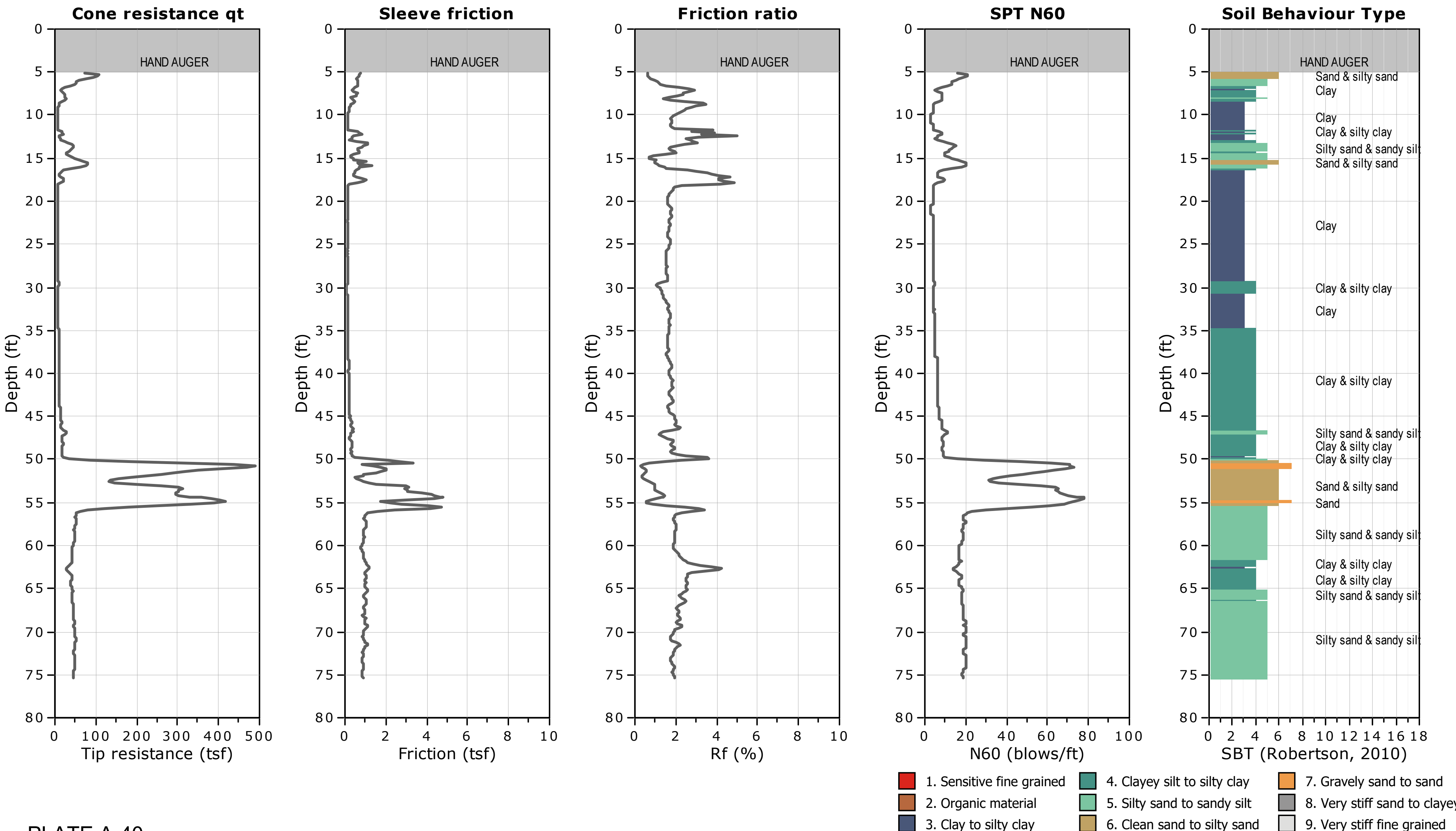


PLATE A-40

Gregg Drilling, LLC
www.greggdrilling.com

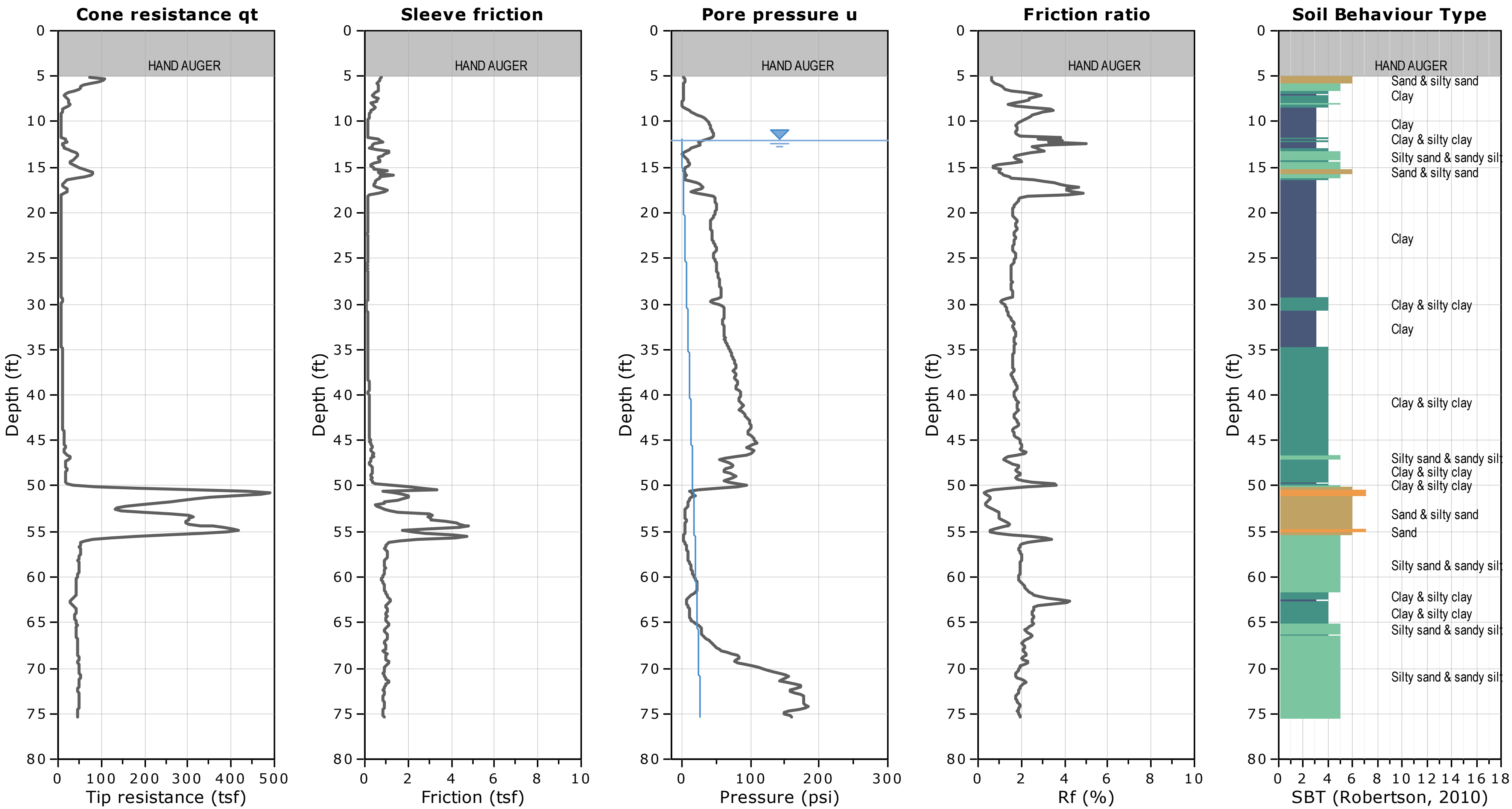
CPT: CPT-06

CLIENT: FUGRO

FIELD REP: REZA RAHIMNEJAD

SITE: LANEY COLLEGE, OAKLAND, CA

Total depth: 75.30 ft, Date: 1/3/2020



1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey
9. Very stiff fine grained

WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-41

Gregg Drilling, LLC
www.greggdrilling.com

CPT: SCPT-07

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

Total depth: 75.13 ft, Date: 1/3/2020

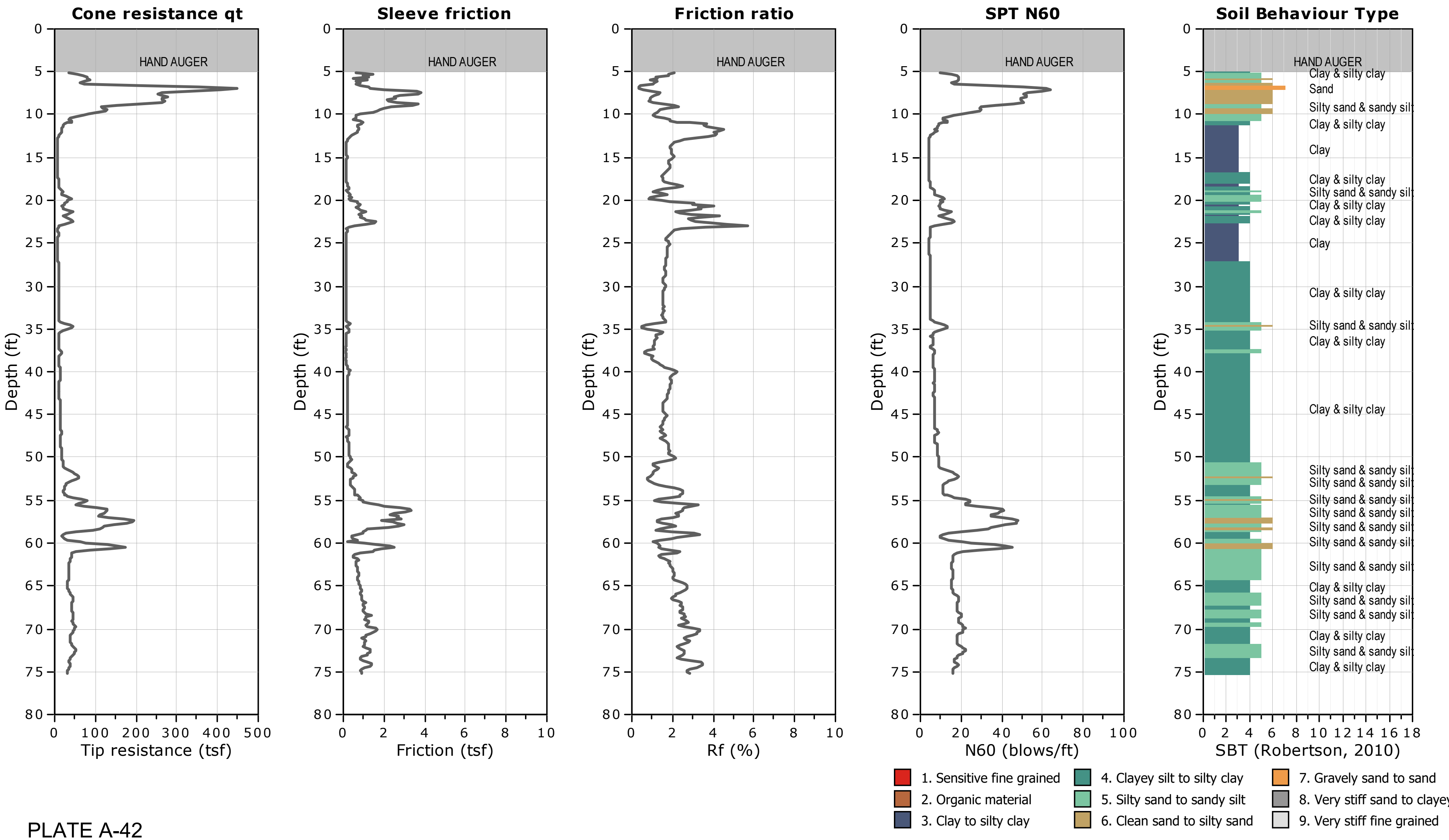


PLATE A-42

Gregg Drilling, LLC
www.greggdrilling.com

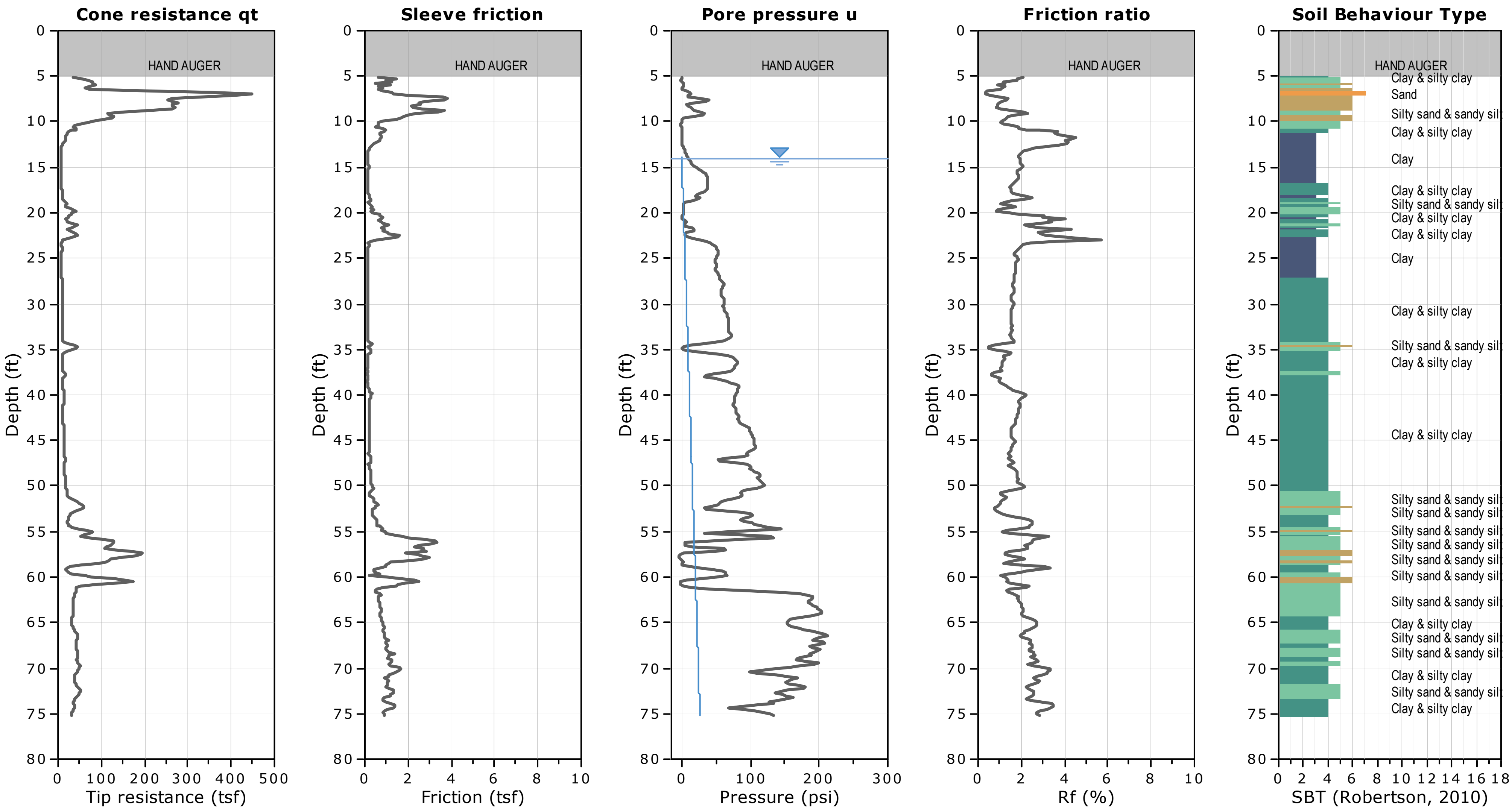
CPT: SCPT-07

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

Total depth: 75.13 ft, Date: 1/3/2020



1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey
9. Very stiff fine grained

WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-43

Gregg Drilling, LLC
www.greggdrilling.com

CPT: CPT-08

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

Total depth: 51.67 ft, Date: 1/2/2020

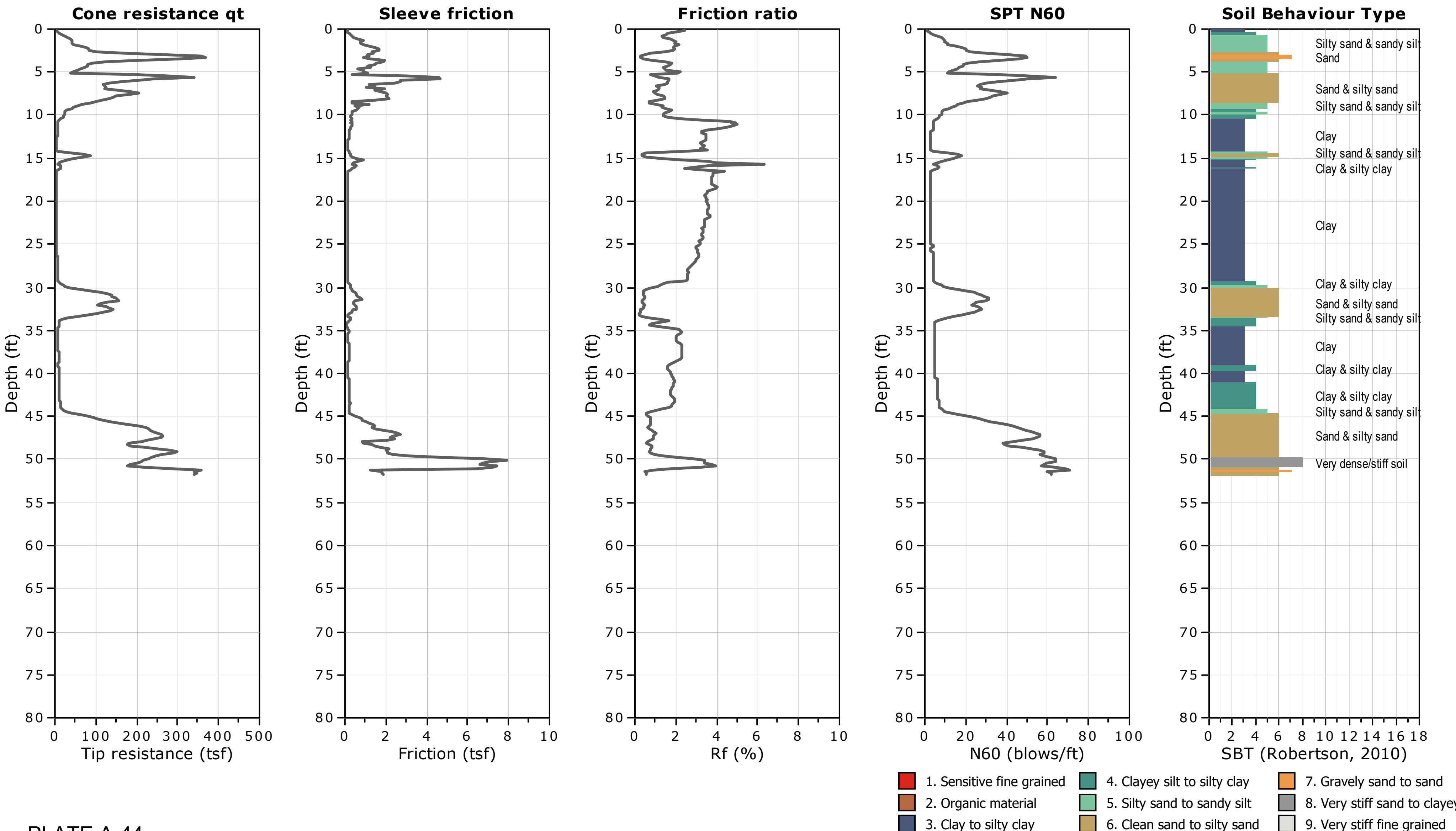


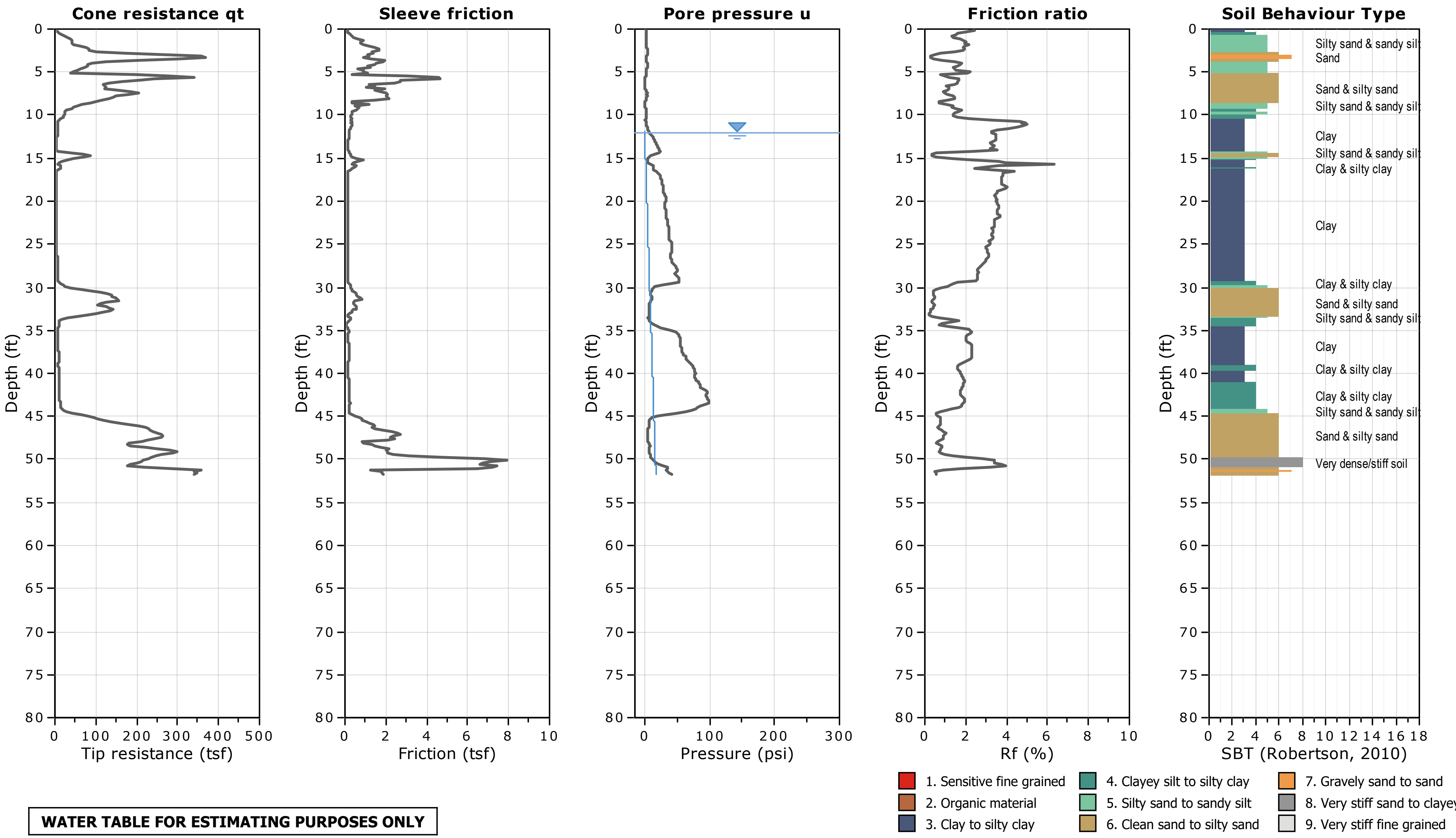
PLATE A-44

Gregg Drilling, LLC
www.greggdrilling.com

CPT: CPT-08

CLIENT: FUGRO
SITE: LANEY COLLEGE, OAKLAND, CA

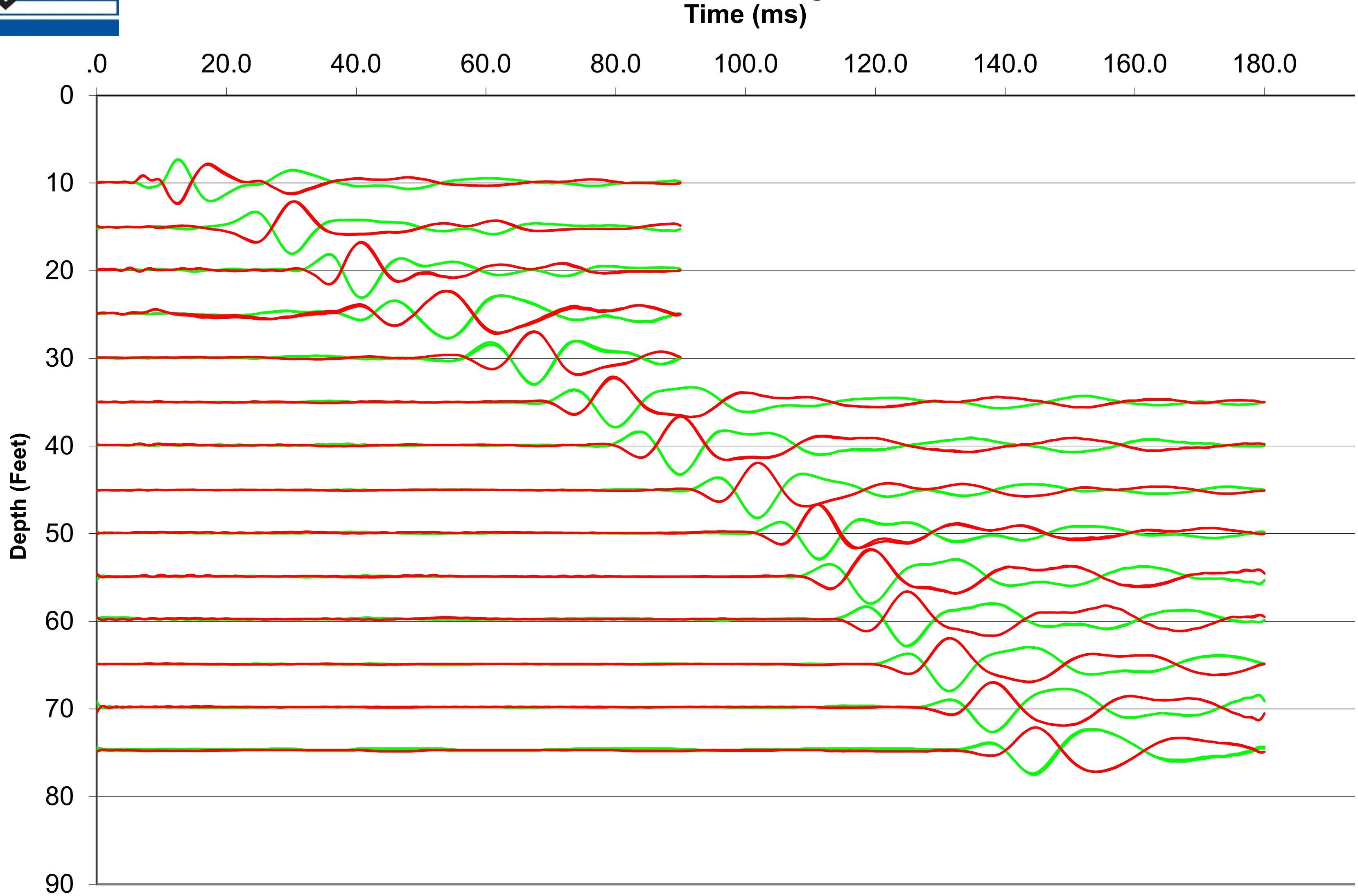
FIELD REP: REZA RAHIMNEJAD
Total depth: 51.67 ft, Date: 1/2/2020



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-45

Waveforms for Sounding SCPT-07



Shear Wave Velocity Calculations

Laney College

SCPT-07

Geophone Offset: 0.66 Feet
Source Offset: 1.67 Feet

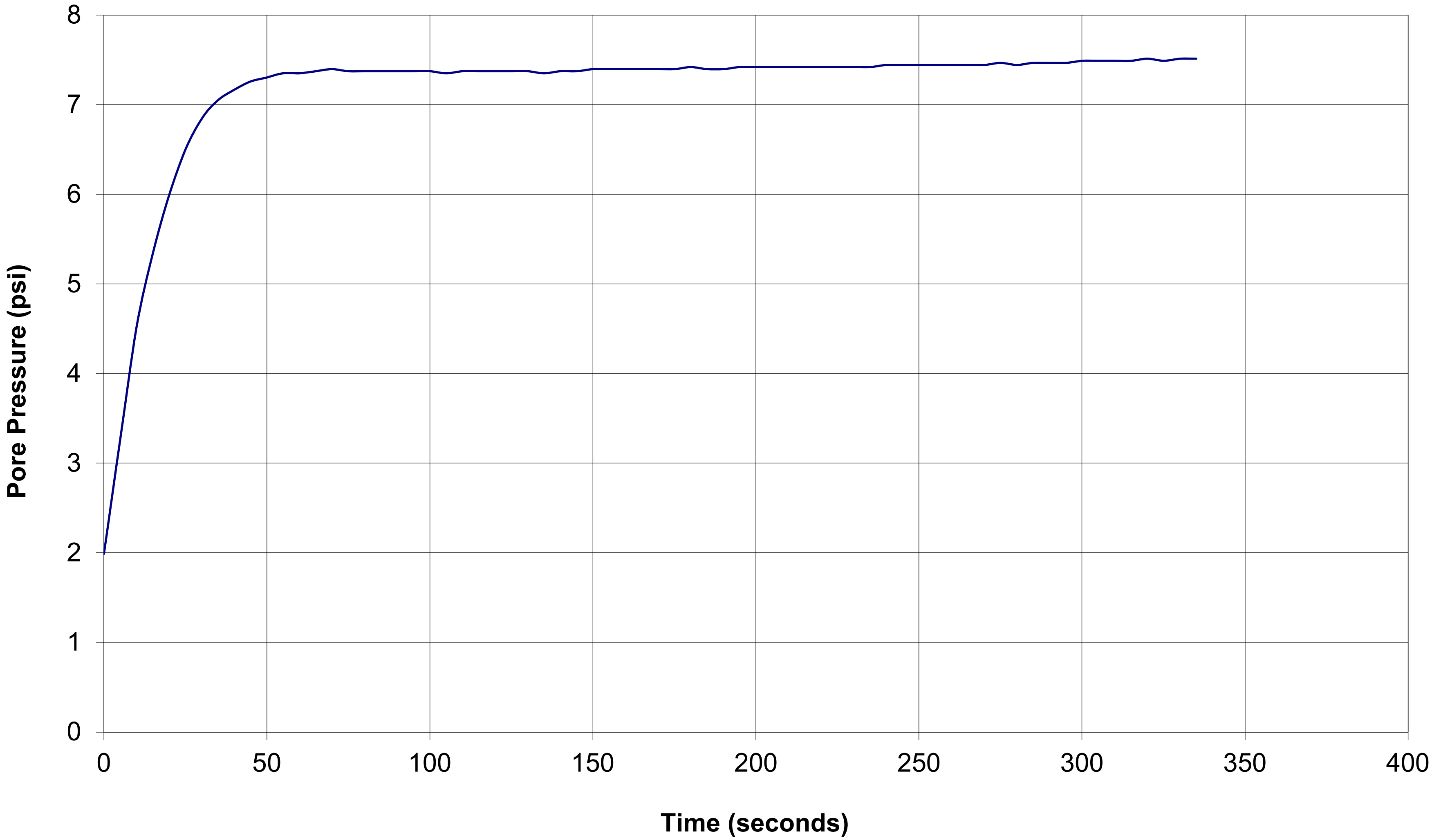
01/03/20

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
10.01	9.35	9.49	9.49	14.8000			
15.09	14.43	14.53	5.03	27.0000	12.2000	412.6	11.89
20.01	19.35	19.42	4.90	37.7000	10.7000	457.7	16.89
25.10	24.44	24.50	5.07	49.0500	11.3500	446.7	21.90
30.02	29.36	29.41	4.91	63.5000	14.4500	339.9	26.90
35.10	34.44	34.49	5.08	76.2500	12.7500	398.3	31.90
40.03	39.37	39.40	4.92	86.2000	9.9500	494.1	36.91
45.11	44.45	44.48	5.08	98.1500	11.9500	425.2	41.91
50.03	49.37	49.40	4.92	107.6500	9.5000	517.7	46.91
55.12	54.46	54.48	5.08	115.6000	7.9500	639.3	51.92
60.04	59.38	59.40	4.92	121.1000	5.5000	894.4	56.92
65.12	64.46	64.49	5.08	127.5500	6.4500	788.1	61.92
70.05	69.39	69.41	4.92	133.8000	6.2500	787.2	66.93
75.13	74.47	74.49	5.08	140.0500	6.2500	813.4	71.93

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-04
Depth (ft): 31.33
Site: Laney College
Engineer: Reza Rahimnejad



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-05
Depth (ft): 41.17
Site: Laney College
Engineer: Reza Rahimnejad

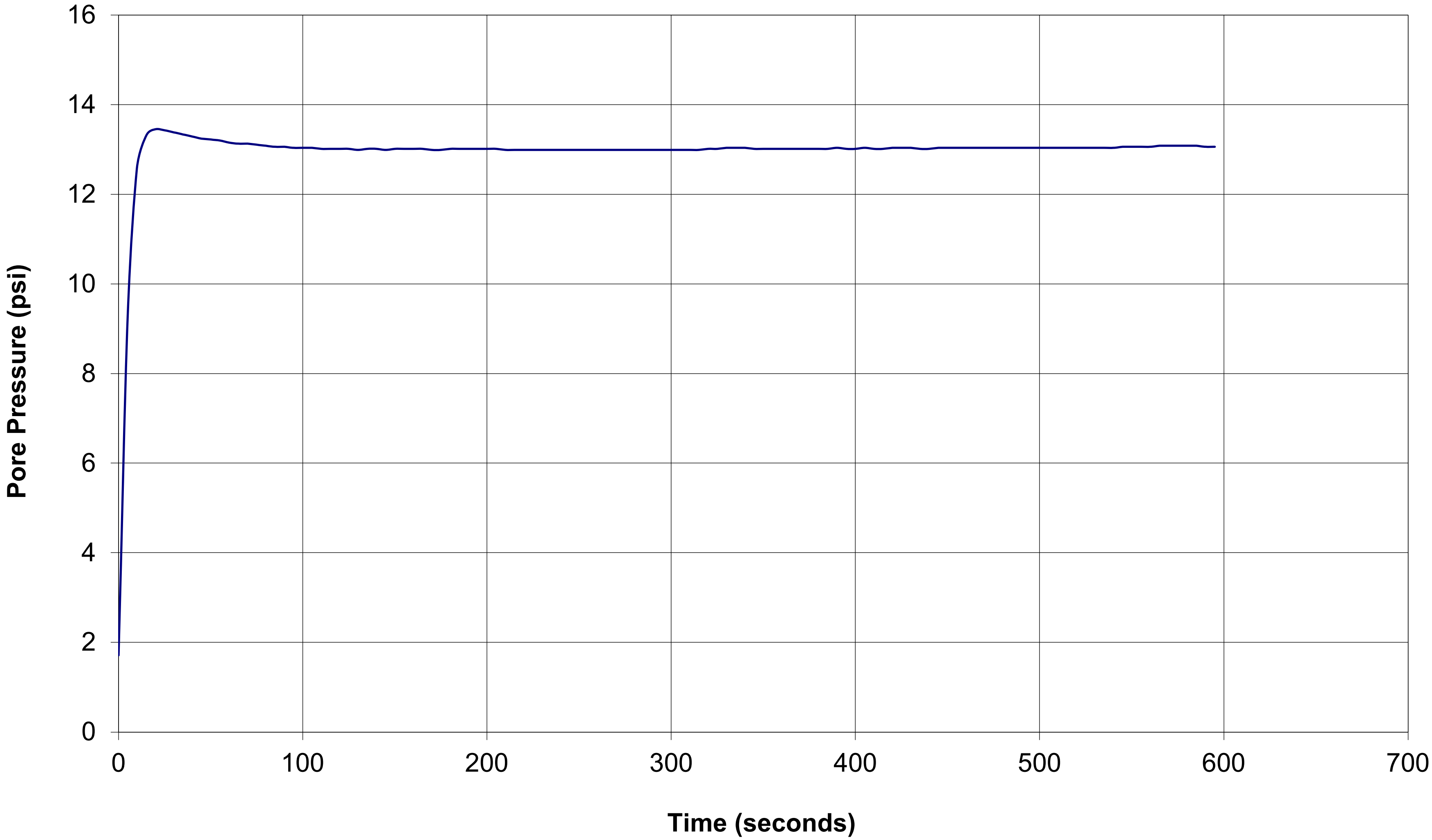
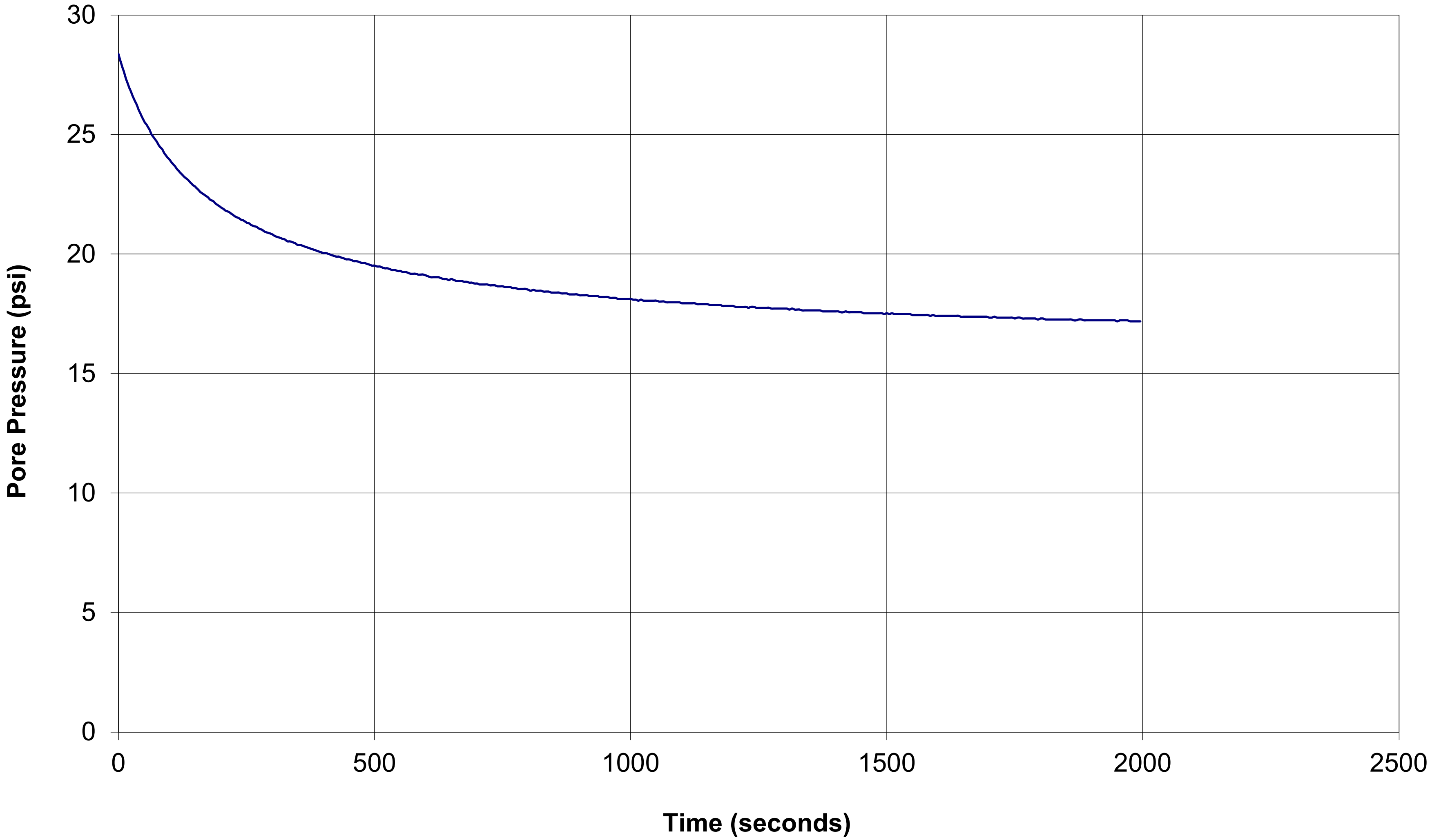


PLATE A-49

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-08
Depth (ft): 51.67
Site: Laney College
Engineer: Reza Rahimnejad



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: SCPT-07
Depth (ft): 57.58
Site: Laney College
Engineer: Reza Rahimnejad

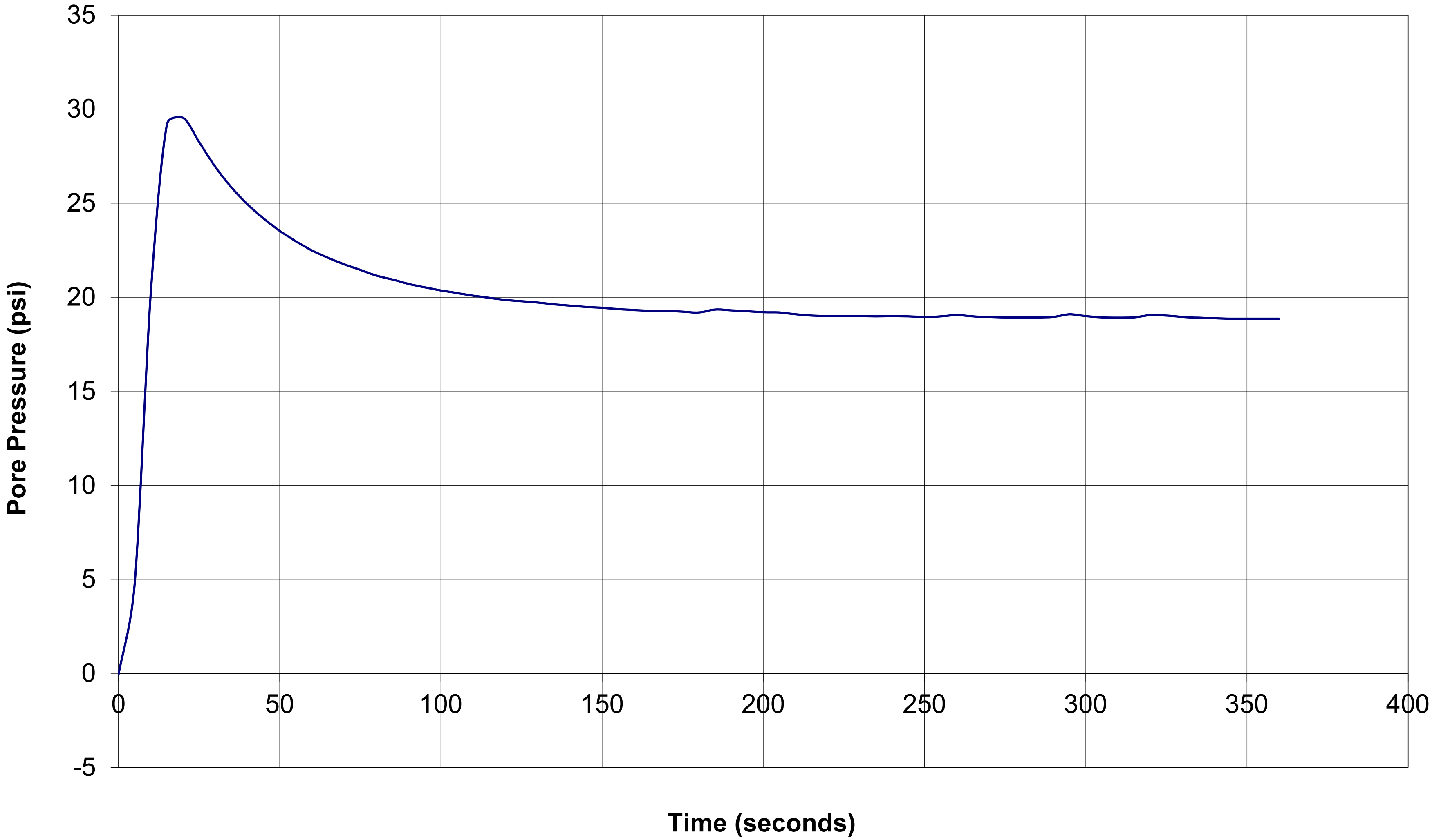


PLATE A-51

Table 1: Cone Penetration Testing Summary

CPT Sounding Identification	Date	Termination Depth (ft)	Depth of Soil Samples (ft)	Depth of Groundwater Samples (ft)	Depth of Pore Pressure Dissipation Tests (ft)
CPT-16	11/22/2022	100.23	-	-	48.06
CPT-17	11/22/2022	100.23	-	-	-
CPT-18	11/18/2022	80.22	-	-	-
CPT-19	11/17/2022	65.29	-	-	30.02
CPT-20	11/17/2022	80.22	-	-	-
CPT-21	11/18/2022	80.38	-	-	-
CPT-22	11/17/2022	45.28	-	-	-
CPT-23	11/17/2022	71.03	-	-	-
CPT-24	11/17/2022	80.22	-	-	-
CPT-25	11/18/2022	80.22	-	-	27.23

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-16

CLIENT: FUGRO USA LAND, INC.

SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24

Total depth: 100.23 ft, Date: 11/22/2022

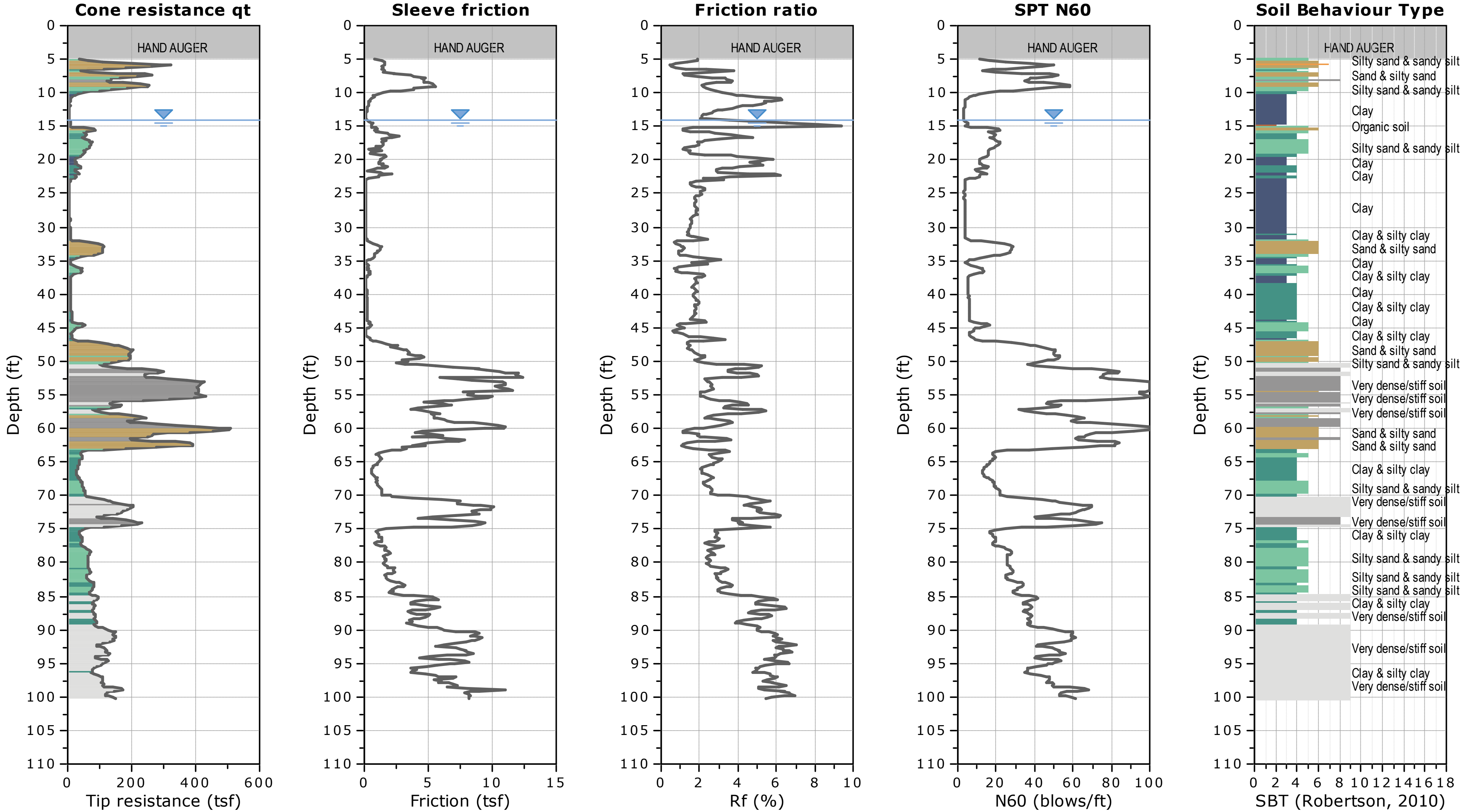


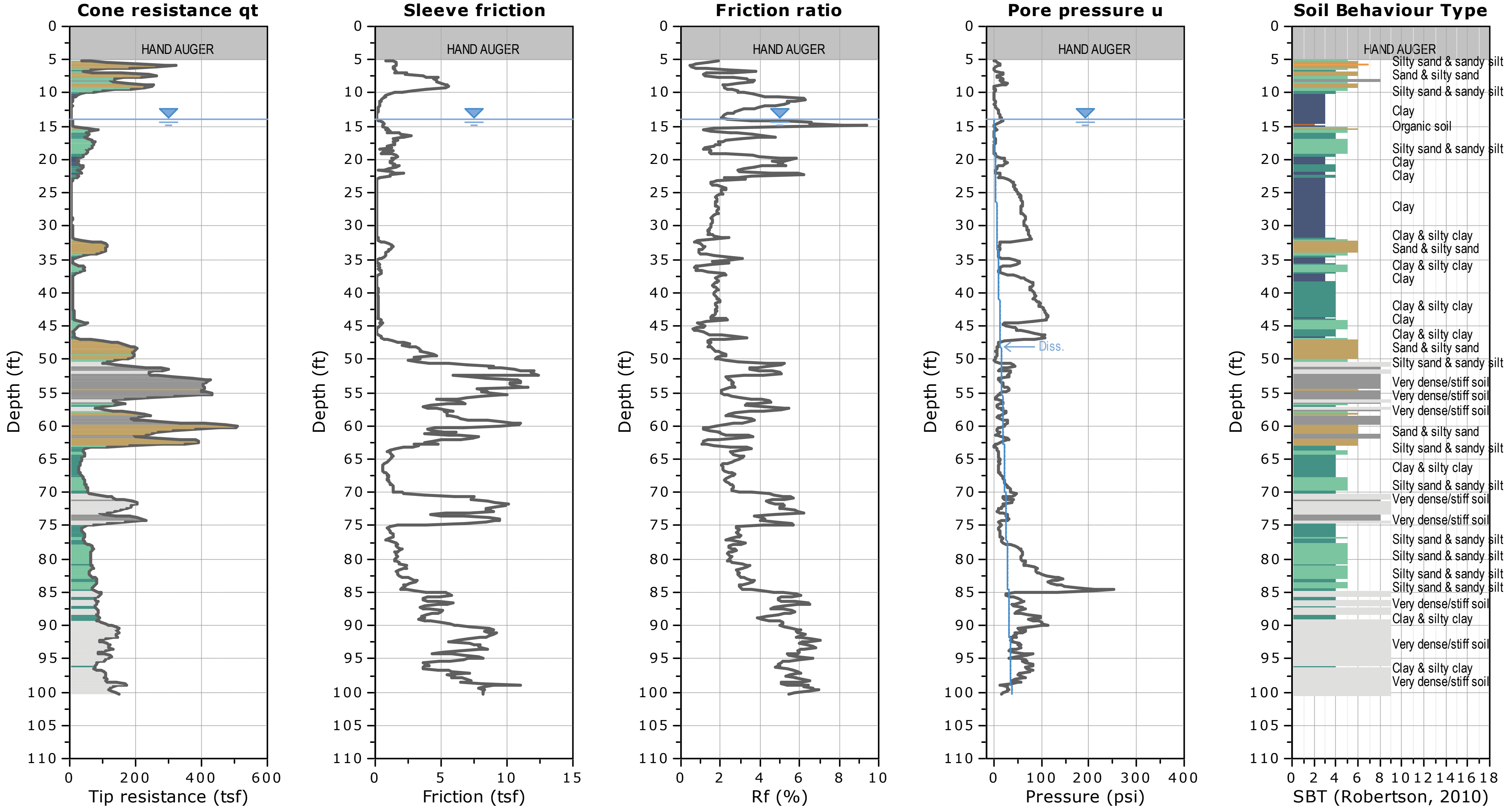
PLATE A-53

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-16

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 100.23 ft, Date: 11/22/2022

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-54

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-17

CLIENT: FUGRO USA LAND, INC.

SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24

Total depth: 100.23 ft, Date: 11/22/2022

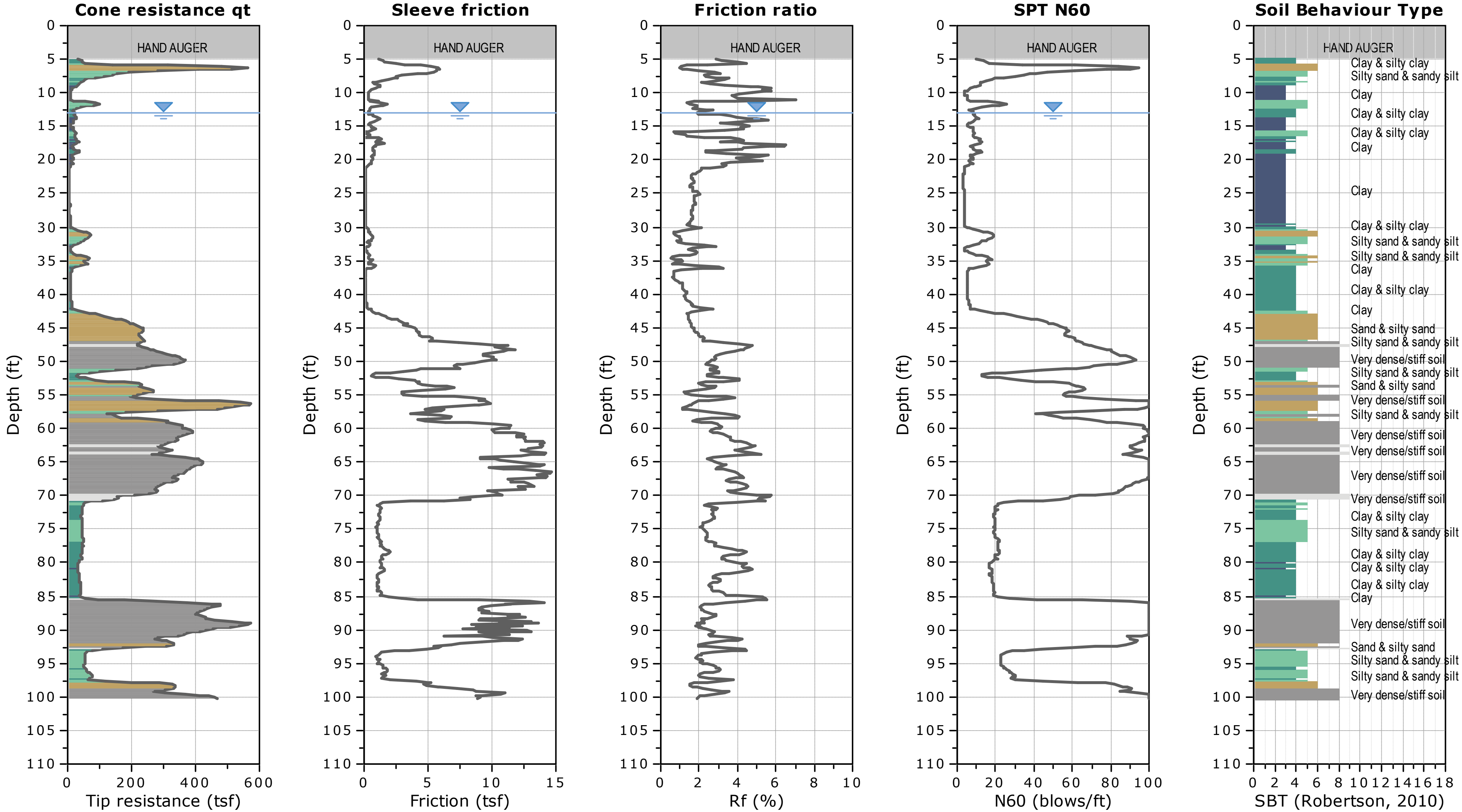


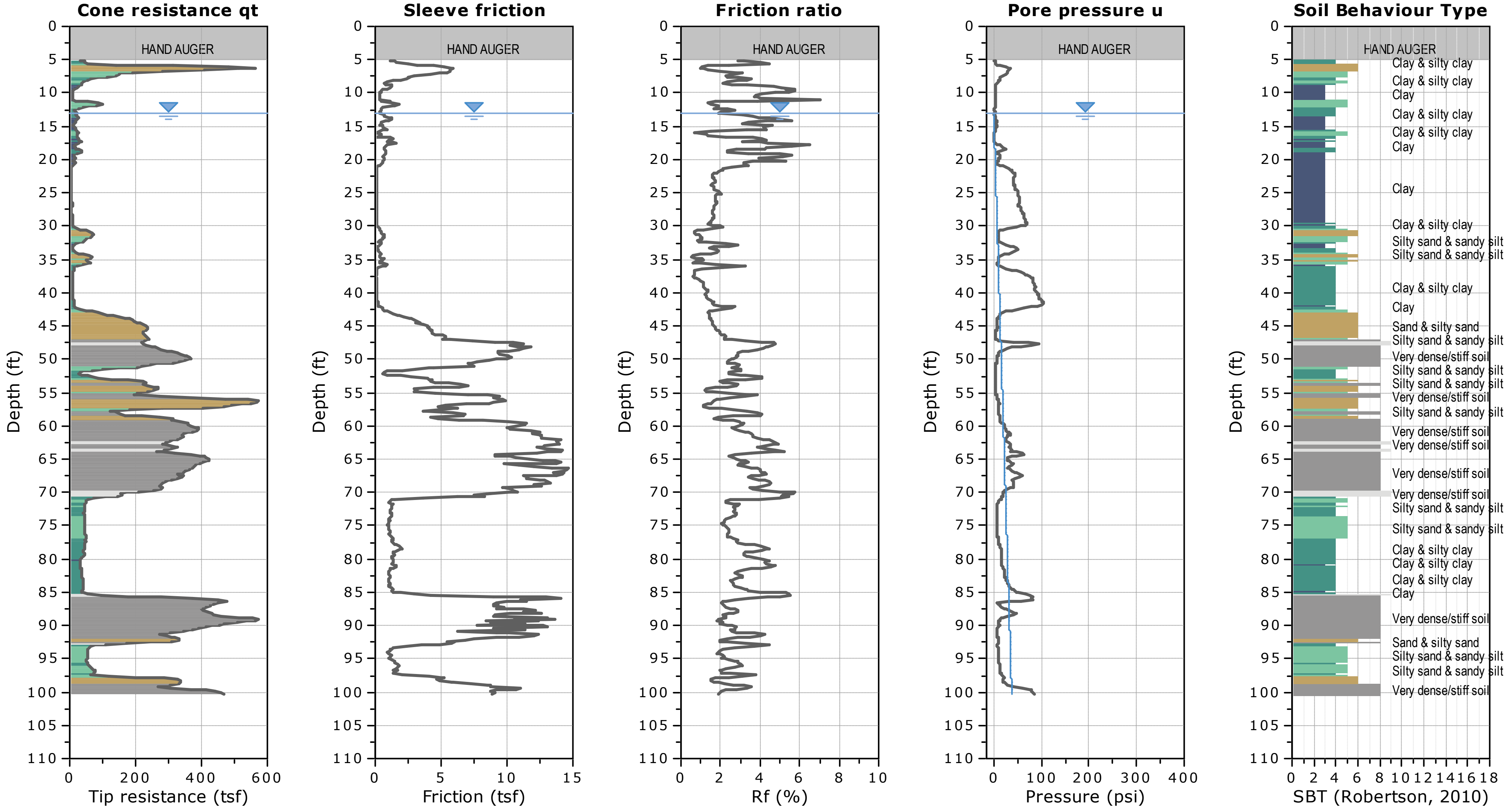
PLATE A-55

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-17

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 100.23 ft, Date: 11/22/2022

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-56

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-18

CLIENT: FUGRO USA LAND, INC.

SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT

Cone ID: GDC-24

Total depth: 80.22 ft, Date: 11/18/2022

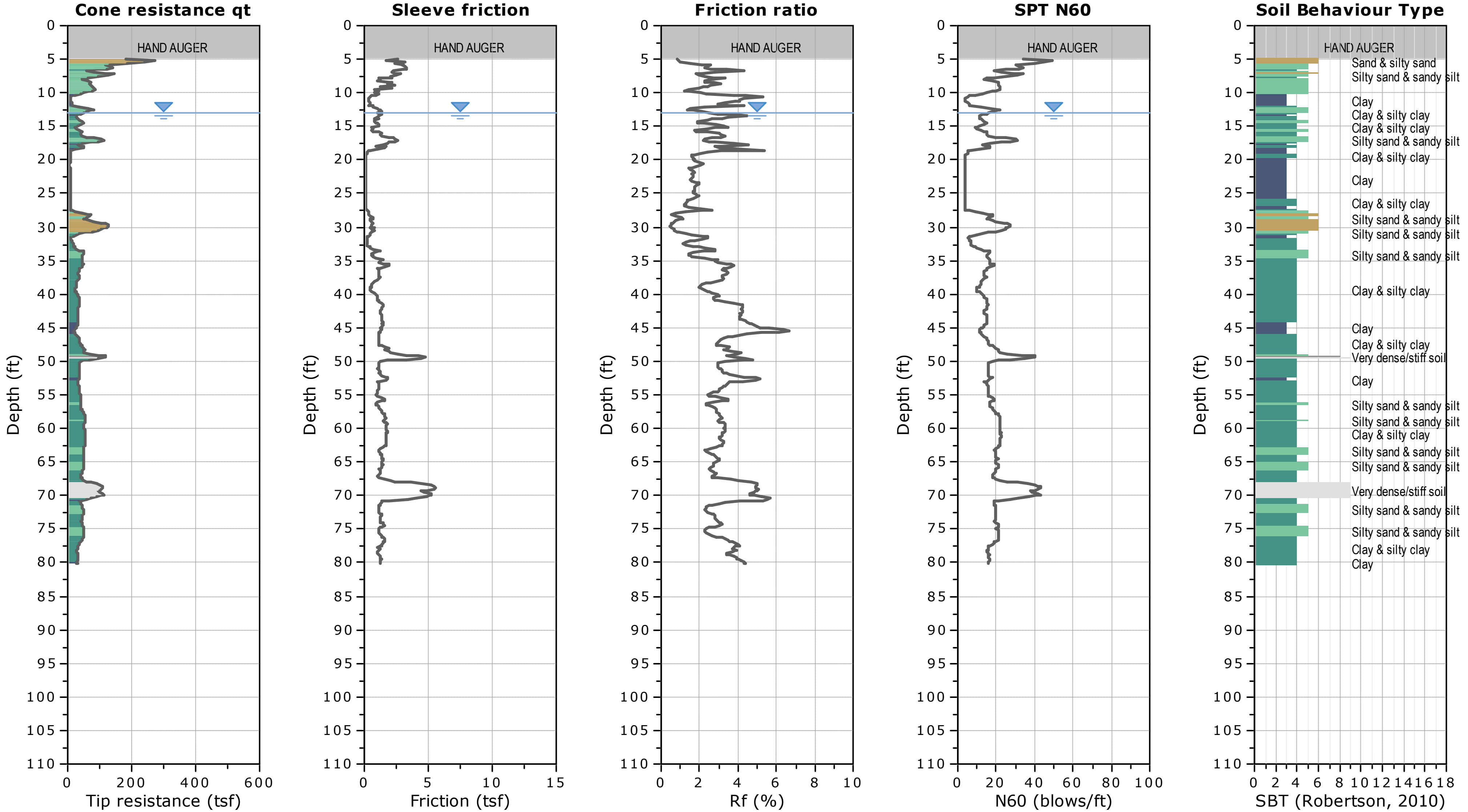


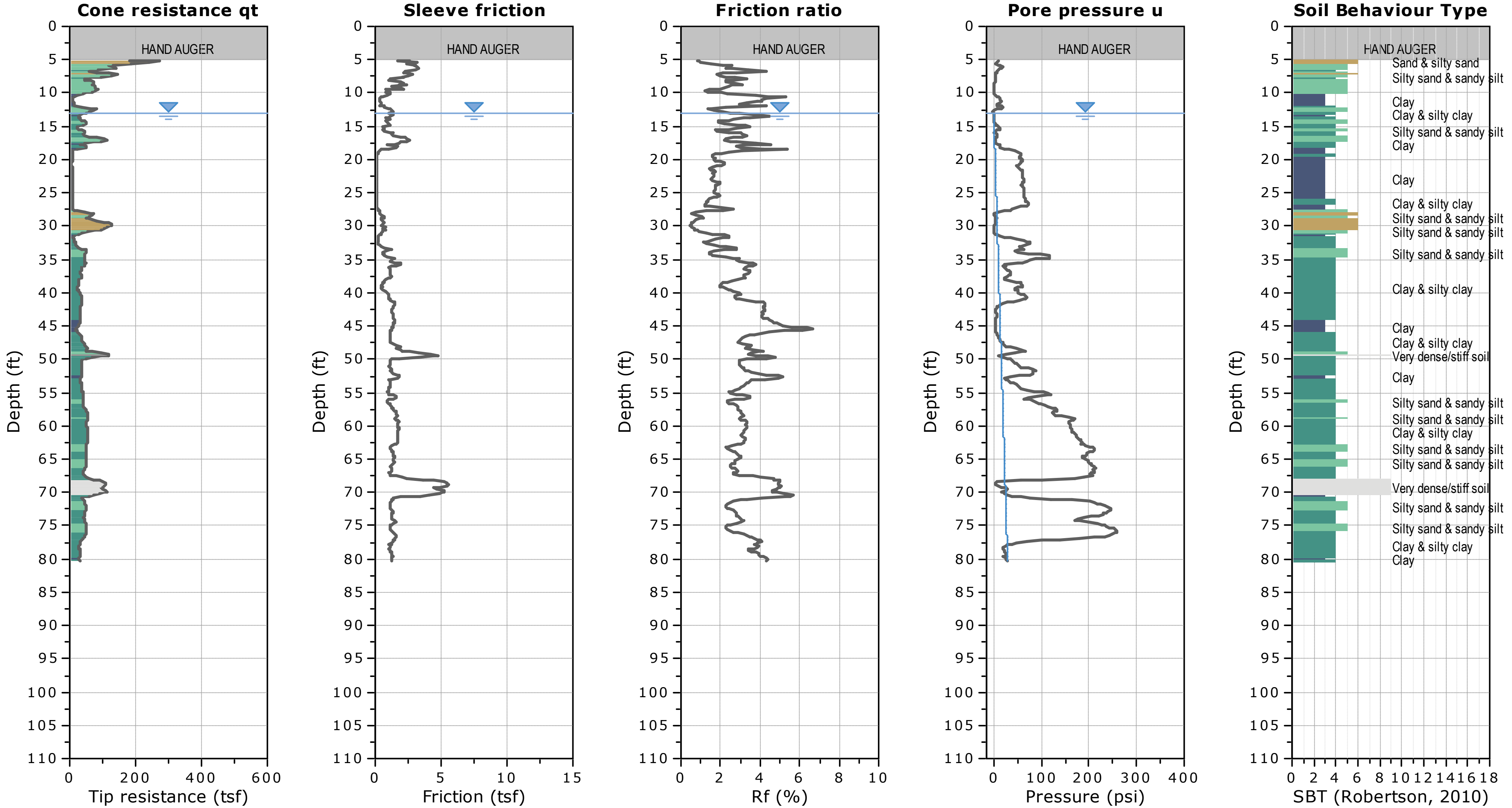
PLATE A-57

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-18

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 80.22 ft, Date: 11/18/2022



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-58

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-19

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 65.29 ft, Date: 11/17/2022

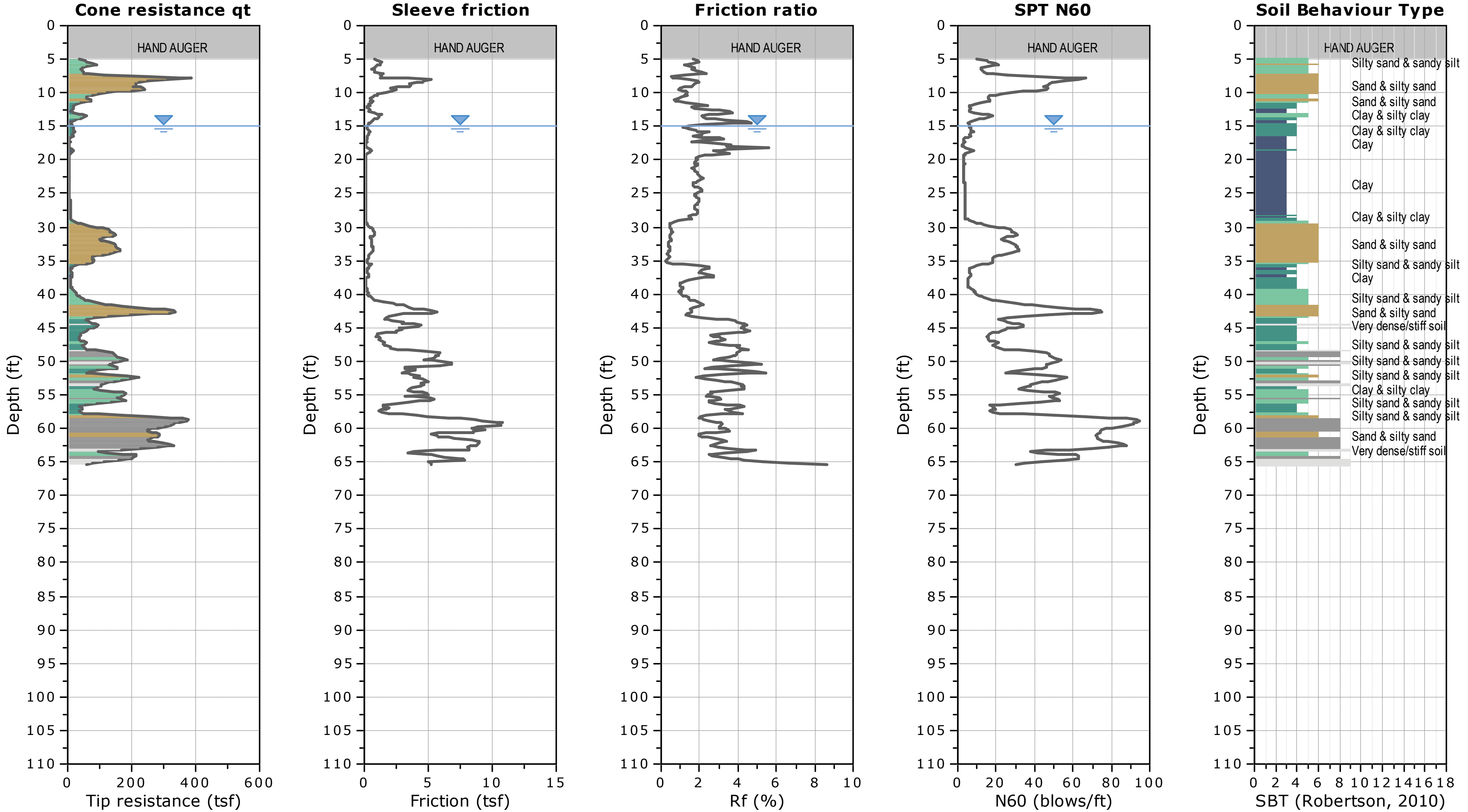


PLATE A-59

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-19

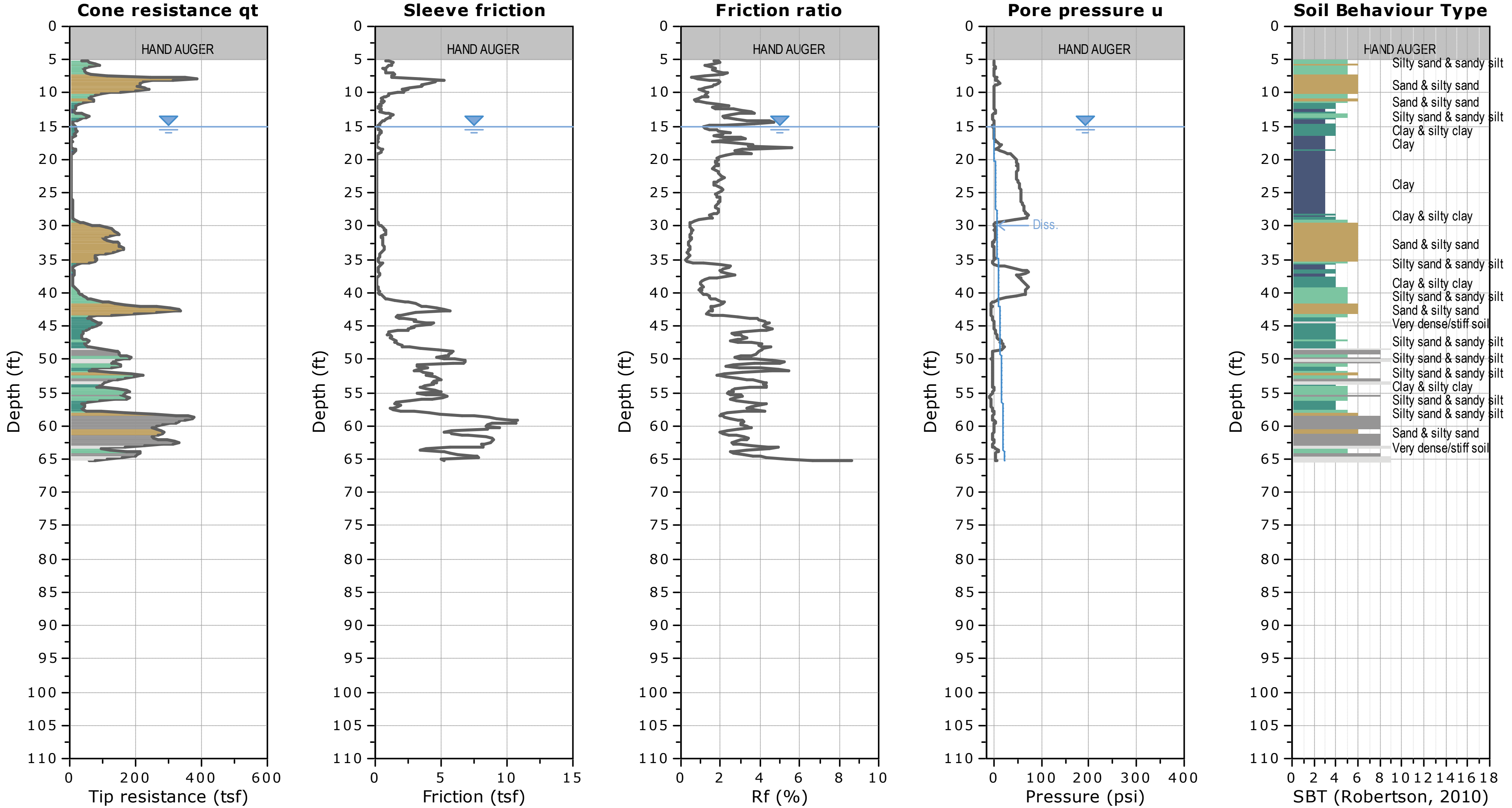
CLIENT: FUGRO USA LAND, INC.

SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT

Cone ID: GDC-24

Total depth: 65.29 ft, Date: 11/17/2022



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-60

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-20

CLIENT: FUGRO USA LAND, INC.

SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT

Cone ID: GDC-24

Total depth: 80.22 ft, Date: 11/17/2022

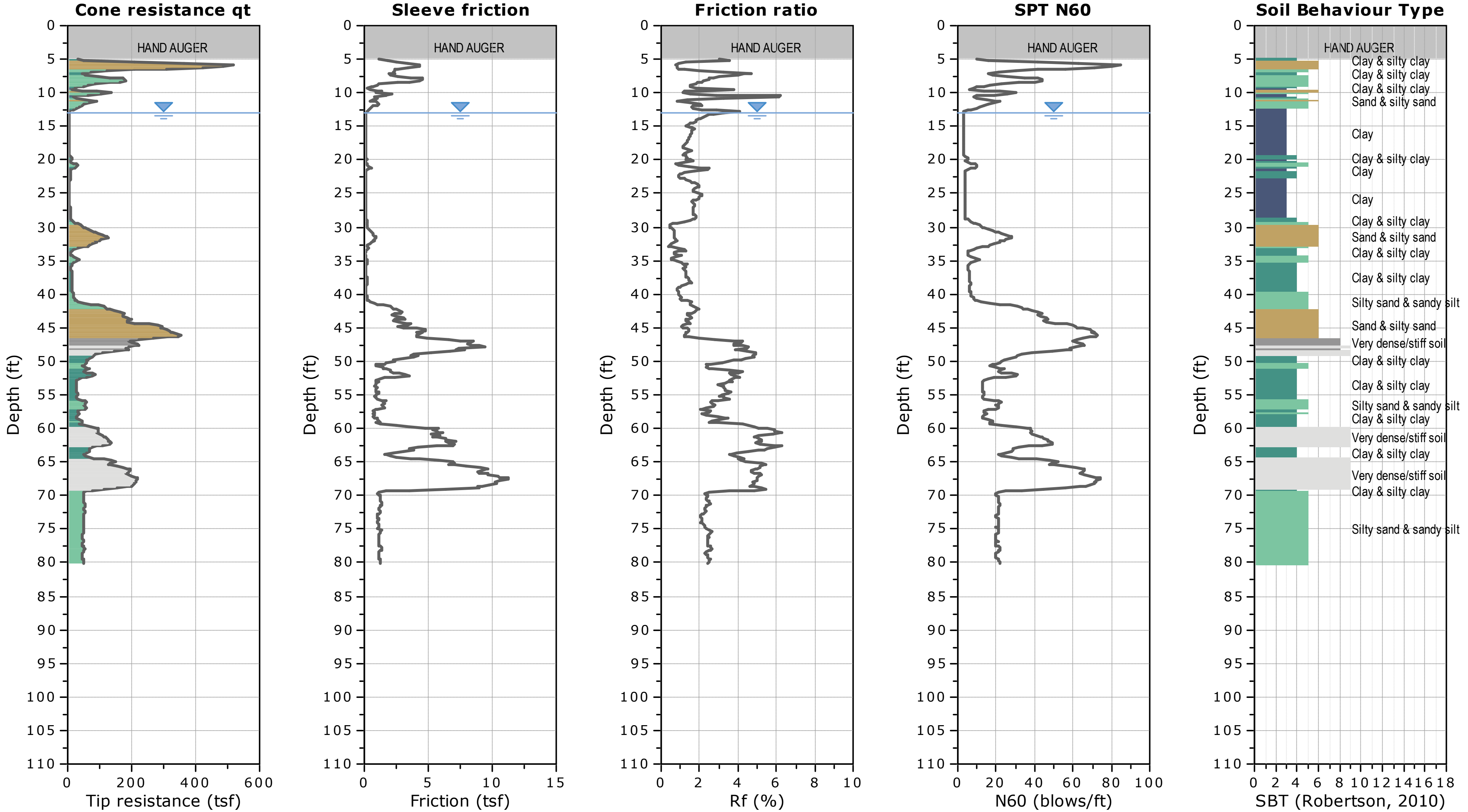


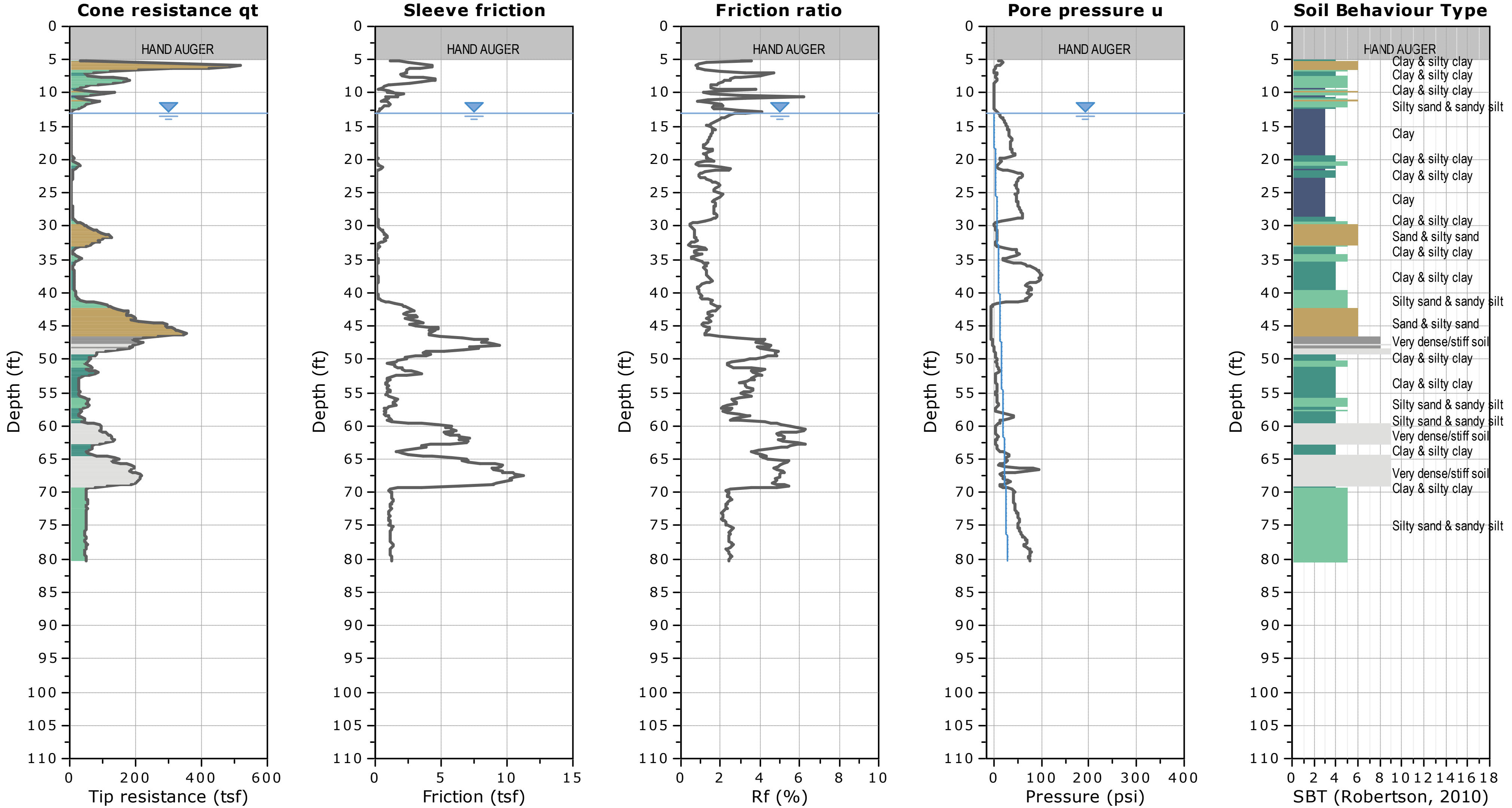
PLATE A-61

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-20

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 80.22 ft, Date: 11/17/2022



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-62

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-21

CLIENT: FUGRO USA LAND, INC.

SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 80.38 ft, Date: 11/18/2022

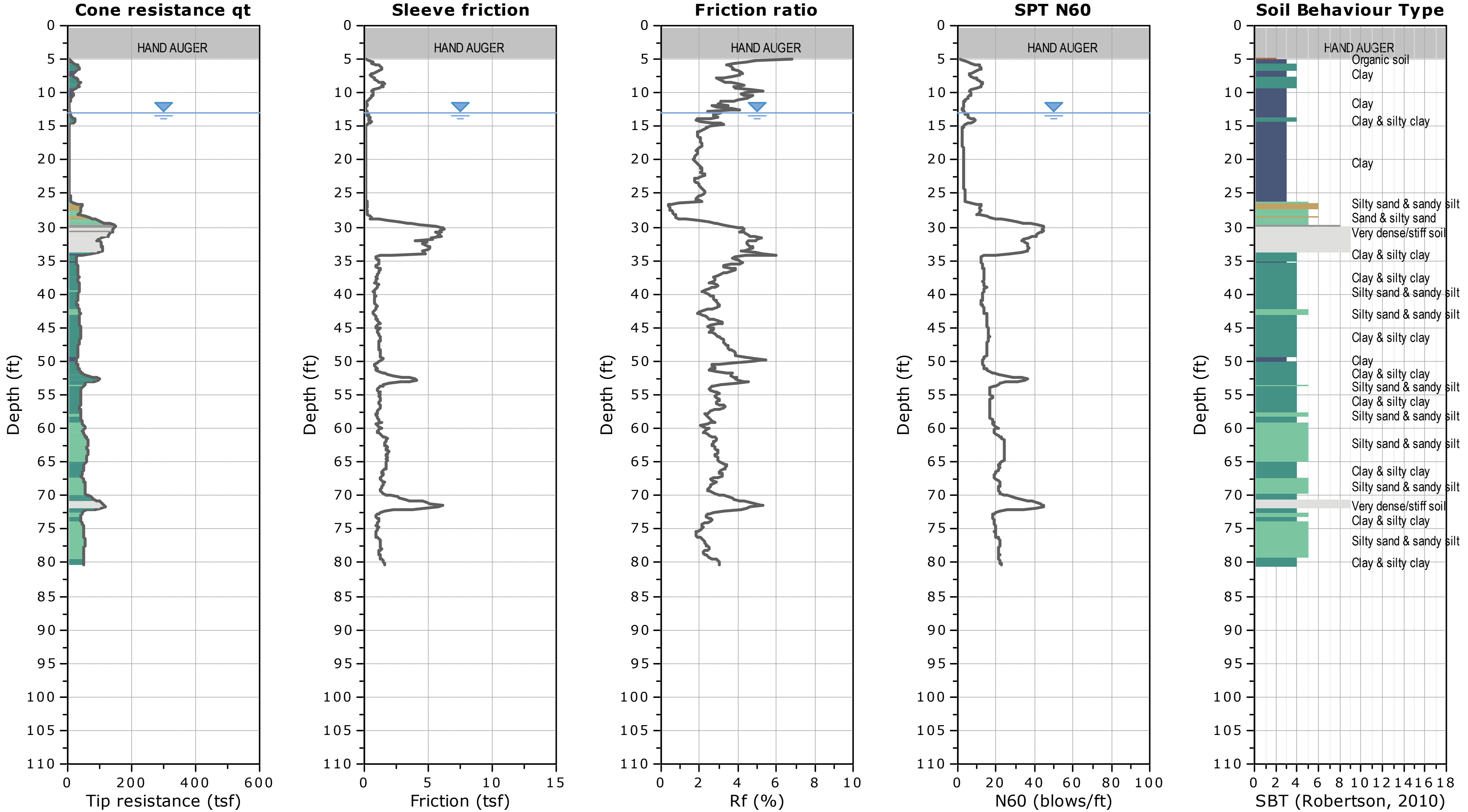


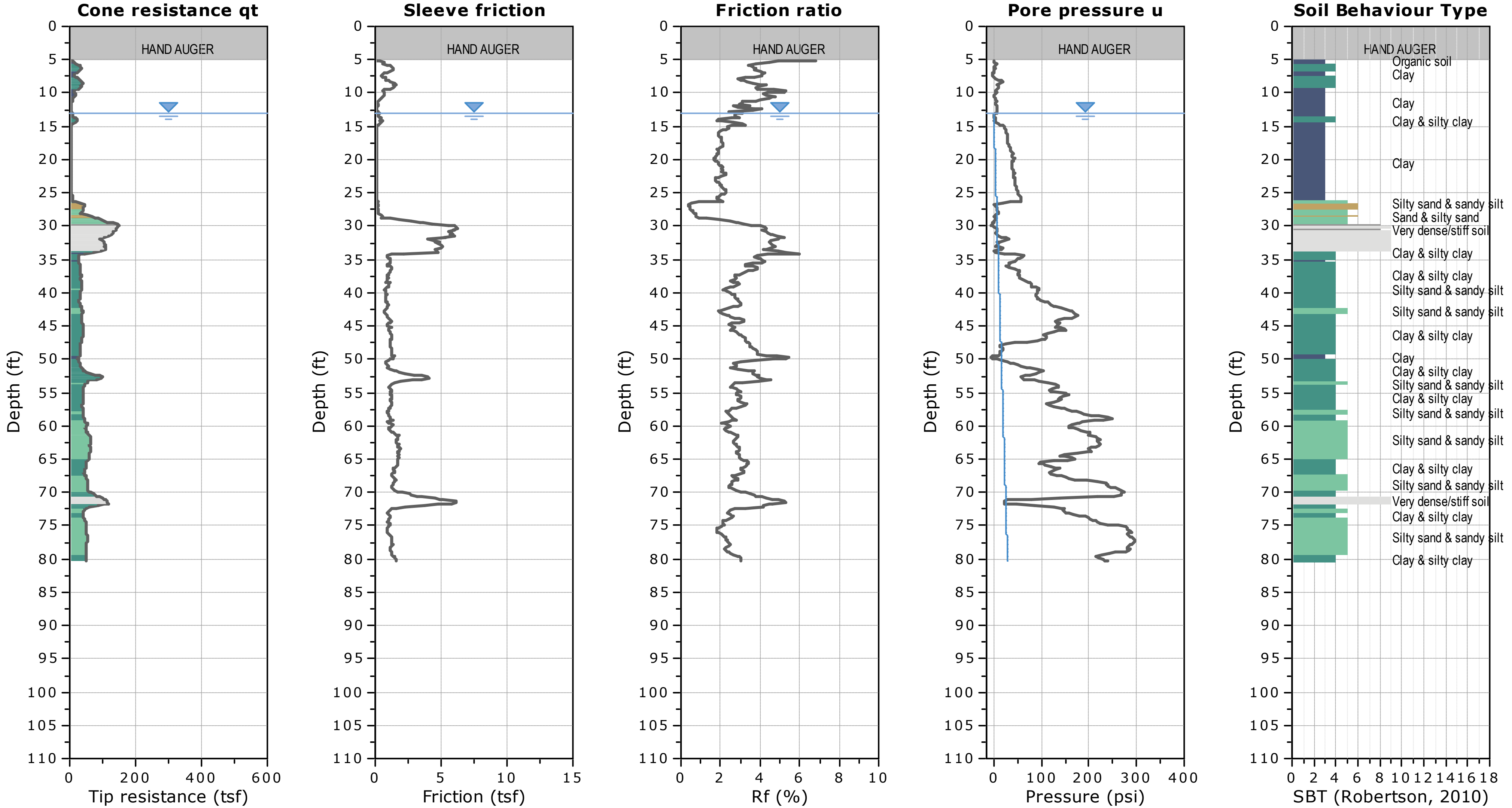
PLATE A-63

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-21

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 80.38 ft, Date: 11/18/2022



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-64

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-22

CLIENT: FUGRO USA LAND, INC.

SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT

Cone ID: GDC-24

Total depth: 45.28 ft, Date: 11/17/2022

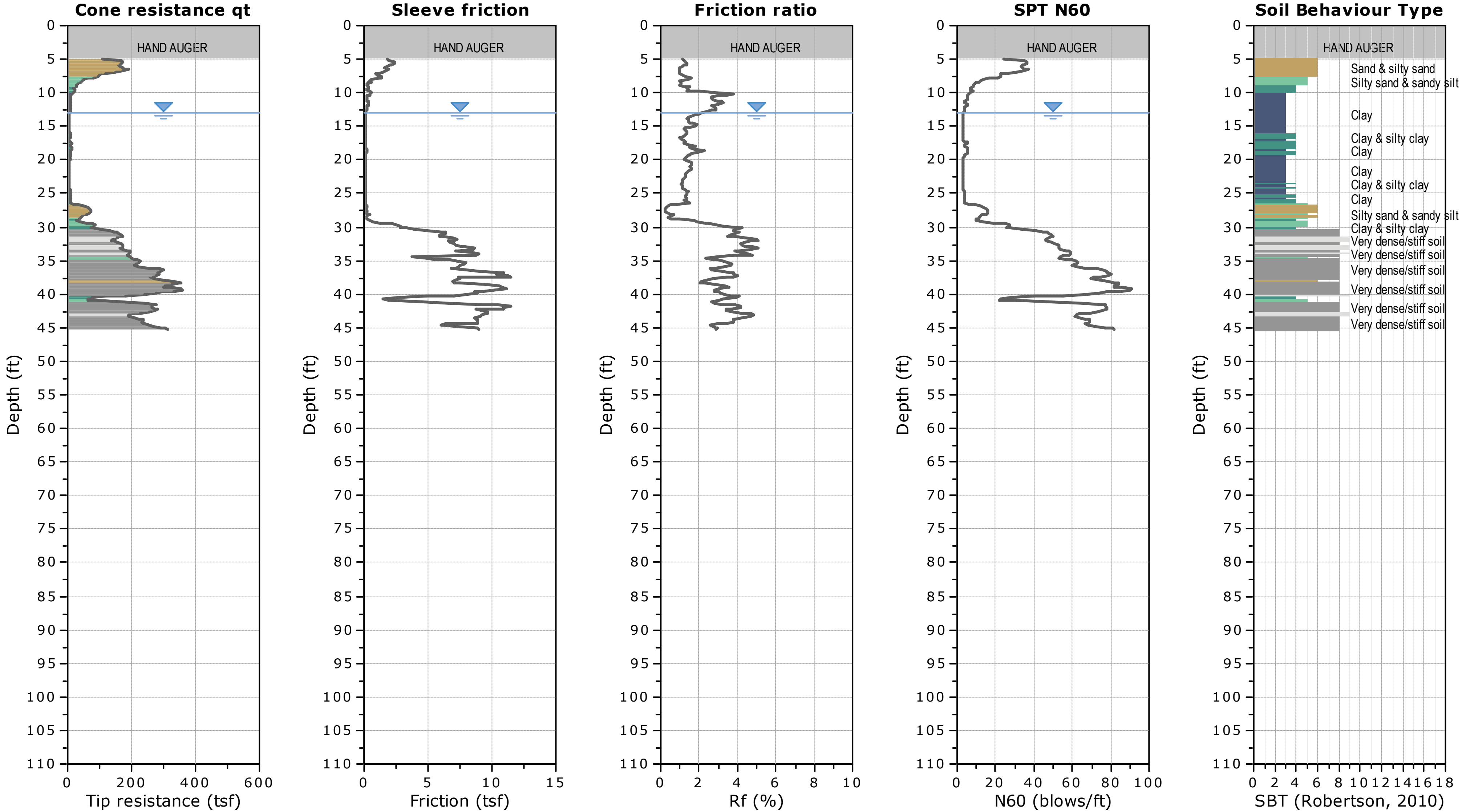


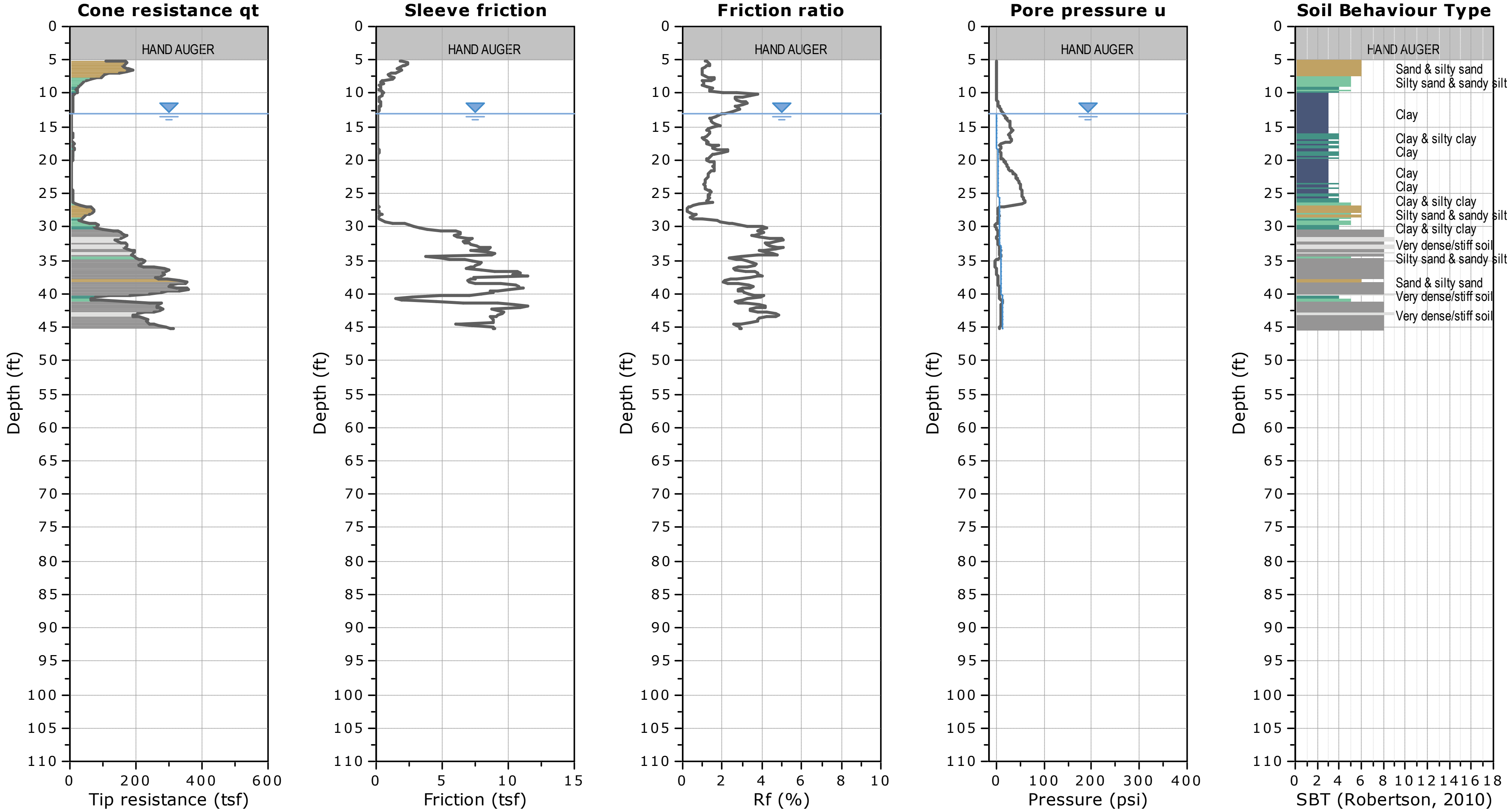
PLATE A-65

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-22

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 45.28 ft, Date: 11/17/2022

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-66

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-23

FIELD REP: ABDUL SADAT
Cone ID: GDC-24

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

Total depth: 71.03 ft, Date: 11/17/2022

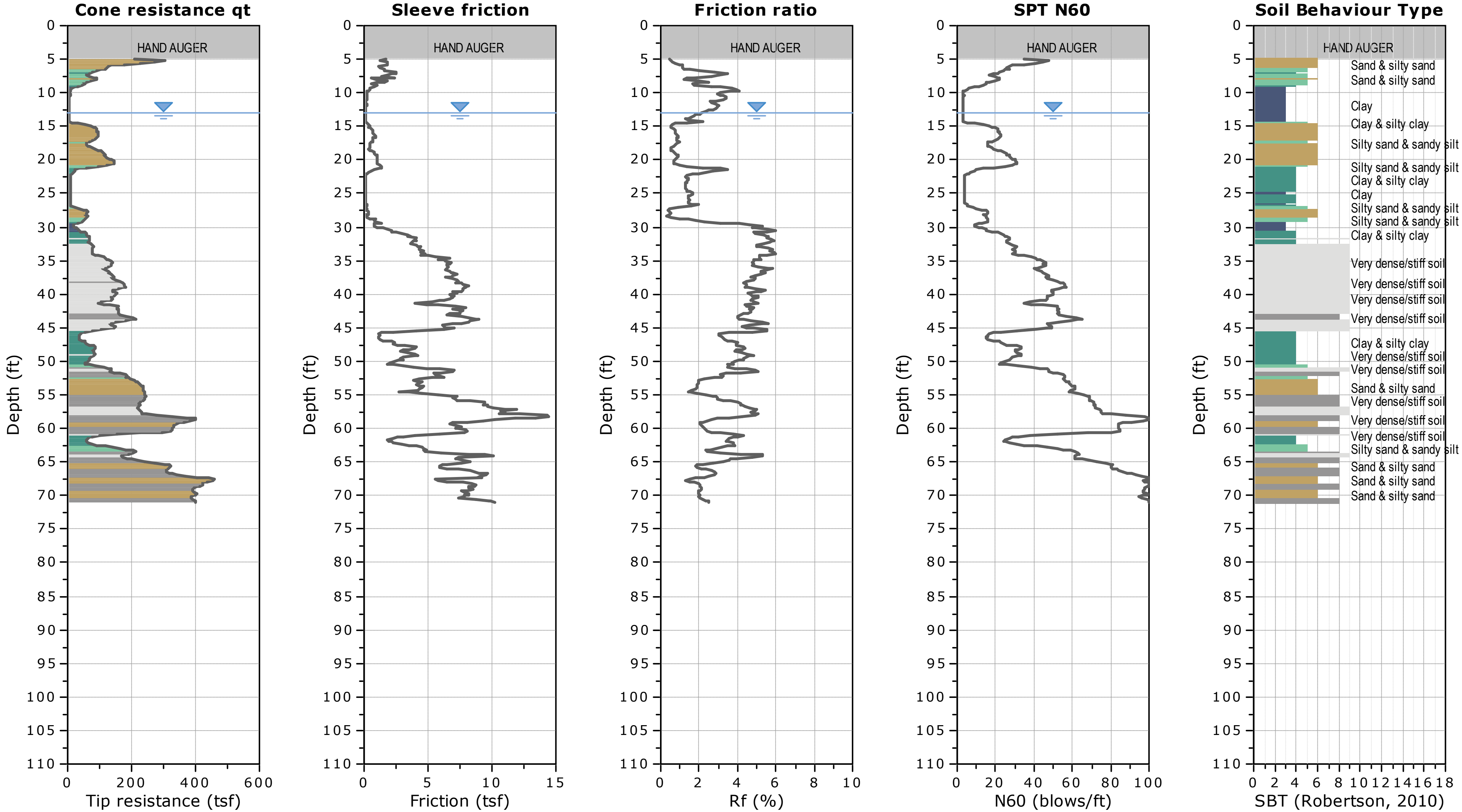


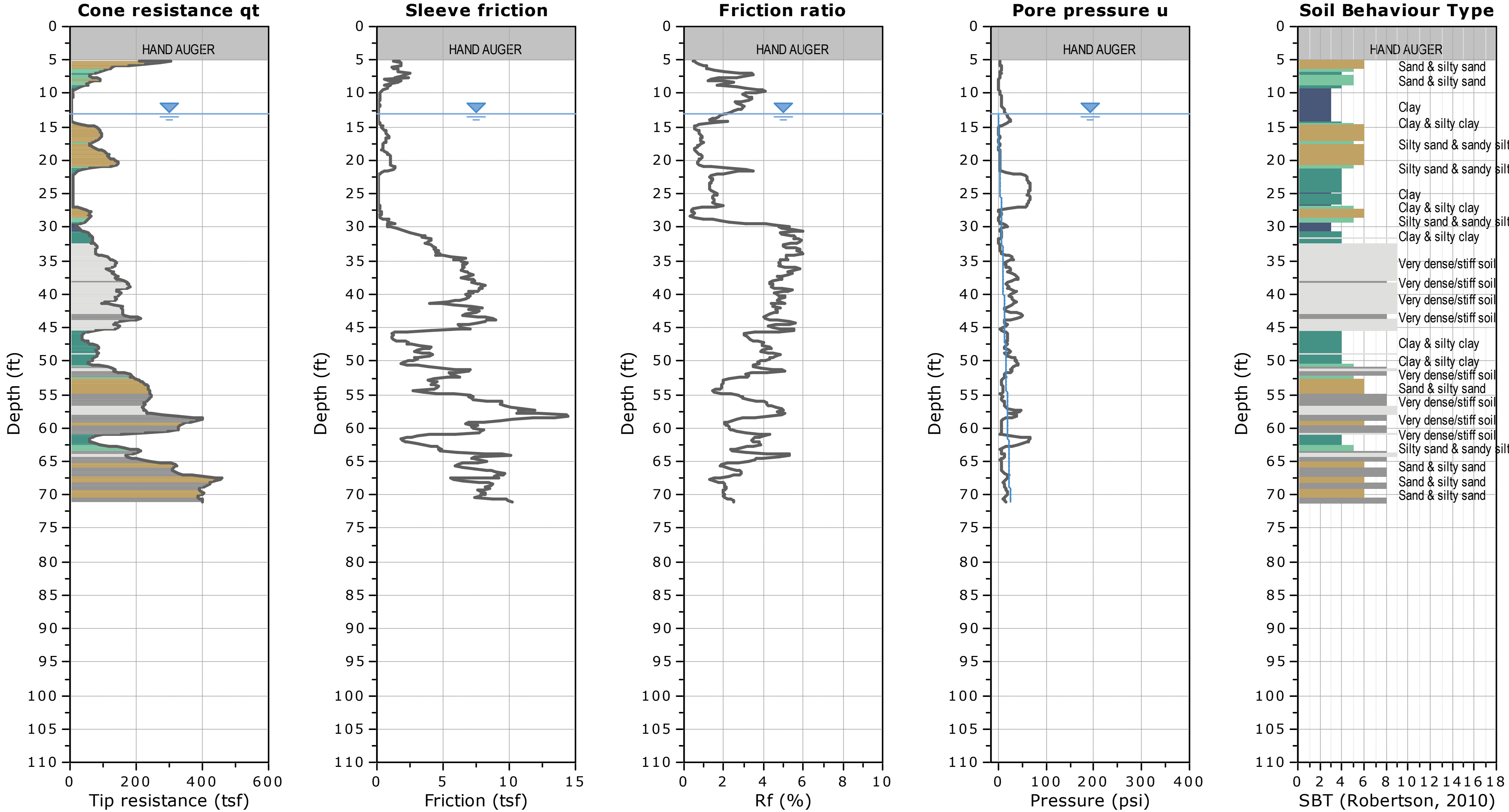
PLATE A-67

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-23

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 71.03 ft, Date: 11/17/2022



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-68

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-24

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 80.22 ft, Date: 11/17/2022

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

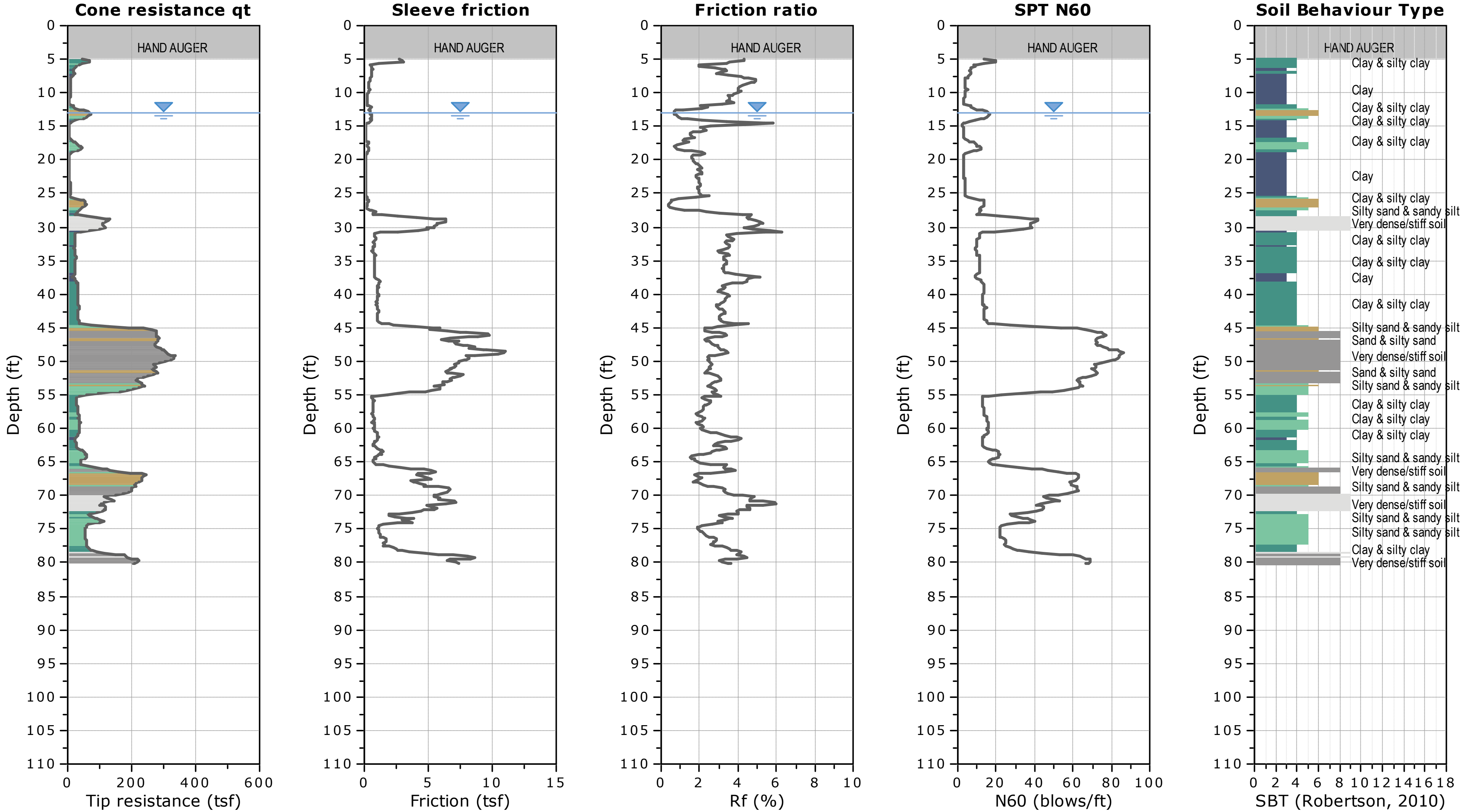


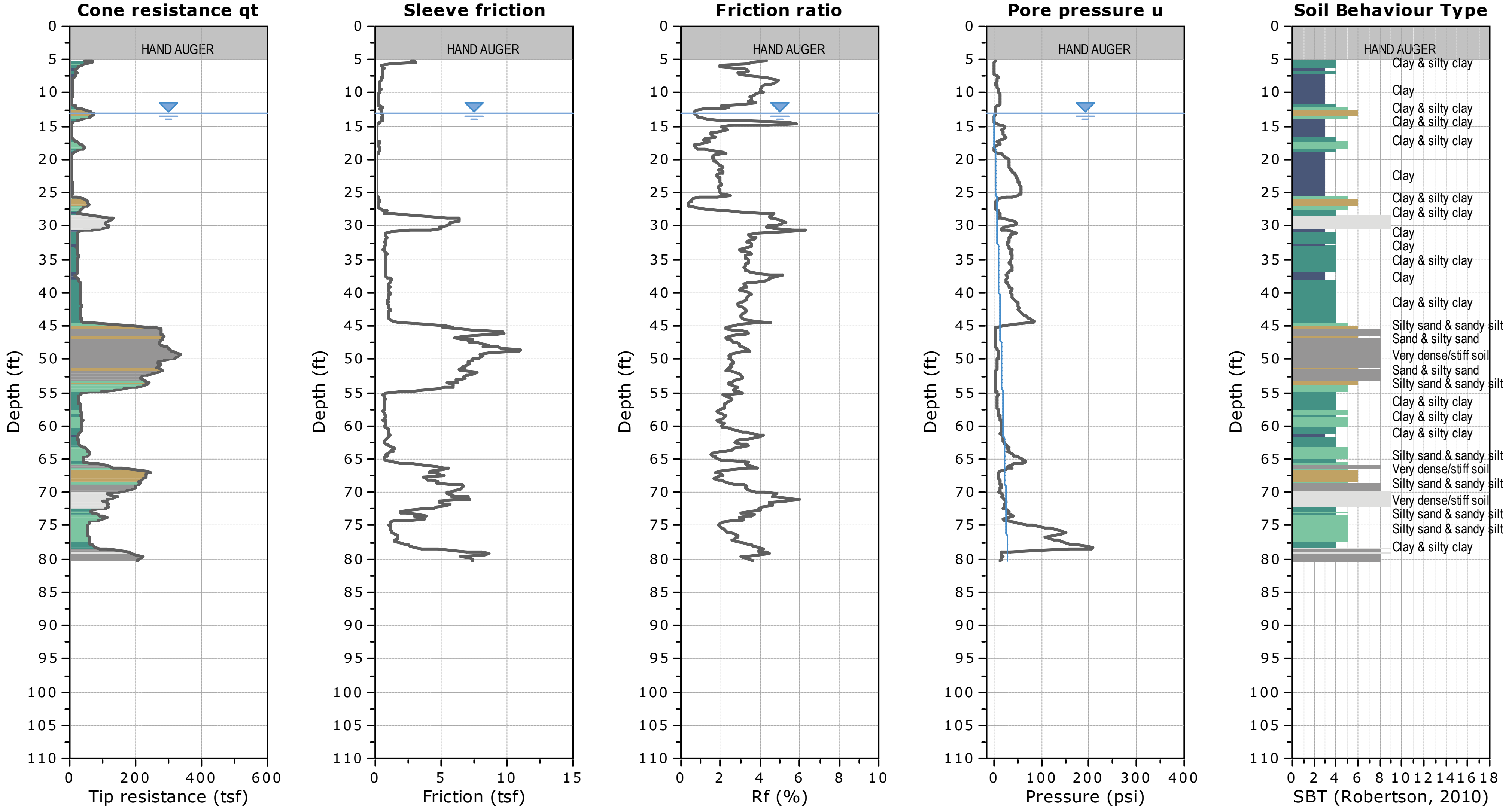
PLATE A-69

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-24

FIELD REP: ABDUL SADAT
Cone ID: GDC-24
Total depth: 80.22 ft, Date: 11/17/2022

CLIENT: FUGRO USA LAND, INC.
SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA



WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-70

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-25

CLIENT: FUGRO USA LAND, INC.

SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT
Cone ID: GDC-24

Total depth: 80.22 ft, Date: 11/18/2022

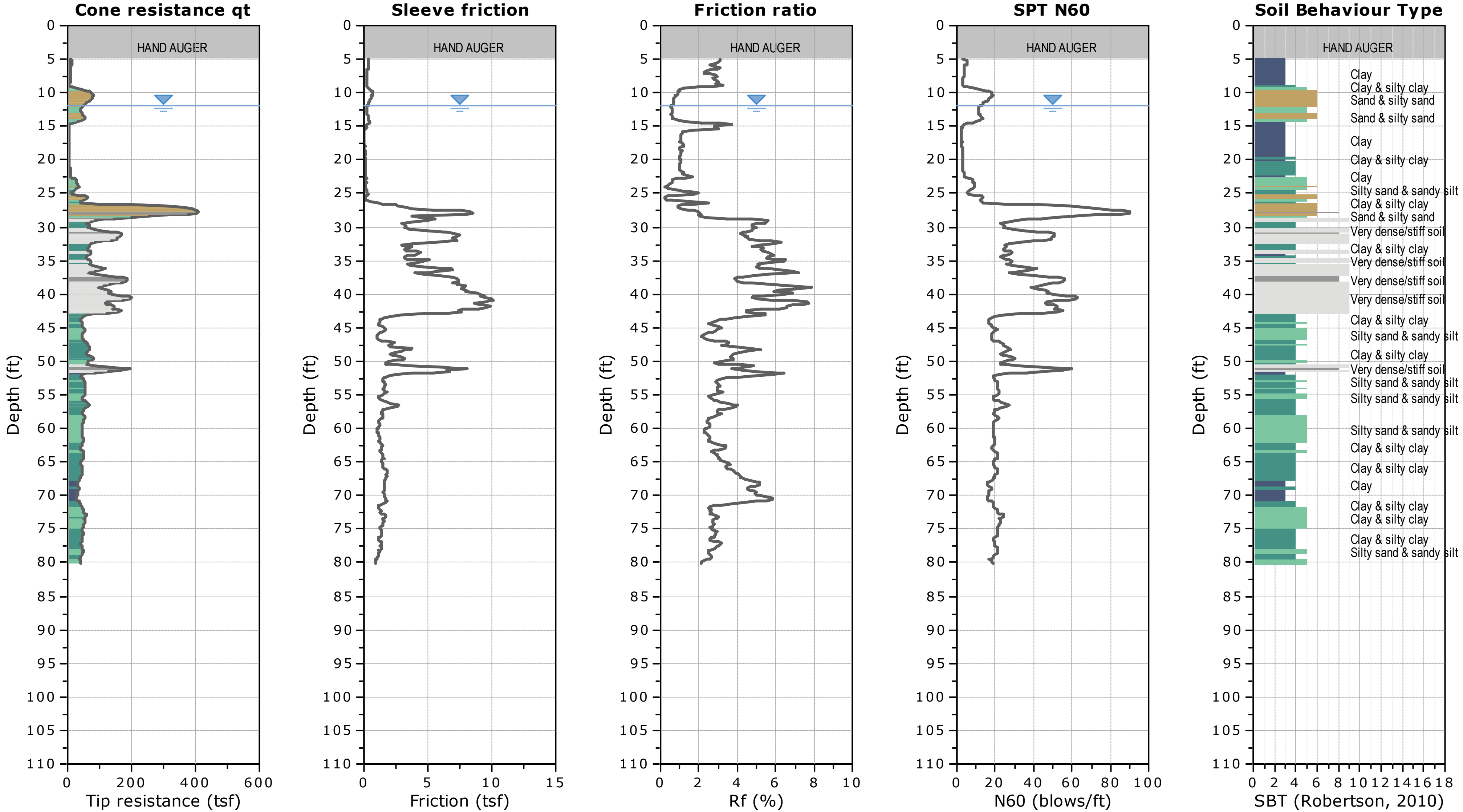


PLATE A-71

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-25

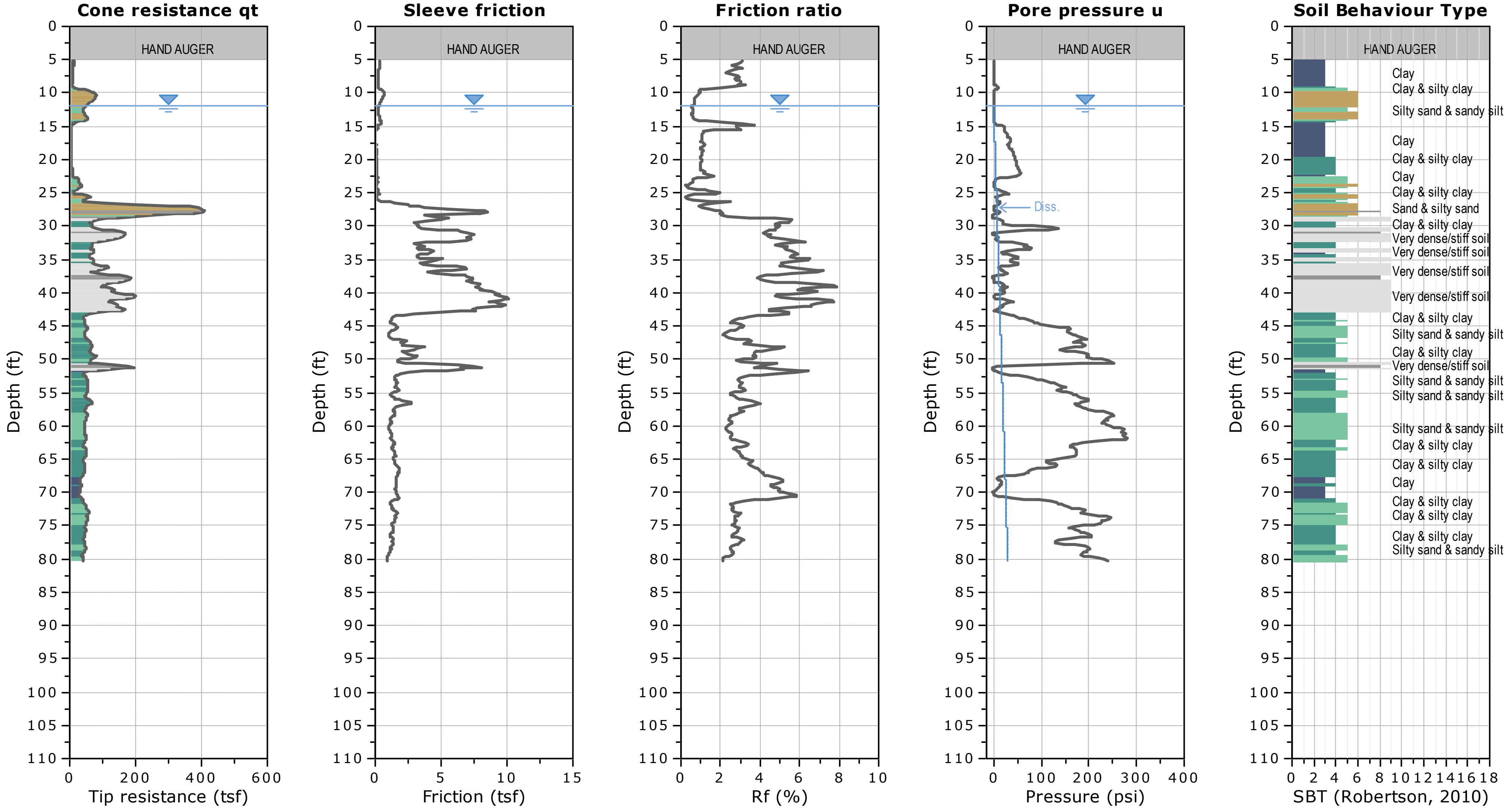
CLIENT: FUGRO USA LAND, INC.

SITE: LANEY COLLEGE LLRC BLDG, OAKLAND, CA

FIELD REP: ABDUL SADAT

Cone ID: GDC-24

Total depth: 80.22 ft, Date: 11/18/2022



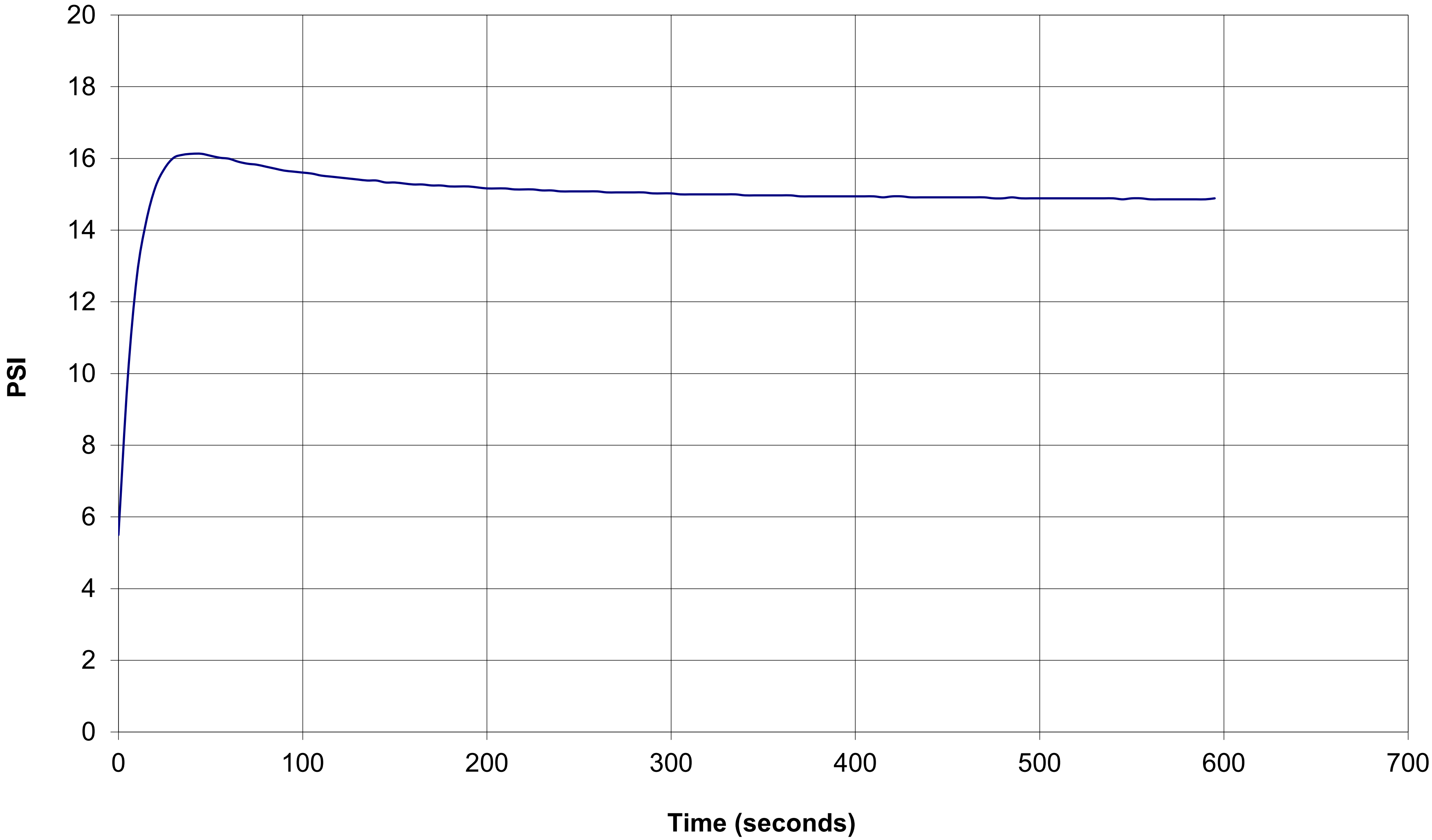
WATER TABLE FOR ESTIMATING PURPOSES ONLY

PLATE A-72

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-16
Depth (ft): 48.06
Site: Laney College
Engineer: Abdul Sadat



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-19
Depth (ft): 30.02
Site: Laney College
Engineer: Abdul Sadat

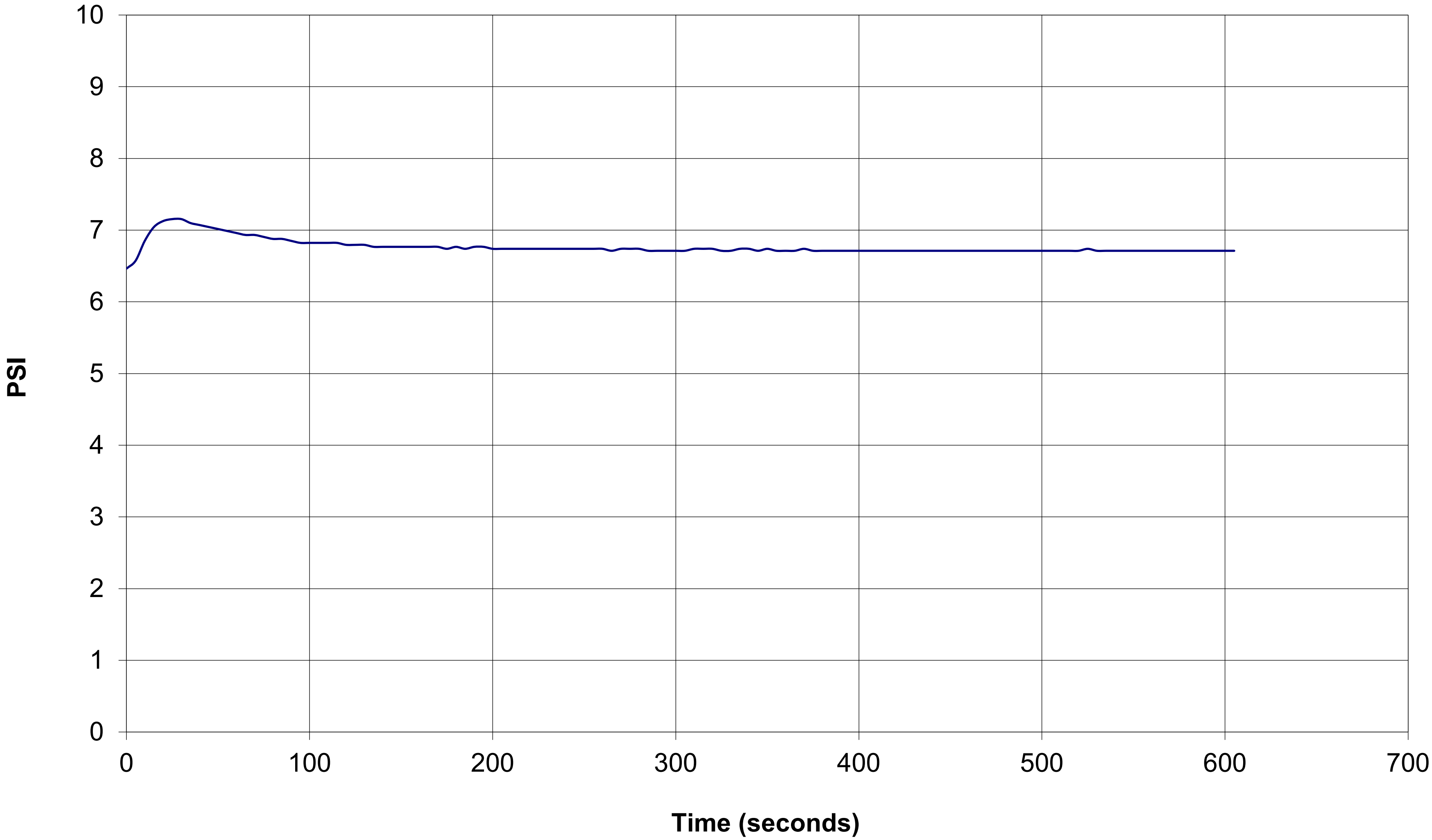


PLATE A-74

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-25
Depth (ft): 27.23
Site: Laney College
Engineer: Abdul Sadat

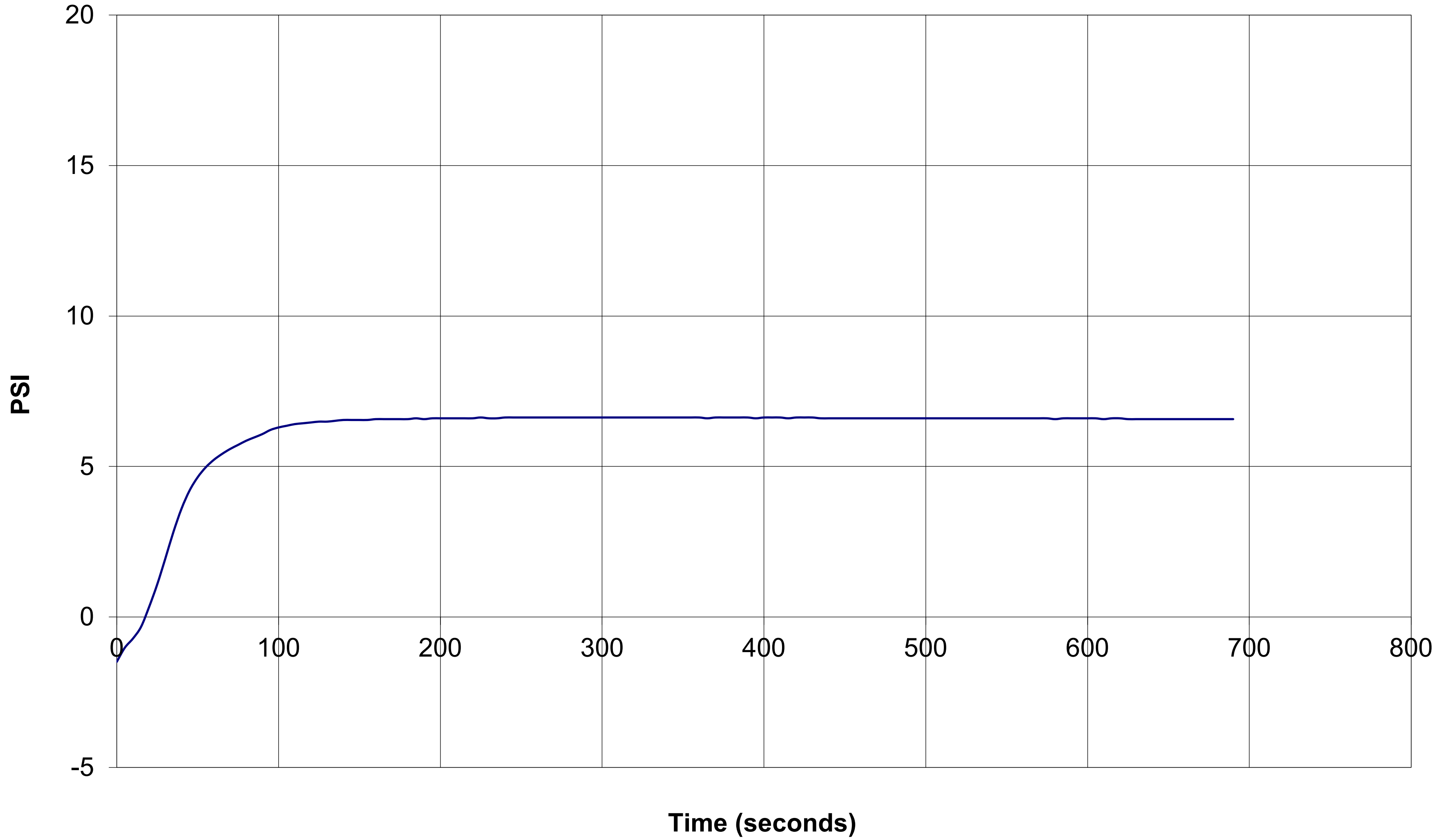


PLATE A-75

Supplement B

Laboratory Testing Program

Peralta Community College District
Project No. 04.72190021

DRILL HOLE	DEPTH, ft	SAMPLE NUMBER	MATERIAL DESCRIPTION	UWW pcf	UDW pcf	MC%	FINES %	ATTERBURG LIMITS: LL	ATTERBURG LIMITS: PI	COMPACTION TEST: MAX DD pcf	COMPACTION TEST: OPT MC %	DIRECT SHEAR: C ksf	DIRECT SHEAR: PHI deg	COMPRESSIVE STRENGTH TESTS: Qu, ksf	COMPRESSIVE STRENGTH TESTS: S_u (Cell Prs.) ksf	CORROSIVITY TESTS: R	CORROSIVITY TESTS: pH	CORROSIVITY TESTS: Cl	CORROSIVITY TESTS: So_4	R-VALUE	EXPANSION INDEX	ORGANIC CONTENT (%)	TEST LISTING
2019-CPT-01	2.5	S1	SILTY SAND (SM)													6400	7.59	N.D.	22				Co
2019-CPT-02	4.5	S2	PEAT (PT)			55																	M
2019-CPT-02	5.5	S3	Fat CLAY (CH)			58																	M
2019-CPT-03	4.0	S1	CLAYEY SAND (SC)			13	20									2600	7.97	N.D.	16				M, FC, Co
2020-B-01	11.0	S5	SILTY SAND with GRAVEL (SM)	112	91	24	21																T, M, S
2020-B-01	16.0	S7	Poorly-graded SAND with SILT (SP-SM)	119	95	26	6															5	T, M, O, S
2020-B-01	17.0	S8	ORGANIC CLAY with SAND (OH)			82																21.2	M, O
2020-B-01	21.0	S9	Fat CLAY (CH)			53																6.6	M, O
2020-B-01	27.0	S10	Fat CLAY (CH)	95	52	83																	T, M, C
2020-B-01	30.0	S11	Fat CLAY (CH)	109	69	58	93	73	43						0.48(2.2)								T, M, A, S, Q
2020-B-01	31.0	S11	Poorly-graded SAND with SILT (SP-SM)	120	94	27	8																T, M, S
2020-B-01	40.5	S13	Fat CLAY (CH)	101	59	71									0.73(2.6)								T, M, Q
2020-B-01	51.0	S15	SILTY SAND (SM)	132	112	18	16																T, M, S
2020-B-01	55.0	S16	SILTY SAND (SM)	135	116	17	17																T, M, S
2020-B-01	66.0	S17	SILTY SAND (SM)			19	19																M, FC
2020-B-01	76.0	S18	Lean CLAY (CL)			37																	M

Classification Tests
UWW = Unit Wet Weight
UDW = Unit Dry Weight
MC = Moisture Content
Fines = % Passing #200 Sieve
LL = Liquid Limit
PI = Plasticity Index

Direct Shear Test
C = Assigned Cohesion, ksf
PHI = Assigned Friction Angle, degrees

Compaction Test
MAX DD = Maximum Dry Density
OPT MC = Optimum Moisture Content

Compressive Strength Tests
Qu = Unconfined Compression
Su = Undrained Shear Strength
u = Unconsolidated Undrained
p = Pocket Penetrometer
t = Torvane
m = Miniature Vane

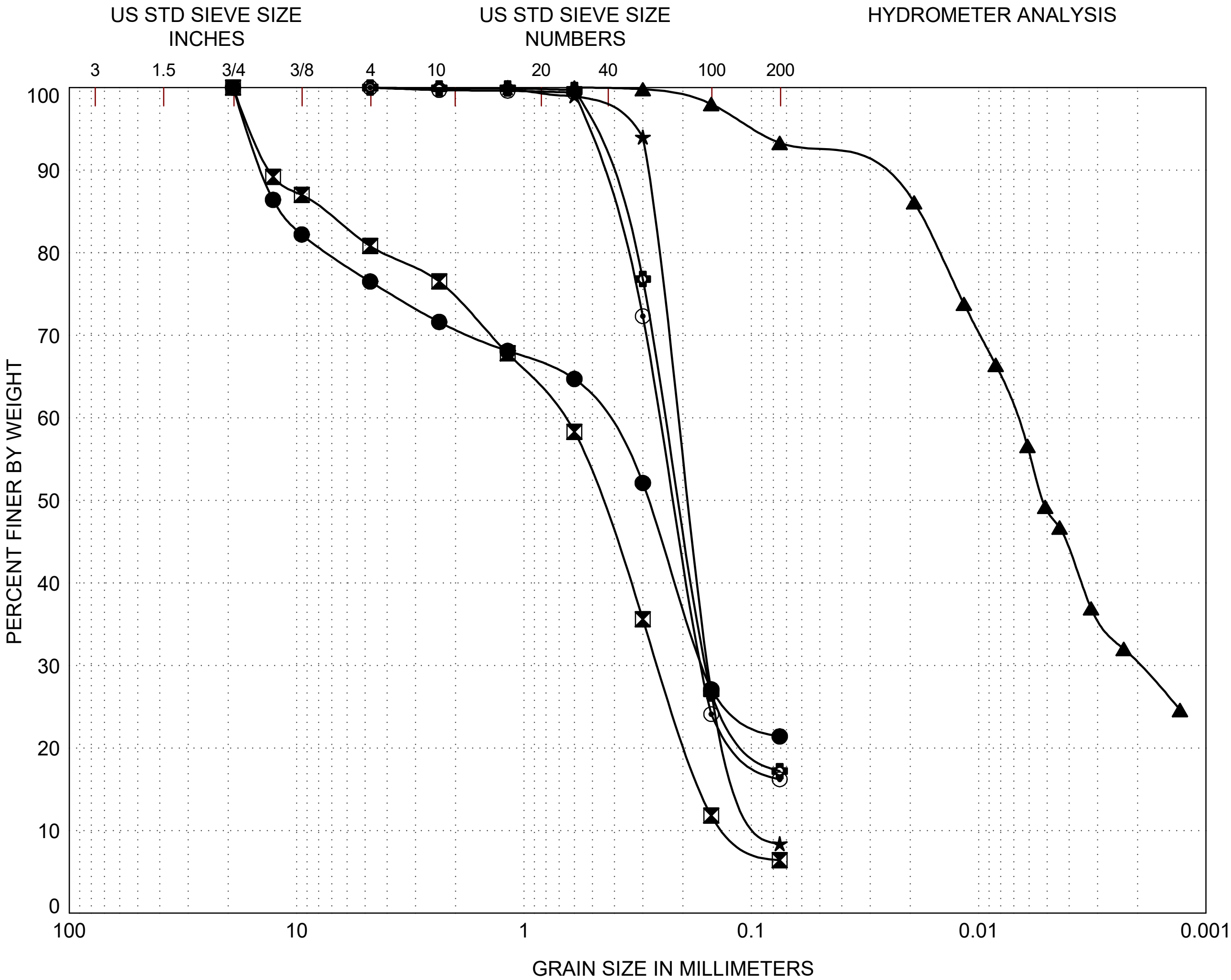
Corrosivity Tests
R = Resistivity, ohm-cm
pH = pH
Cl = Chloride, ppm
SO_4 = Sulfate, ppm

Test Listing Abbreviations
M = Moisture Content
T = Total & Dry Unit Weight
S = Sieve Analysis
FC = % Passing #200 Sieve
H = Hydrometer Analysis
A = Atterberg Limits
P = Compaction Test
D = Direct Shear Test
C = Consolidation Test
Co = Corrosivity Tests
CU = CU Triaxial
U = UU Triaxial
R = R-Value
SE = Sand Equivelant
O = Organic Content

SUMMARY OF LABORATORY TEST RESULTS

Laney College Library Learning Resource Center
Oakland, California

PLATE B-1

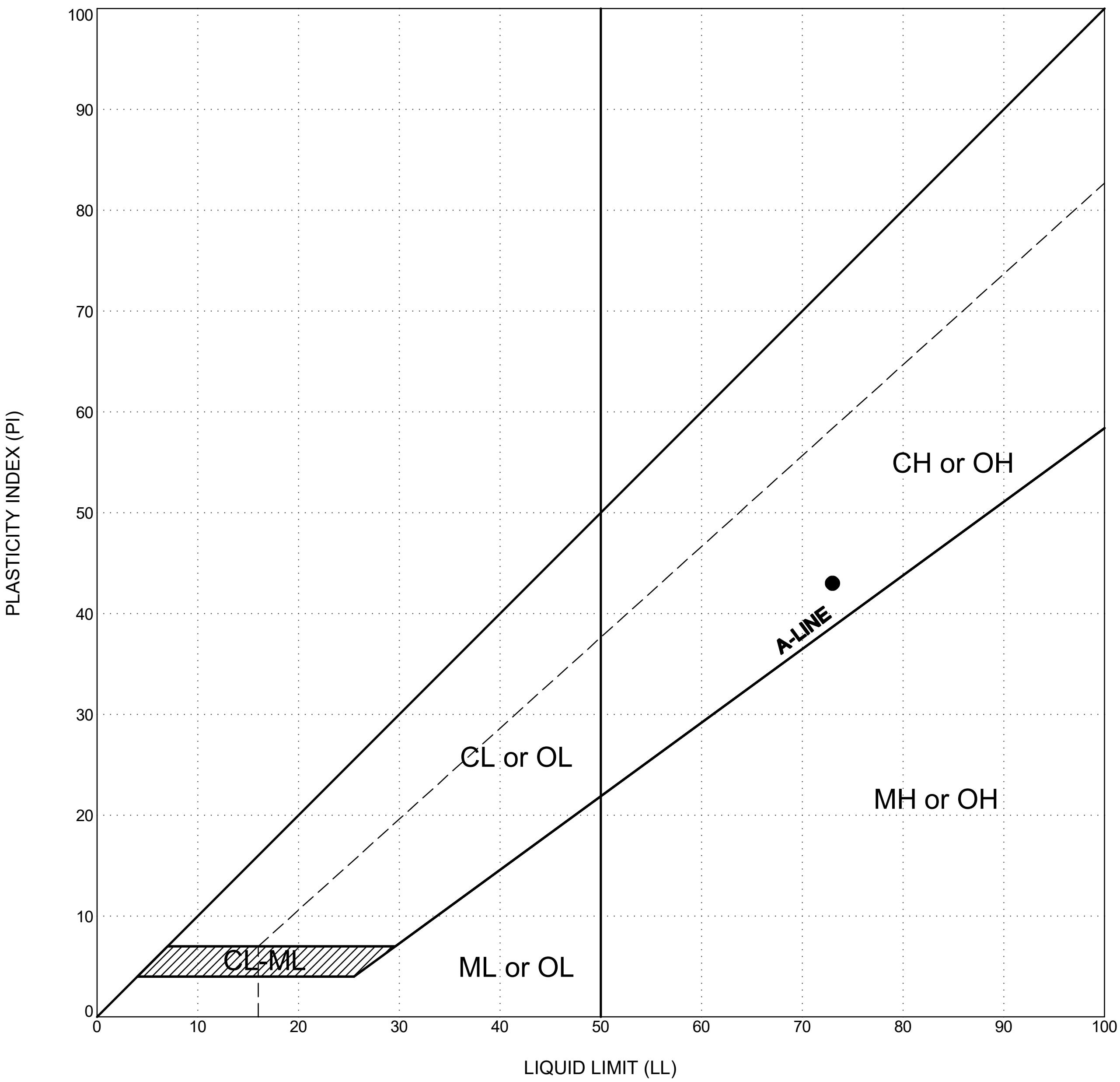


GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

LEGEND (location)	(depth,ft)	CLASSIFICATION	Cc	Cu	D10	D30	D60
● 2020-B-01	11.0	SILTY SAND with GRAVEL (SM)				0.16	0.46
⊠ 2020-B-01	16.0	Poorly-graded SAND with SILT (SP-SM)	0.8	5.7	0.12	0.25	0.68
▲ 2020-B-01	30.0	Fat CLAY (CH)				0.00	0.01
★ 2020-B-01	31.0	Poorly-graded SAND with SILT (SP-SM)	1.4	2.7	0.08	0.16	0.21
⊙ 2020-B-01	51.0	SILTY SAND (SM)				0.16	0.25
✿ 2020-B-01	55.0	SILTY SAND (SM)				0.16	0.24

GRAIN SIZE CURVES
Laney College Library Learning Resource Center
Oakland, California

PLATE B-2



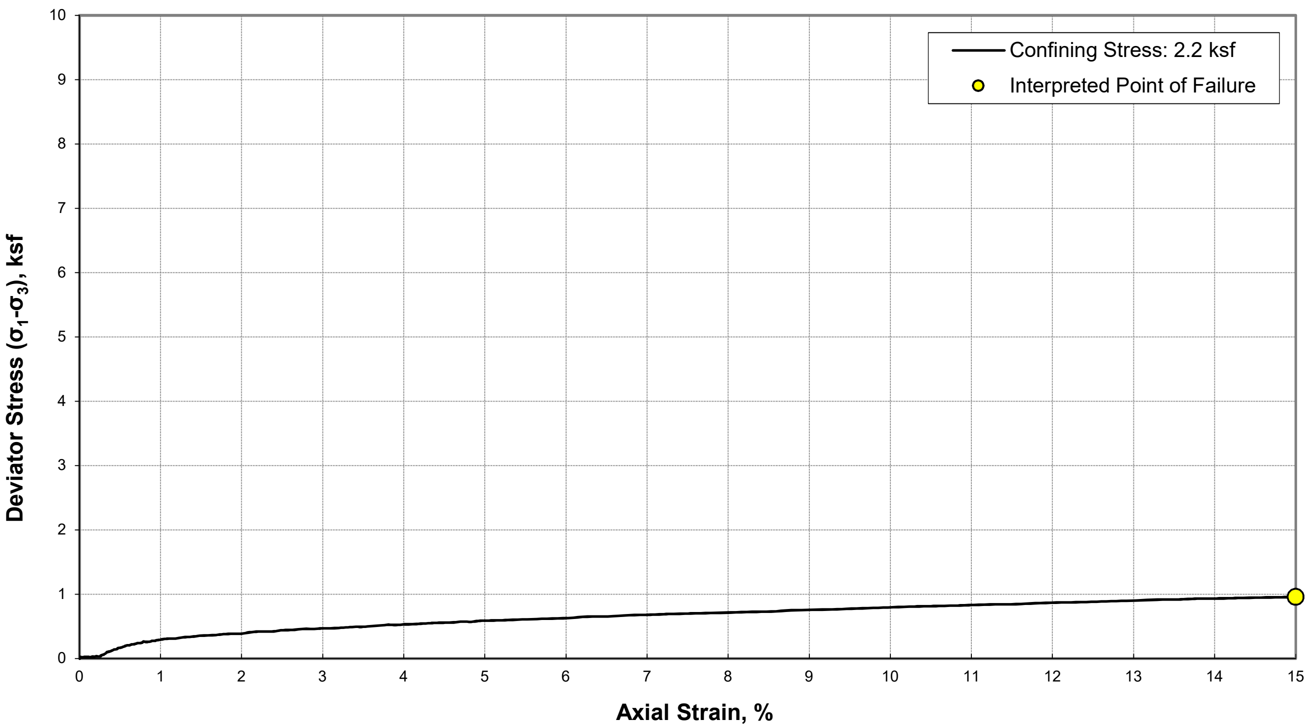
LEGEND			CLASSIFICATION	ATTERBERG LIMITS TEST RESULTS		
	location	depth, ft		LIQUID LIMIT(LL)	PLASTIC LIMIT(PL)	PLASTICITY INDEX (PI)
●	2020-B-01	30.0	Fat CLAY (CH)	73	30	43

PLASTICITY CHART

Laney College Library Learning Resource Center
Oakland, California

PLATE B-3

FUGRO



SAMPLE ID

Boring Number:	B-01
Sample Number:	S11
Sample Depth:	30.0 ft
USCS Classification:	Fat CLAY (CH): olive gray

SAMPLE PROPERTIES

Water Content, %	58.3%
Dry Unit Weight, pcf	68.7
Diameter, in	2.39
Height, in	5.60

SAMPLE IMAGES

CLASSIFICATION

Sieve Size	% Passing	Other Parameters	
3/8-in. (9.5mm)	---	Liquid Limit	---
#4 (4.75mm)	---	Plastic Limit	---
#16 (1.18mm)	---	Plasticity Index	---
#30 (0.6mm)	---	Estimated Gs	2.65
#100 (0.150mm)	---	S_u from T_v, ksf	---
#200 (0.075mm)	---	S_u from PP, ksf	---

TEST SUMMARY

Maximum Deviator Stress, ksf	0.96
Undrained Shear Strength, ksf	0.48
Axial Strain at Failure, %	15.0
Strain Rate, %/min	1.0
Cell Pressure, ksf	2.2
Tested By:	JB
Date Tested:	1/20/20

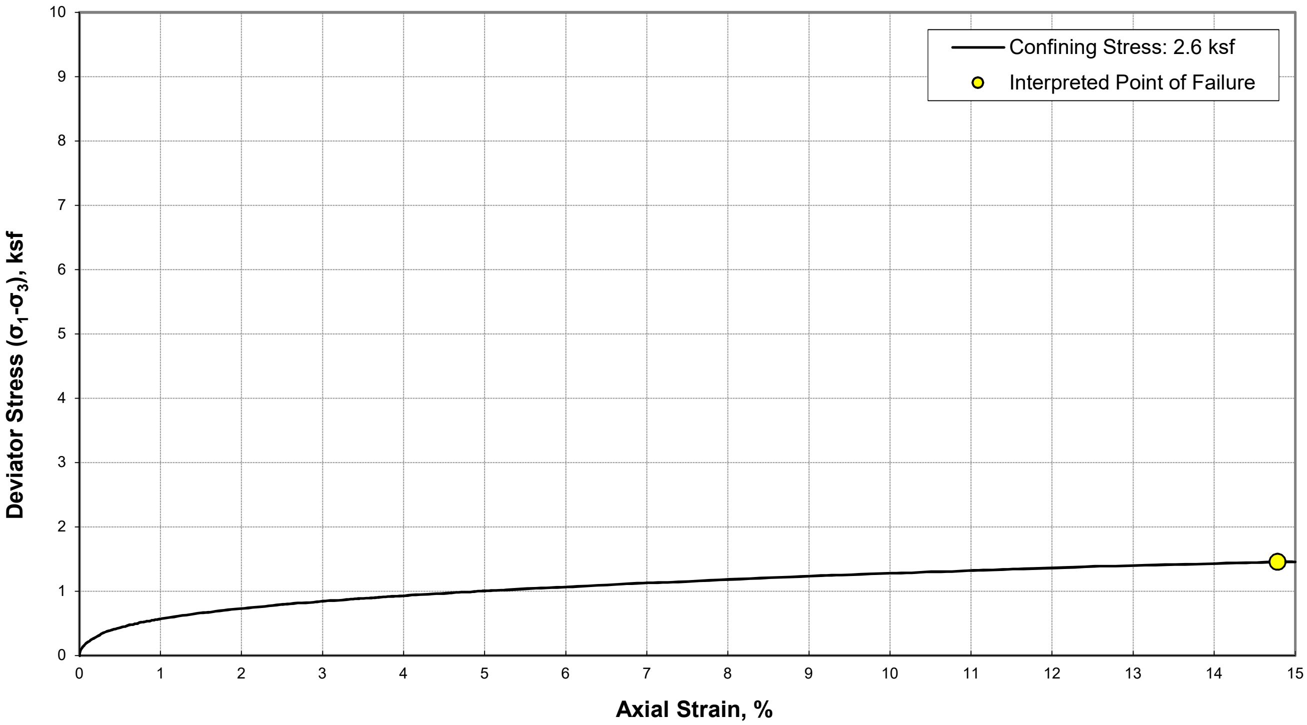
REMARKS

Test Method: ASTM 2850

Note presence of approximate 2" sized gravel in upper portion of UU test sample.

UNCONSOLIDATED, UNDRAINED TRIAXIAL TEST

Laney College Library Learning Resource Center
Oakland, California



SAMPLE ID

Boring Number:	B-01
Sample Number:	S13
Sample Depth:	40.5 ft
USCS Classification:	Fat CLAY (CH): olive gray

SAMPLE PROPERTIES

Water Content, %	71.3%
Dry Unit Weight, pcf	59.1
Diameter, in	2.39
Height, in	5.79

SAMPLE IMAGES


CLASSIFICATION

Sieve Size	% Passing
3/8-in. (9.5mm)	---
#4 (4.75mm)	---
#16 (1.18mm)	---
#30 (0.6mm)	---
#100 (0.150mm)	---
#200 (0.075mm)	---

Other Parameters	
Liquid Limit	---
Plastic Limit	---
Plasticity Index	---
Estimated Gs	2.65
S_u from T_v, ksf	---
S_u from PP, ksf	---

TEST SUMMARY

Maximum Deviator Stress, ksf	1.46
Undrained Shear Strength, ksf	0.73
Axial Strain at Failure, %	14.8
Strain Rate, %/min	1.0
Cell Pressure, ksf	2.6
Tested By:	JB
Date Tested:	1/20/20

REMARKS

Test Method: ASTM 2850

UNCONSOLIDATED, UNDRAINED TRIAXIAL TEST

Laney College Library Learning Resource Center
Oakland, California

SUMMARY OF LABORATORY TEST RESULTS

Project: Laney College Library Learning Resource Center
Address: Oakland, California
Owner: Peralta Community College District

Job Number: 04.72190021
Date: 1/28/2020
Lab ID: 10044

Source:
Location Sampled: B-01, Laney College Library
Date Sampled: N/A
Sample By: N/A
Test Methods: ASTM D2974

Sample No.	Depth (ft)	Sample Description	Water Content (%)	Ash Content (%)	Organic Content (%)
B-01	16	Poorly Graded SAND with SILT (SP - SM)	25.9	95.0	5.0
B-01	17	Organic CLAY with SAND (OH)	82.5	78.8	21.2
B-01	21	Fat CLAY (CH)	53.4	93.4	6.6

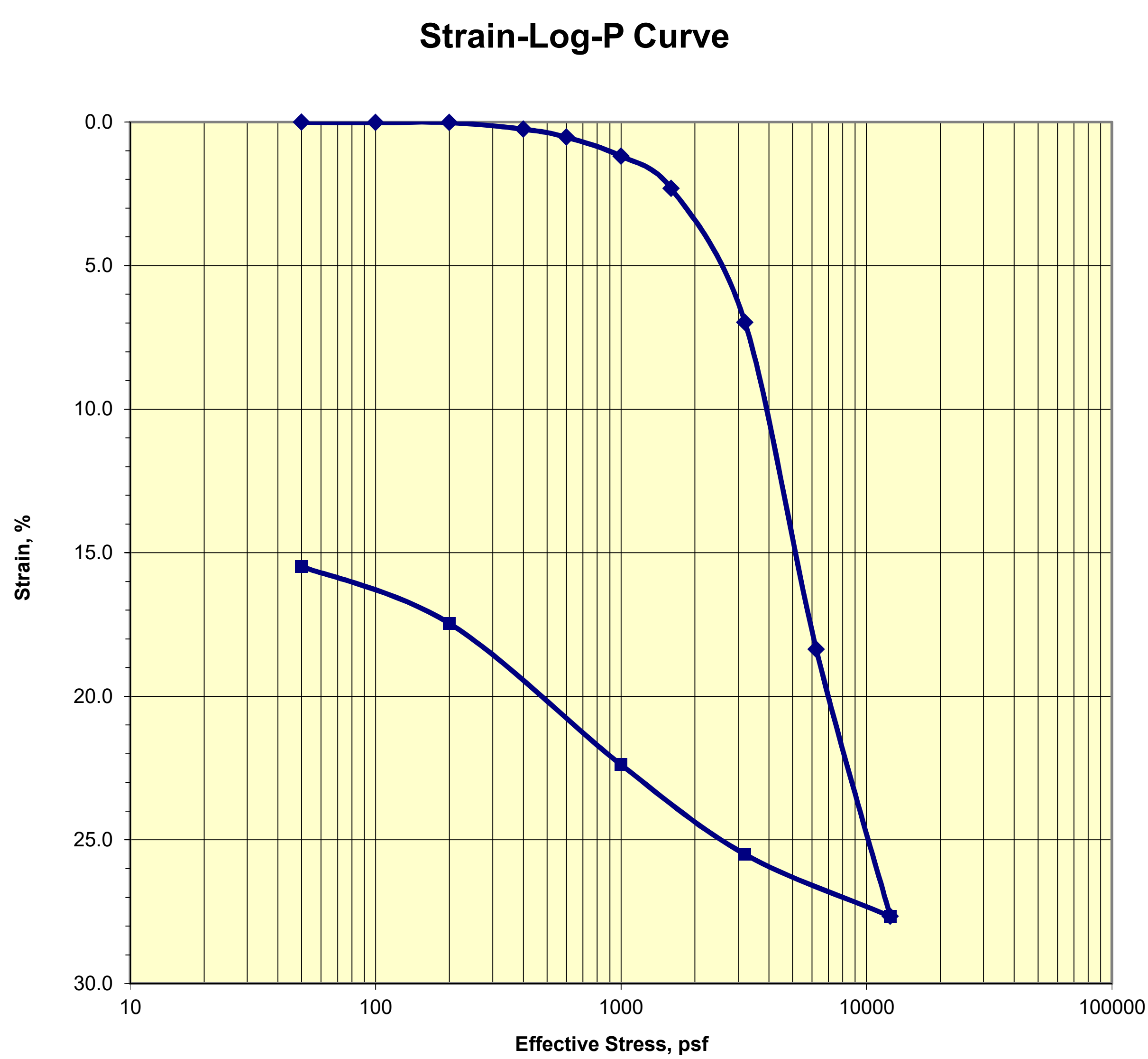
Remarks: None

Distribution:

Consolidation Test

ASTM D2435

Job No.:	446-303	**Boring:**	B-01	**Run By:**	MD
Client:	Fugro USA Land, Inc.	**Sample:**		**Reduced:**	PJ
Project:	04.72190021	**Depth, ft.:**	25-27.5(Tip-3")	**Checked:**	PJ/DC
Soil Type:	Greenish Gray CLAY (Bay Mud)			**Date:**	2/4/2020



Assumed Gs 2.75	Initial	Final
Moisture %:	82.9	65.6
Dry Density, pcf:	51.8	61.2
Void Ratio:	2.316	1.804
% Saturation:	98.4	100.0

Remarks:

California State Certified Laboratory No. 2153

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

10 April 2019

Job No. 1904058
Cust. No. 11608

Mr. Franco A. DePaola
Fugro Consultants, Inc.
1777 Botelho Drive, Suite 262
Walnut Creek, CA 94596

Subject: Project No.: 04.72190021
Project Name: Laney College, 900 Fallon St., Oakland, CA
Corrosivity Analysis – ASTM Test Methods with Brief Evaluation

Dear Mr. DePaola:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on April 05, 2019. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are 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 concentrations reflect none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentrations are 16 & 22 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils are 7.59 & 7.97, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials are 270 & 280-mV. These samples are 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.

Cheryl McMillen 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:	Fugro West, Inc.
Client's Project No.:	04.72190021
Client's Project Name:	Laney College, 900 Fallon St., Oakland, CA
Date Sampled:	29-Mar-19
Date Received:	5-Apr-19
Matrix:	Soil
Authorization:	Signed Chain of Custody

Date of Report: 11-Apr-2019

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)*
1904058-001	CPT-03 @ 4' - 5' (S-3)	270	7.97	-	2,600	-	N.D.	16
1904058-002	CPT-01 @ 2.5' - 3' (S-1)	280	7.59	-	6,400	-	N.D.	22

	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:	9-Apr-2019	9-Apr-2019	-	5-Apr-2019	-	9-Apr-2019	9-Apr-2019

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

PLATE B-9

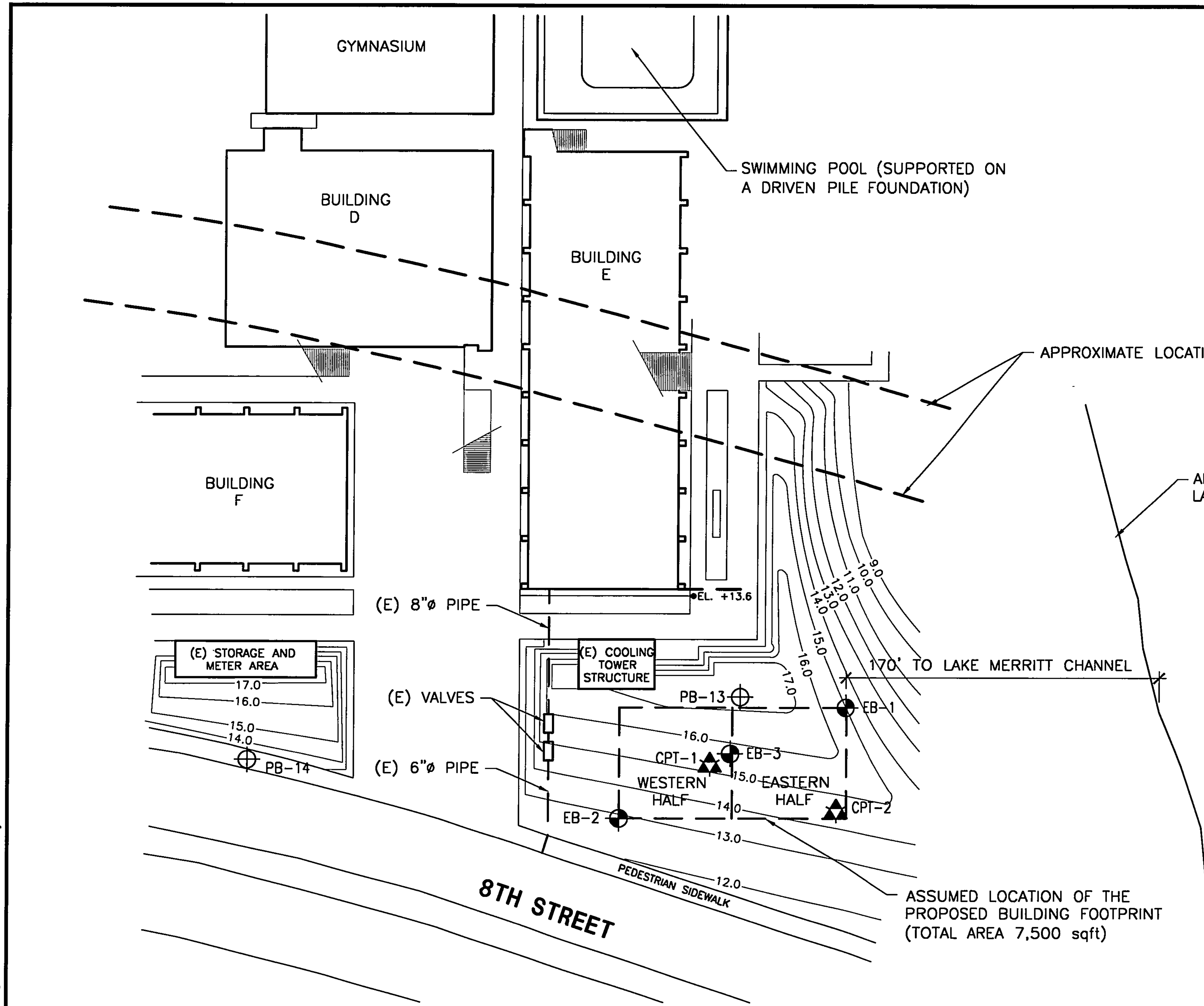
Quality Control Summary - All laboratory quality control parameters were found to be within established limits

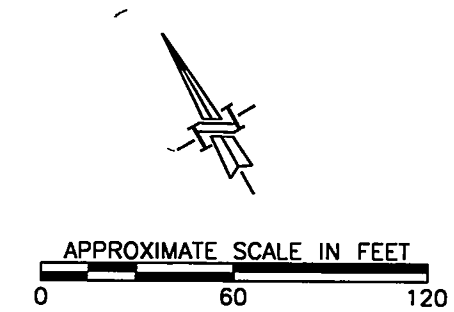
Supplement C

Previous Field Exploration Logs

and Laboratory Test Results

C.1 Exploration Boring Logs and CPTs by Fugro, February 2002, Fugro No. 1430.001





LEGEND

- EB-3 APPROXIMATE LOCATION OF EXPLORATORY BORING (2002)
- CPT-2 APPROXIMATE LOCATION OF CONE PENETRATION TEST (2002)
- PB-14 APPROXIMATE LOCATION OF PREVIOUS BORING BY OTHERS (1968)

NOTE:

1) GROUND CONTOUR LINES WERE BASED ON THE 1968 SITE PLAN. IT MAY HAVE CHANGED OVER TIME.

2) WESTERN HALF OF NEW ART BUILDING SUPPORTED BY 50' LONG PILES.

EASTERN HALF OF NEW ART BUILDING SUPPORTED BY 60' LONG PILES.

FUGRO

FUGRO WEST INC.
425 Roland Way.
Oakland, California. 94621
Tel:(510)568-4001 Fax:(510)568-2205

DRAWN BY:	ROC
PREP'D BY:	NS
APP'D BY:	SR
SCALE:	1" = 60'
DATE:	25FEB02
DWG FILE:	1430.001-01

SITE PLAN

NEW ART BUILDING AT LANEY COLLEGE
OAKLAND, CALIFORNIA

FIGURE
1
PROJECT No.
1430.001

K:\Projects\1430.001\1430.001-01.dwg, 14:38 27MAR02 PLOTTED BY: ROceallaigh

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	14.4 Feet	LOGGED BY	NS
DEPTH TO GROUND WATER	15 feet	BORING DIAMETER	8-inch	DATE DRILLED	2/26/02

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
FILL: CLAY (CL), dark brown, mottled, sandy (fine- to medium-grained), some silt, damp	Firm								
FILL: SAND (SM/SC), brown, mottled, fine- to coarse-grained, silty, some clay, trace gravel and shell fragment, damp	Medium Dense				19				
					13				
			5		21	16	116		PP = 2.5
grades to gray-brown at 6 feet									
					49	11	126		% of Passing #200 Sieve = 24
grades to blue-gray-brown, some silt at 10 feet			10						
BAY MUD: CLAY (CH), black, some sand (fine- to coarse-grained), some silt, mild hydrocarbon odor, trace wood fragment, moist									
	Firm				9				No Recovery
			15			▼			
grades to wet at 16 feet	Soft				6	▽			No Recovery
					21				See Note 7
strong hydrocarbon odor, with high amount of wood fragment, metal pieces, and other debris at 20 feet			20						
grades to blue-gray, silty below 23 feet	Firm				9	77	54		PP = 0.5
			25						
					8	74	56	1.1	PP = 1.0

FUGRO
425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

NEW ART BUILDING AT LANEY COLLEGE
Oakland, CA

PROJECT NO.	DATE	BORING NO.
1430.001	February, 2002	EB-1

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	14.4 Feet	LOGGED BY	NS
DEPTH TO GROUND WATER	15 feet	BORING DIAMETER	8-inch	DATE DRILLED	2/26/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
BAY MUD: CLAY (CH), continued									
SAND (SM), dark green-gray, fine-grained, silty, some clay, trace shell fragment, wet	Loose				10	22	102		PP = 3.0
			35						
BAY MUD: CLAY (CH), blue-gray, silty, trace sand (fine- to medium-grained), wet	Firm				9	76	55	0.4*	PP = 1.5, See Note 8
			40						
SAND (SM/SC), blue-gray, fine- to medium-grained, silty, with clay, trace shell fragment, wet	Medium Dense				32	16	112		
			45						
	Very Dense				83/9"				
			50						
	Dense				37				
			55						
	Very Dense				63				

FUGRO

425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

NEW ART BUILDING AT LANEY COLLEGE
Oakland, CA

PROJECT NO.	DATE	BORING NO.
1430.001	February, 2002	EB-1

File Name. G:\ENGINEER\GINTW\PROJECTS\21127GI.GPJ Report Template: FUGRO Output Date: 3/26/02

DRILL RIG	**Mobile B-61, HSA**	SURFACE ELEVATION	**14.4 Feet**	LOGGED BY	**NS**
DEPTH TO GROUND WATER	**15 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**2/26/02**

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
SAND (SM/SC), continued grades to blue-gray-brown, trace gravel at 60 feet									
grades to brown, clayey below 63 feet	Dense		65	X	61	22	105	2.3	% of Passing #200 Sieve = 43 PP = 4.0
			70	X	67				PP = 4.5
			75	X	73				PP = 2.5

Bottom of Boring = 75 Feet

Notes:

1. The stratification lines represent the approximate boundaries between material types and the transition may be gradual.
2. For an explanation of penetration resistance values, see first page of Appendix A.
3. A 140-lb safety hammer falling 30 inches was used to drive the sampler.
4. Ground water was encountered originally at depth of about 17 feet, and at depth of about 15 feet two hours later.
5. The borehole was backfilled with lean cement immediately upon completion of the drilling.
6. PP = Pocket Penetrometer Reading (tsf).
7. High value of blow count is due to localized encountering metal, brick, and/or concrete debris.
8. Low shear strength was probably caused by severe sample disturbance.

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EXPLORATORY BORING LOG

NEW ART BUILDING AT LANEY COLLEGE
Oakland, CA

PROJECT NO.	DATE	BORING NO.
1430.001	**February, 2002**	**EB-1**

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	12.8 Feet	LOGGED BY	NS
DEPTH TO GROUND WATER	45 feet	BORING DIAMETER	8-inch	DATE DRILLED	2/26/02

DESCRIPTION AND CLASSIFICATION			DEPTH	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						
FILL: CLAY (CL), dark brown, mottled, sandy (fine- to medium-grained), some silt, damp	Firm								
FILL: SAND (SM), brown, fine- to coarse-grained, silty, trace clay and gravel, damp	Medium Dense				15	13	110	1.3	PP = 2.0
					23				
	Loose		5		10				
grades to black, gravelly (subangular to subrounded) at 6 feet									
BAY MUD: CLAY (CH), blue-gray, silty, trace sand (fine- to coarse-grained) and wood fragmentl, moist	Soft				3				PP = 0.5
			10						
	Very Soft				2	50	74	0.2	PP < 0.5
grades to mottled shades of black-brown, trace shell fragment at 15 feet			15						
grades to dark gray-brown, mild hydrocarbon odor at 18 feet	Soft				4	78	54	0.3	PP = 0.5
			20						
					4				PP = 1.5

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EXPLORATORY BORING LOG

NEW ART BUILDING AT LANEY COLLEGE
Oakland, CA

PROJECT NO.	DATE	BORING NO.
1430.001	February, 2002	EB-2

DRILL RIG	**Mobile B-61, HSA**	SURFACE ELEVATION	**12.8 Feet**	LOGGED BY	**NS**
DEPTH TO GROUND WATER	**45 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**2/26/02**

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
BAY MUD: CLAY (CH), continued	Soft								
grades to gravelly (rounded to subrounded),wet at 28 feet									
CLAY (CL/GC), blue-gray, gravelly, some silt and sand, wet	Hard				57	17	114	9.1	
			30						
SAND (SP/SM), light brown, medium- to coarse-grained, trace gravel (subangular to subrounded) and silt, wet									
	Dense				37				
			35						
					32				% of Passing #200 Sieve = 19 between 29 feet to 59 feet
			40						
					32	▼			
			45						
					37				

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EXPLORATORY BORING LOG

NEW ART BUILDING AT LANEY COLLEGE
Oakland, CA

PROJECT NO.	DATE	BORING NO.
1430.001	**February, 2002**	**EB-2**

DRILL RIG	**Mobile B-61, HSA**	SURFACE ELEVATION	**12.8 Feet**	LOGGED BY	**NS**
DEPTH TO GROUND WATER	**45 feet**	BORING DIAMETER	**8-inch**	DATE DRILLED	**2/26/02**

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
SAND (SP/SM), continued	Dense								
			55						
CLAY (CL), olive-brown, silty, with sand (fine- to medium-grained), wet	Hard				67	21	109	12.3	PP = 4.5
			60						
			65						
grades to dark gray at 69 feet					76				
			70						

Bottom of Boring = 70 Feet

Notes:

1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values, see first page of Appendix A.
3. A 140-lb safety hammer falling 30 inches was used to drive the sampler.
4. Ground water was apparently encountered at depth of 45 feet at the time of drilling.
5. The borehole was backfilled with lean cement immediately upon completion of the drilling.
6. PP = Pocket Penetrometer Reading (tsf).

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EXPLORATORY BORING LOG

NEW ART BUILDING AT LANEY COLLEGE
Oakland, CA

PROJECT NO.	DATE	BORING NO.
1430.001	**February, 2002**	**EB-2**

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	14.3 Feet	LOGGED BY	NS
DEPTH TO GROUND WATER	20 feet	BORING DIAMETER	8-inch	DATE DRILLED	2/26/02

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
FILL: CLAY (CL), dark brown, mottled, sandy (fine- to medium-grained), some silt, damp	Firm								
FILL: SAND (SM), dary gray-brown, mottled shades of green, fine- to coarse-grained, silty, some clay, trace gravel (subangular to subrounded), trace brick pieces, damp	Medium Dense		5		33	15	119	3.2	% of Passing #200 Sieve = 42
hard drilling due to encountering concrete or brick chunk			10		50/4"				No Recovery
BAY MUD: CLAY (CH), black, mottled shades of blue-gray, silty, mild hydrocarbon odor, moist	Very Soft		15						
			20		1	▽			PP < 0.5
			25						

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EXPLORATORY BORING LOG

NEW ART BUILDING AT LANEY COLLEGE
Oakland, CA

PROJECT NO.	DATE	BORING NO.
1430.001	February, 2002	EB-3

DRILL RIG	Mobile B-61, HSA	SURFACE ELEVATION	14.3 Feet	LOGGED BY	NS
DEPTH TO GROUND WATER	20 feet	BORING DIAMETER	8-inch	DATE DRILLED	2/26/02

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT)	WATER CONTENT(%)	DRY DENSITY (PCF)	UNCONFINED COMPRESSIVE STRENGTH (KSF)	OTHER TESTS
BAY MUD: CLAY (CH), continued	Very Soft								
SAND (SM - SC), dark gray, medium- to coarse-grained, with silt, strong hydrocarbon odor, trace shell fragment, wet	Loose to medium dense				18				No Recovery
			30		8				
			35						
CLAY (CL), blue-gray, silty, with sand (fine- to coarse-grained), trace gravel, wet	Very Stiff				44	18	112	2.6	
			40						
SAND (SM - SC), dark brown, mottled shades of green, fine- to coarse-grained, clayey, some silt, trace gravel, wet	Dense								
			45						
					52				
			50						

Bottom of Boring = 50 Feet

Notes:

1. The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.
2. For an explanation of penetration resistance values, see first page of Appendix A.
3. A 140-lb safety hammer falling 30 inches was used to drive the sampler.
4. Ground water was encountered originally at depth of about 20 feet at the time of drilling.
5. The borehole was backfilled with lean cement immediately upon completion of the drilling.
6. PP = Pocket Penetrometer Reading (tsf).

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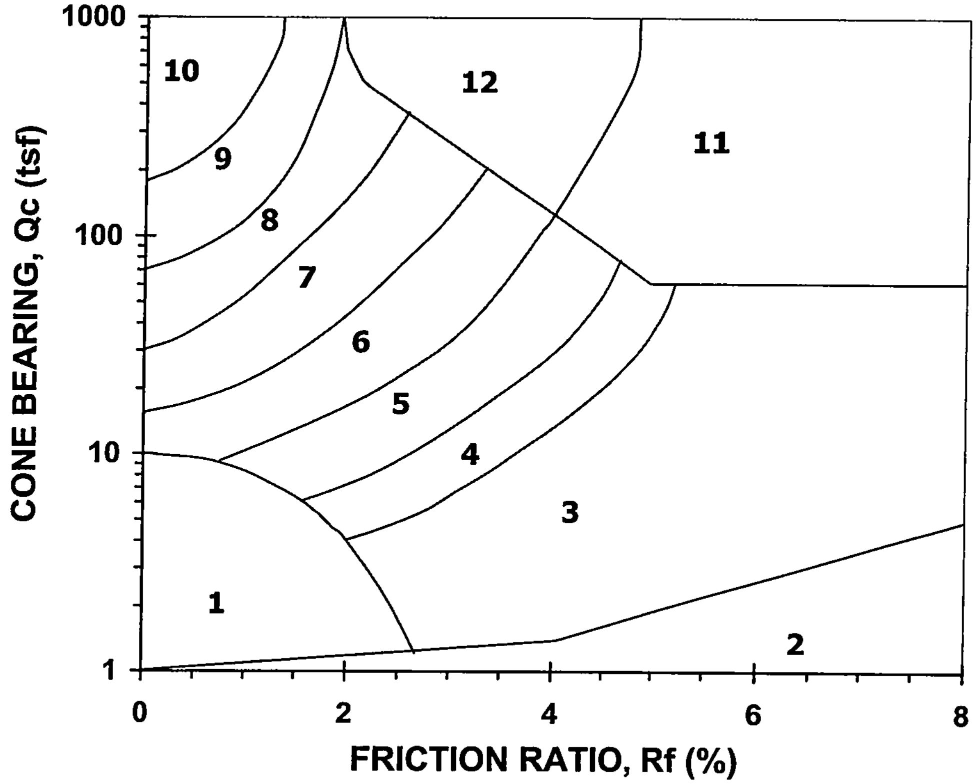
EXPLORATORY BORING LOG

NEW ART BUILDING AT LANEY COLLEGE
Oakland, CA

PROJECT NO.	DATE	BORING NO.
1430.001	February, 2002	EB-3

SIMPLIFIED SOIL BEHAVIOR TYPE CLASSIFICATION FOR STANDARD ELECTRONIC CONE PENETROMETER



ZONE	Qc/N[1]	Su Factor (Nk)[2]	SOIL BEHAVIOR TYPE[1]
1	2		Sensitive Fine Grained
2	1	for Zones 1 to 6	Organic Material
3	1	10 for Qc <= 9 tsf	CLAY
4	1.5	12 for Qc = 9 to 12 tsf	Silty CLAY to CLAY
5	2	15 for Qc > 12 tsf	Clayey SILT to Silty CLAY
6	2.5		Sandy SILT to Clayey SILT
7	3	---	Silty SAND to Sandy SILT
8	4	---	SAND to Silty SAND
9	5	---	SAND
10	6	---	Gravelly SAND to SAND
11	1	15	Very Stiff Fine Grained (*)
12	2	---	SAND to Clayey SAND (*)

(*) Overconsolidated or Cemented

Qc = Tip Bearing
Fs = Sleeve Friction
Rf = Fs/Qc*100 = Friction Ratio

References: [1]Robertson, 1986, Olsen, 1988
[2]Bonaparte & Mitchell, 1979 (young bay mud Qc <= 9)
[2]Estimated from local experience (fine grained soils Qc > 9)

Note: Testing performed in accordance with ASTM D3441

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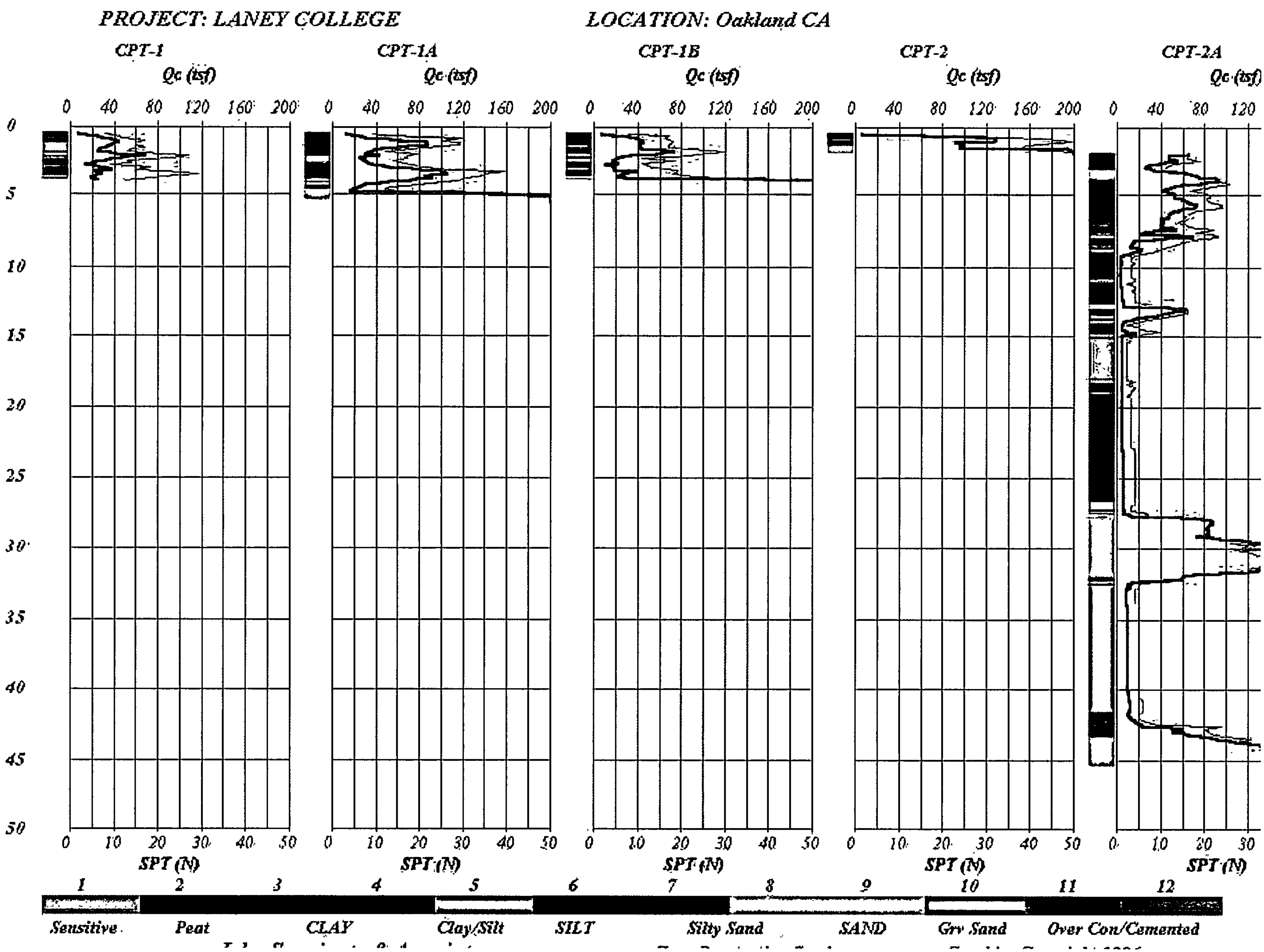
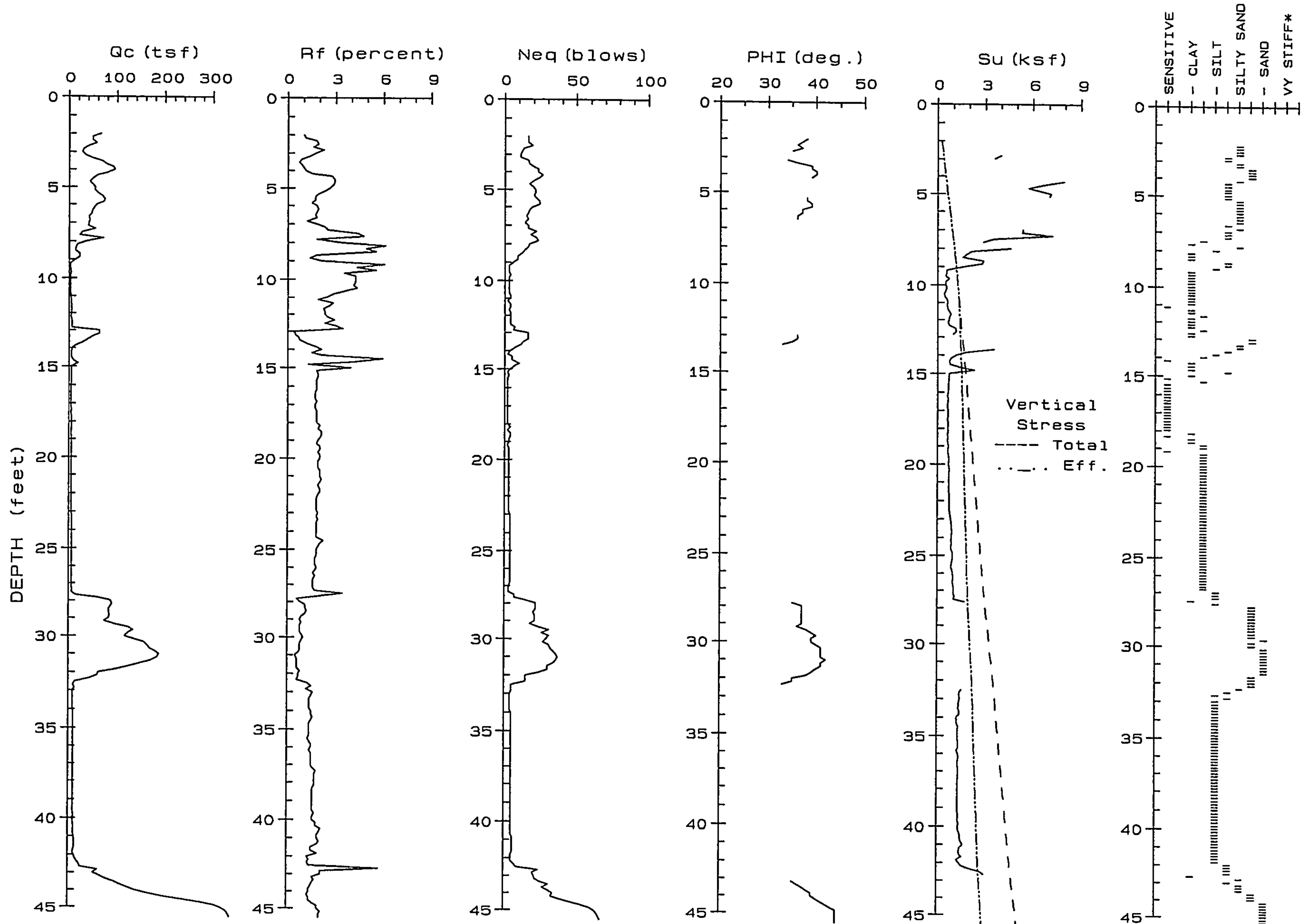


RESULTS OF CONE PENETROMETER TESTS (CPTs)



Terminated at 45.5 feet

Groundwater measured at 11.0 feet

PROJECT: LANEY COLLEGE
LOCATION: Oakland CA
PROJ. NO.: 21127-G1 (MWH-37)

CPT NO.: CPT-2A
DATE : 02-26-2002

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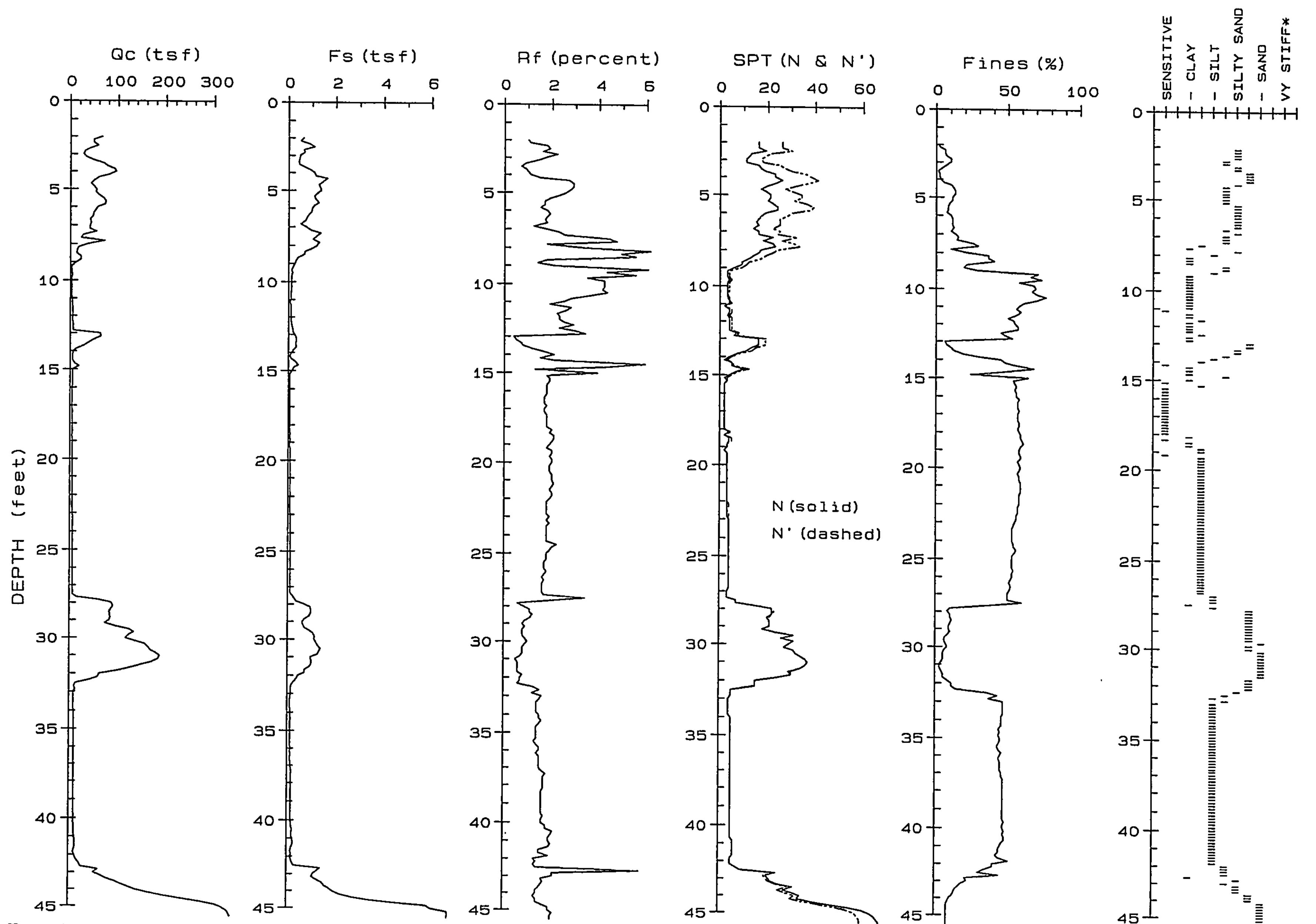

Terminated at 45.5 feet

Groundwater measured at 11.0 feet

PROJECT: LANEY COLLEGE
LOCATION: Oakland CA
PROJ. NO.: 21127-G1 (MWH-37)

CPT NO.: CPT-2A
DATE : 02-26-2002

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PROJECT: LANEY COLLEGE
LOCATION: Oakland CA
PROJ. NO.: 21127-G1(MWH-37)

CPT NO.: CPT-2A
DATE : 02-26-2002
Groundwater measured at 11.0 feet

DEPTH (feet)	Qc (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT (N')	TotHzStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
2.00	65.31	0.633	1.0	16	26	0.25	38	----	SAND to Silty SAND	''
2.50	55.84	1.069	1.9	19	30	0.31	37	----	Silty SAND to Sandy SILT	130-140
3.00	26.84	0.490	1.8	11	17	0.37	----	3.55	Sandy SILT to Clayey SILT	120-130
3.50	68.70	0.451	0.7	17	27	0.43	39	----	SAND to Silty SAND	''
4.00	93.79	1.068	1.1	23	38	0.50	40	----	''	''
4.50	49.78	1.437	2.9	20	32	0.56	----	6.60	Sandy SILT to Clayey SILT	130-140
5.00	53.17	1.359	2.6	21	34	0.63	----	7.05	''	''
5.50	67.05	1.135	1.7	22	36	0.70	38	----	Silty SAND to Sandy SILT	''
6.00	56.26	0.986	1.8	19	30	0.77	37	----	''	''
6.50	46.38	0.771	1.7	15	25	0.83	36	----	''	''
7.00	40.32	0.739	1.8	16	25	0.90	----	5.32	Sandy SILT to Clayey SILT	''
7.50	26.43	1.166	4.4	18	26	0.97	----	3.46	Silty CLAY to CLAY	''
8.00	34.45	1.180	3.4	17	24	1.03	----	4.52	Clayey SILT to Silty CLAY	''
8.50	12.15	0.668	5.5	12	17	1.10	----	1.55	CLAY	120-130
9.00	12.82	0.270	2.1	6	9	1.16	----	1.63	Clayey SILT to Silty CLAY	110-120
9.50	3.06	0.167	5.5	3	4	1.20	----	0.49	CLAY	90-100
10.00	3.14	0.131	4.2	3	4	1.25	----	0.50	''	''
10.50	2.67	0.114	4.3	3	3	1.30	----	0.40	''	''
11.00	3.82	0.096	2.5	4	5	1.35	----	0.63	''	''
11.50	4.21	0.110	2.6	4	5	1.39	----	0.70	''	''
12.00	4.25	0.097	2.3	4	5	1.44	----	0.71	''	''
12.50	6.40	0.149	2.3	4	5	1.49	----	1.13	Silty CLAY to CLAY	''
13.00	62.43	0.258	0.4	16	19	1.54	36	----	SAND to Silty SAND	120-130
13.50	38.26	0.318	0.8	13	15	1.60	33	----	Silty SAND to Sandy SILT	110-120
14.00	6.53	0.139	2.1	4	5	1.66	----	1.14	Silty CLAY to CLAY	90-100
14.50	4.72	0.276	5.9	5	6	1.70	----	0.77	CLAY	100-110
15.00	4.41	0.172	3.9	4	5	1.76	----	0.71	''	90-100
15.50	4.27	0.080	1.9	2	2	1.81	----	0.67	Sensitive Fine Grained	''
16.00	4.19	0.075	1.8	2	2	1.85	----	0.65	''	''
16.50	4.13	0.072	1.7	2	2	1.90	----	0.64	''	85-90
17.00	4.17	0.072	1.7	2	2	1.94	----	0.64	''	''
17.50	4.17	0.075	1.8	2	2	1.99	----	0.63	''	90-100
18.00	4.24	0.076	1.8	2	2	2.04	----	0.64	''	''
18.50	4.23	0.090	2.1	4	5	2.08	----	0.64	CLAY	''
19.00	4.39	0.085	1.9	3	3	2.13	----	0.67	Silty CLAY to CLAY	''
19.50	4.45	0.089	2.0	3	3	2.18	----	0.67	''	''
20.00	4.47	0.089	2.0	3	3	2.23	----	0.67	''	''
20.50	4.70	0.091	1.9	3	3	2.27	----	0.71	''	''
21.00	4.68	0.093	2.0	3	3	2.32	----	0.70	''	''
21.50	4.74	0.092	1.9	3	3	2.37	----	0.71	''	''
22.00	4.80	0.093	1.9	3	3	2.42	----	0.72	''	''
22.50	4.86	0.089	1.8	3	3	2.46	----	0.73	''	''
23.00	5.38	0.100	1.9	4	4	2.51	----	0.82	''	''
23.50	5.77	0.106	1.8	4	4	2.56	----	0.90	''	''
24.00	5.81	0.104	1.8	4	4	2.61	----	0.90	''	''
24.50	5.73	0.125	2.2	4	4	2.65	----	0.88	''	''
25.00	5.95	0.108	1.8	4	4	2.70	----	0.92	''	''
25.50	5.87	0.102	1.7	4	4	2.75	----	0.90	''	''
26.00	6.01	0.109	1.8	4	4	2.80	----	0.92	''	''
26.50	6.11	0.102	1.7	4	4	2.84	----	0.94	''	''
27.00	6.52	0.105	1.6	3	3	2.89	----	1.02	Clayey SILT to Silty CLAY	''
27.50	6.50	0.224	3.4	7	7	2.94	----	1.01	CLAY	100-110
28.00	86.18	0.708	0.8	22	22	3.00	37	----	SAND to Silty SAND	120-130
28.50	81.76	0.985	1.2	20	21	3.06	37	----	''	''
29.00	82.38	0.631	0.8	21	21	3.13	37	----	''	''
29.50	122.59	0.928	0.8	31	31	3.19	39	----	''	''
30.00	116.31	1.119	1.0	29	29	3.25	39	----	''	''
30.50	161.98	1.367	0.8	32	32	3.31	41	----	SAND	''
31.00	185.64	0.963	0.5	37	37	3.38	42	----	''	''
31.50	147.89	0.941	0.6	30	29	3.44	40	----	''	''
32.00	61.40	0.446	0.7	15	15	3.50	35	----	SAND to Silty SAND	''
32.50	13.03	0.172	1.3	5	5	3.56	----	1.50	Sandy SILT to Clayey SILT	100-110
33.00	8.97	0.147	1.6	4	4	3.61	----	1.43	Clayey SILT to Silty CLAY	''
33.50	8.49	0.122	1.4	4	4	3.66	----	1.33	''	90-100
34.00	9.09	0.134	1.5	5	5	3.71	----	1.21	''	''

John Sarmiento & Associates
Cone Penetration Testing Service

PROJECT: LANEY COLLEGE
LOCATION: Oakland CA
PROJ. NO.: 21127-G1(MWH-37)

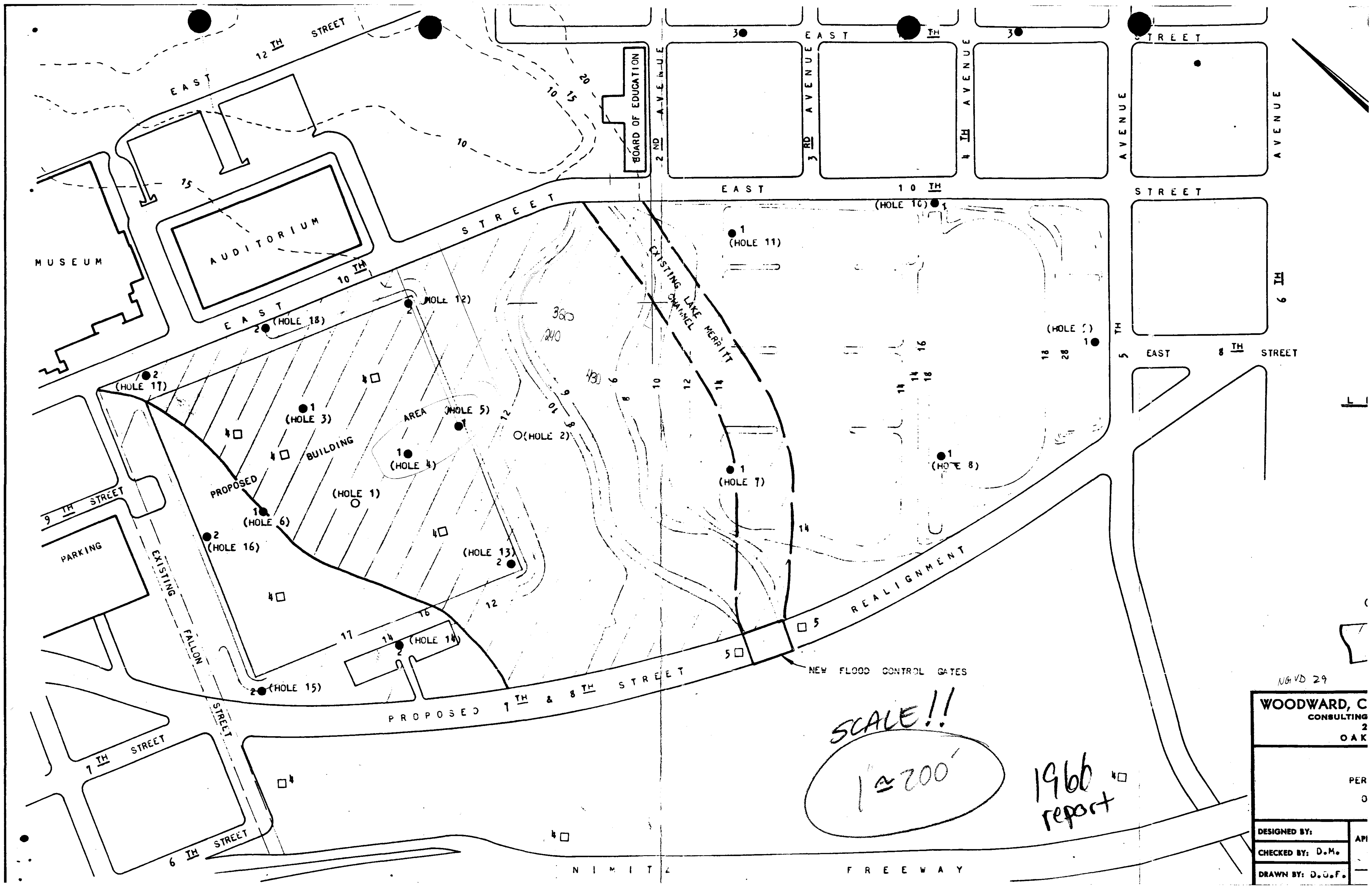
CPT NO.: CPT-2A
DATE : 02-26-2002
Groundwater measured at 11.0 feet

DEPTH (feet)	Qc (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT (N')	TotHzStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
34.50	9.78	0.143	1.5	5	5	3.75	----	1.32	..	..
35.00	9.60	0.138	1.4	5	5	3.80	----	1.28	..	..
35.50	9.93	0.126	1.3	5	5	3.85	----	1.33	..	..
36.00	9.83	0.147	1.5	5	5	3.90	----	1.31	..	100-110
36.50	9.82	0.144	1.5	5	5	3.95	----	1.31	..	90-100
37.00	9.93	0.153	1.5	5	5	4.00	----	1.32	..	100-110
37.50	9.82	0.168	1.7	5	5	4.05	----	1.30	..	..
38.00	9.79	0.163	1.7	5	5	4.10	----	1.29	..	..
38.50	9.87	0.161	1.6	5	5	4.16	----	1.30	..	..
39.00	9.87	0.161	1.6	5	5	4.21	----	1.29	..	..
39.50	10.14	0.167	1.6	5	5	4.26	----	1.33	..	..
40.00	10.14	0.165	1.6	5	5	4.31	----	1.33	..	..
40.50	10.91	0.227	2.1	5	5	4.37	----	1.46	..	..
41.00	11.94	0.232	1.9	6	6	4.42	----	1.62	..	..
41.50	12.51	0.191	1.5	6	6	4.48	----	1.37	..	..
42.00	12.66	0.161	1.3	5	5	4.53	----	1.39	Sandy SILT to Clayey SILT	..
42.50	22.52	0.304	1.4	9	9	4.58	----	2.70	..	110-120
43.00	52.04	1.108	2.1	21	20	4.65	----	6.63	..	130-140
43.50	92.46	1.483	1.6	31	29	4.72	37	----	Silty SAND to Sandy SILT	..
44.00	137.71	1.911	1.4	34	32	4.78	39	----	SAND to Silty SAND	..
44.50	232.01	3.349	1.4	46	42	4.85	42	----	SAND	..
45.00	314.60	5.980	1.9	63	57	4.92	44	----	..	..
45.50	332.90	6.687	2.0	67	59	4.99	44	----	..	..

DEPTH = Sampling interval (2 inches)
Qc = Tip bearing resistance
Fs = Sleeve friction resistance
Rf = Tip/Sleeve ratio
SPT = Equivalent Standard Penetration Test*
TotStr = Total Stress using est. density**
Phi = Soil friction angle*
Su = Undrained Soil Strength* (Nk=10 for Qc<9 tsf) (Nk=12 for Qc=9 to 12 tsf) (Nk=15 for Qc>12 tsf)

References: * Robertson and Campanella, 1988
** Olsen, 1989 *** Durgunoglu & Mitchell, 1975

C.2 Exploratory Boring Logs and Lab Results by WCS, November 1965, WCS No. S10312



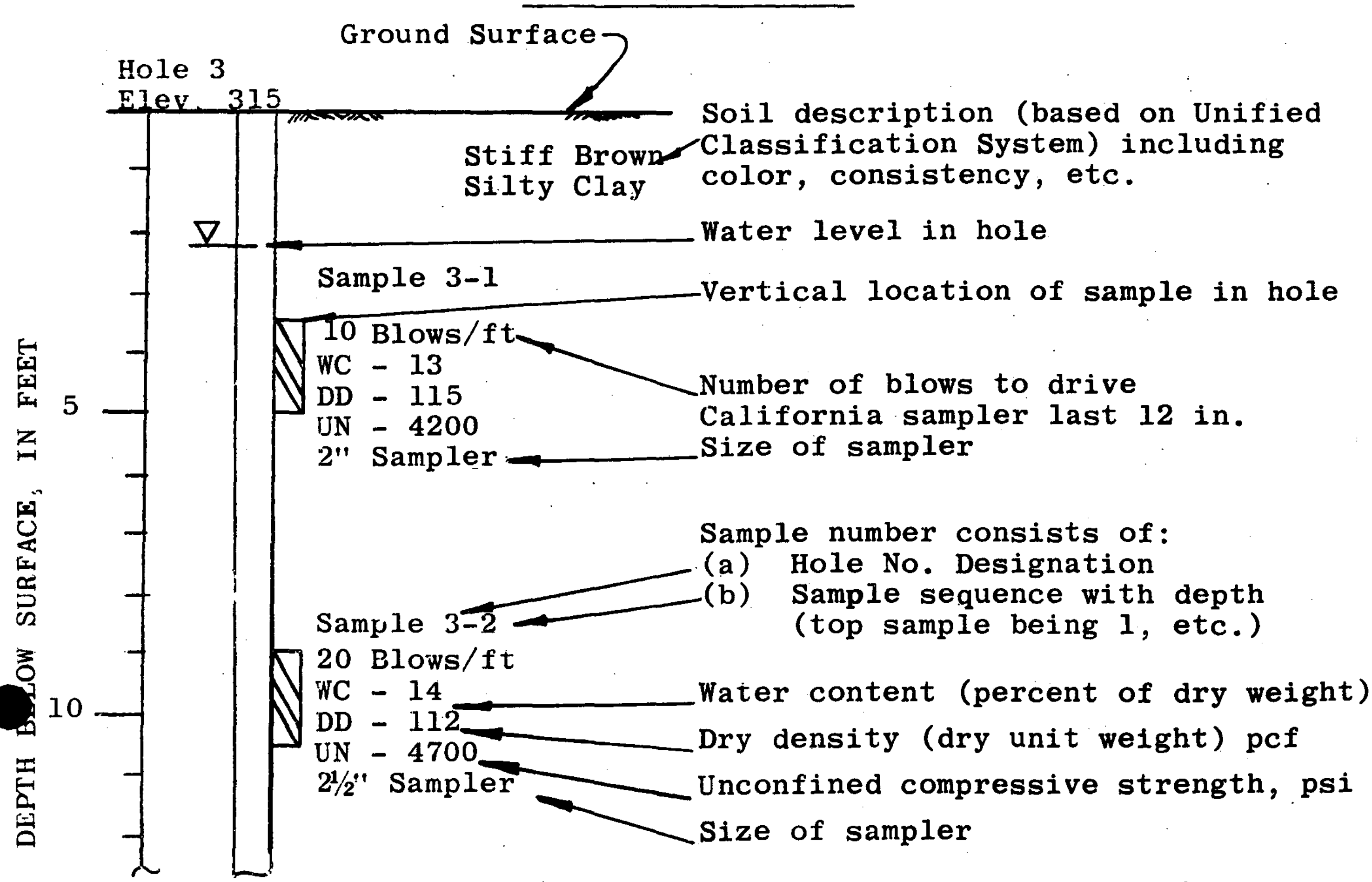
EAST 12 TH STREET
BOARD OF EDUCATION
EAST
TH
STREET
2 ND AVENUE
3 RD AVENUE
4 TH AVENUE
AVENUE
AVENUE
MUSEUM
AUDITORIUM
EAST 10 TH STREET
EAST 10 TH STREET
(HOLE 10)
(HOLE 11)
(HOLE 12)
(HOLE 18)
(HOLE 17)
EXISTING LAKE MERRITT CHANNEL
(HOLE 3)
PROPOSED BUILDING AREA
(HOLE 5)
(HOLE 2)
(HOLE 4)
(HOLE 1)
(HOLE 6)
(HOLE 7)
(HOLE 8)
(HOLE 16)
(HOLE 13)
(HOLE 14)
(HOLE 15)
5 TH EAST 8 TH STREET
6 TH
9 TH STREET
PARKING
EXISTING FALLON STREET
REALIGNMENT
NEW FLOOD CONTROL GATES
PROPOSED 7 TH & 8 TH STREET
7 TH STREET
6 TH STREET
NIMITZ FREEWAY
NGVD 29
SCALE!!
1" ≈ 200'
1966 report
WOODWARD, C
CONSULTING
OAK
PER
DESIGNED BY:
CHECKED BY: D.M.
DRAWN BY: D.G.F.

APPENDIX

NOTES ON FIELD INVESTIGATION

1. Borings were advanced with a 6-in. diameter continuous flight power auger and by wash boring.
2. The Engineering Geologist were M. Conant, R. Russell and C. Taylor
3. In-place samples of the soils were obtained with either drive samplers or Shelby tube samplers. The size of sampler used is indicated at the sample location on the logs of borings.
   a) The 2-in. sampler measures 2-in. I.D. and 2½-in. O.D.. Thin brass liners are enclosed in the sampler. The sampler is driven 18-in. into the soil at the bottom of the holes with a 140 lb. hammer falling 30 in.
   b) The 2½-in. sampler measures 2½-in. I.D. and 2¾-in. O.D. and also contains brass liners. This sampler is driven 24-in. into the soil with a 140 lb. hammer falling 30 in.
   c) Shelby tube samplers are thin-walled brass tubes, measuring either 2.8 or 3.2 I.D., and are pushed into the soil by hydraulic mechanism. Loss of the sample is prevented by either a fixed piston in the Osterberg type sampler or by ball check valve in the open type sampler.
4. When the sampler was withdrawn from the test holes, the brass tubes containing the soils samples were removed, carefully sealed to preserve the natural moisture content, and returned to the laboratory for testing.
5. Classifications are based on the Unified Classification System and are made in the field by our Engineer or Geologist. Classifications of in-place samples are verified by an examination by the Staff Engineer.

KEY TO BORING LOGS



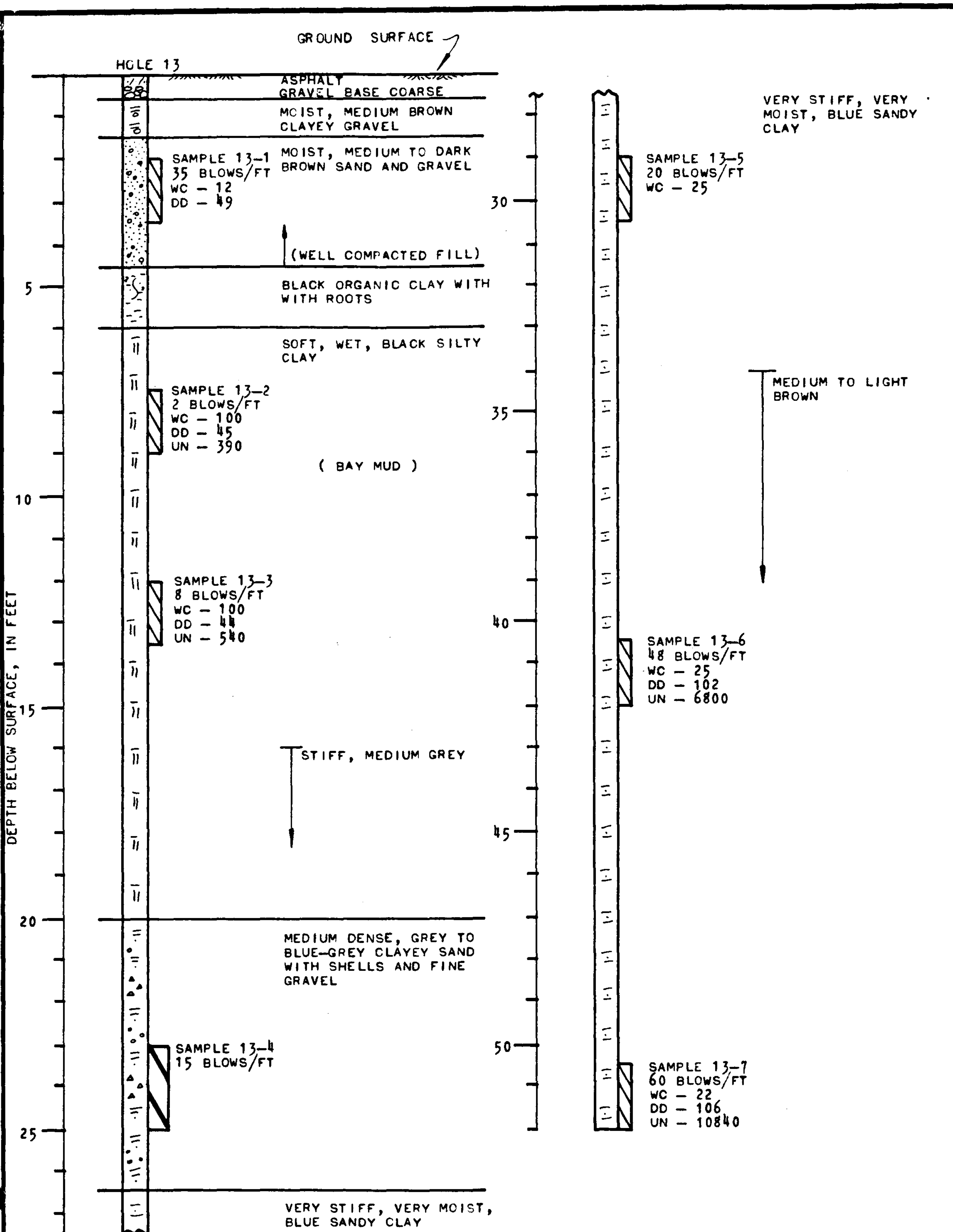


FIG. 20 — LOGS OF BORINGS

WOODWARD-CLYDE-SHERARD & ASSOCIATES
Consulting Soil and Foundation Engineers

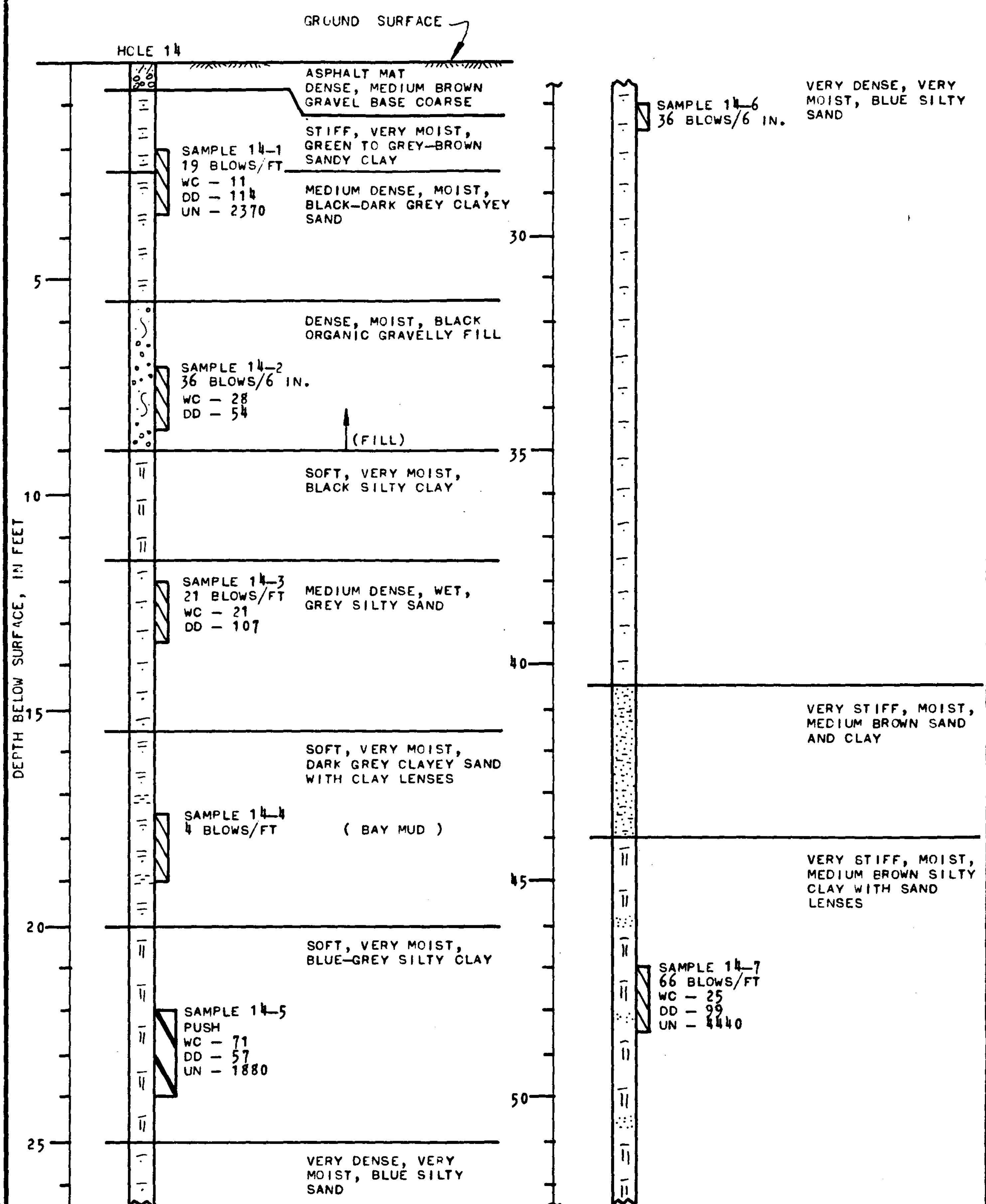


FIG. 21 — LOGS OF BORINGS

WOODWARD-CLYDE-SHERARD & ASSOCIATES
Consulting Soil and Foundation Engineers

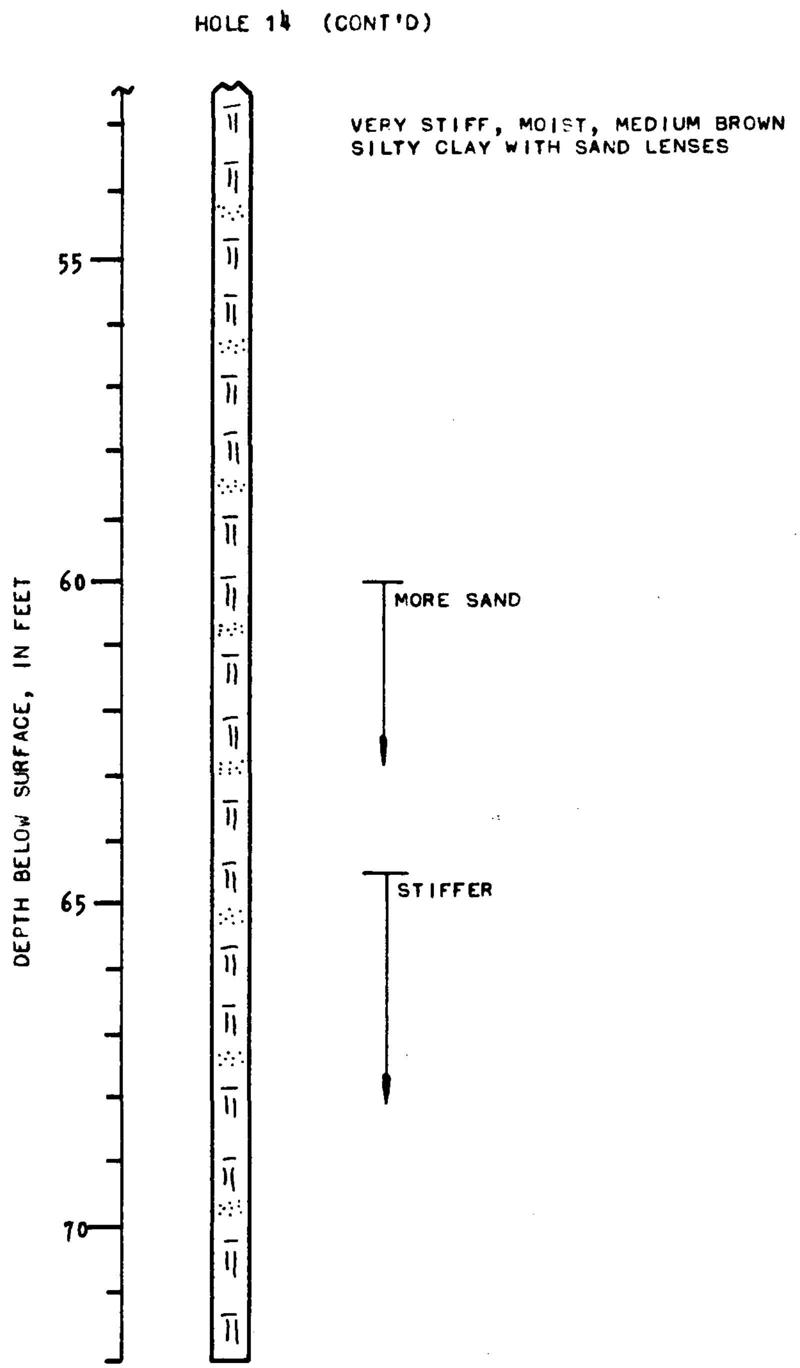
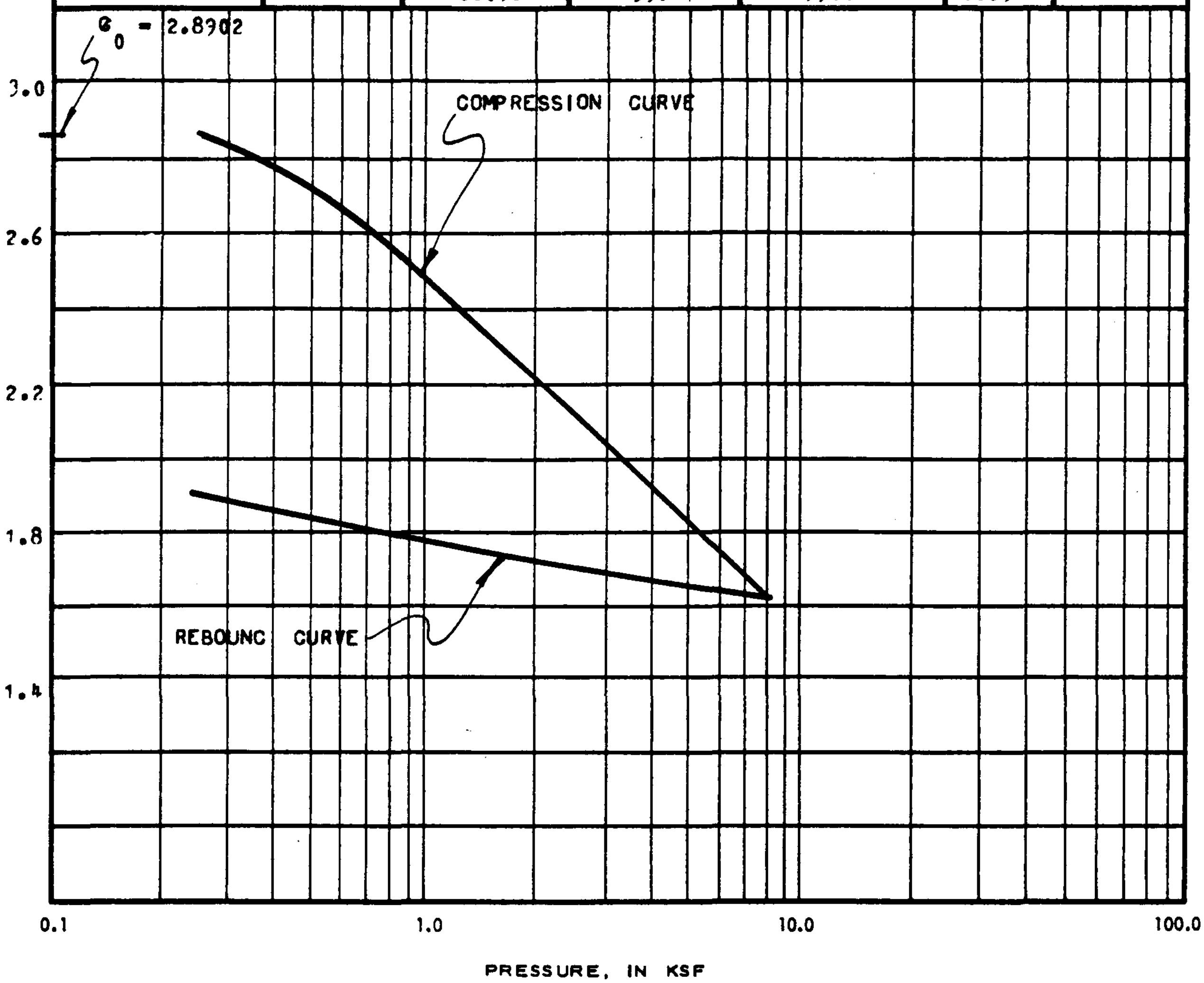
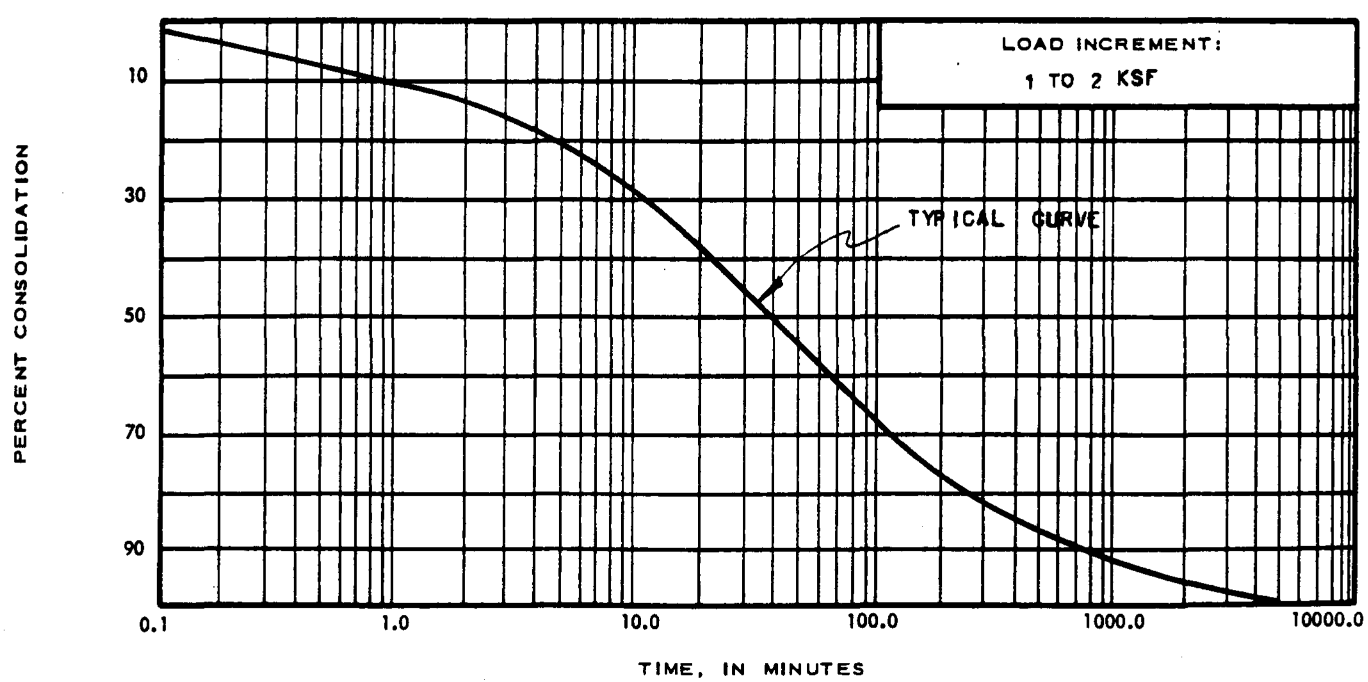


FIG. 22 — LOGS OF BORINGS

SAMPLE NO.	SUMMARY OF TEST RESULTS					
13-3-9	SPECIFIC GRAVITY	MOISTURE CONTENT, (%)	DRY DENSITY, (PCF)	PERCENT OF SATURATION, (%)	HEIGHT (IN.)	DIAMETER (IN.)
INITIAL	2.75	102.24	44.10	97.28	.8981	1.94
FINAL		68.96	53.17	99.81	.6694	





CONSOLIDATION TEST

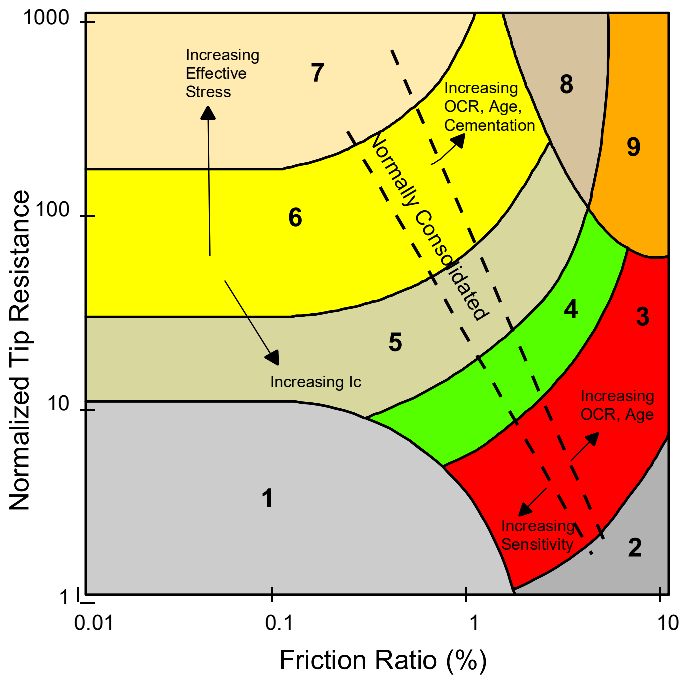
FIG. 26

Supplement D

Liquefaction Triggering and

Post-Liquefaction Deformation Analyses

CPT CORRELATION CHART
(Robertson 1990)



Zone	Soil Behavior Type
1	Sensitive Fine-grained
2	Peats
3	Silty Clay to Clay
4	Clayey Silt to Silty Clay
5	Silty Sand to Sandy Silt
6	Clean Sand to Silty Sand
7	Gravelly Sand to Dense Sand
8	Very Stiff Sand to Clayey Sand*
9	Very Stiff Fine-Grained*

*heavily overconsolidated or cemented

CPT LOG COMPONENTS

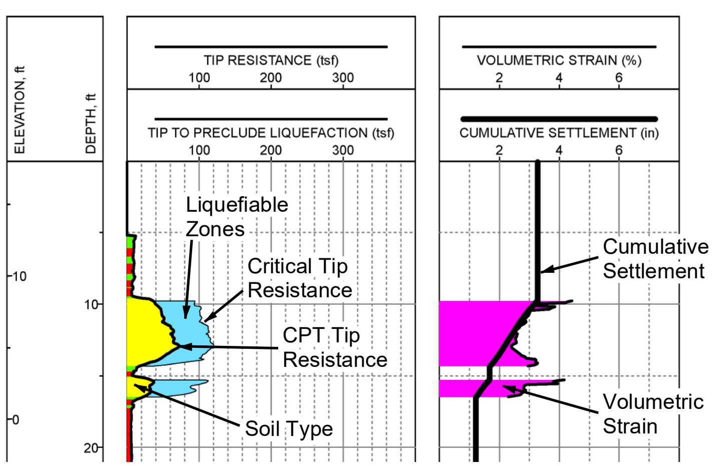
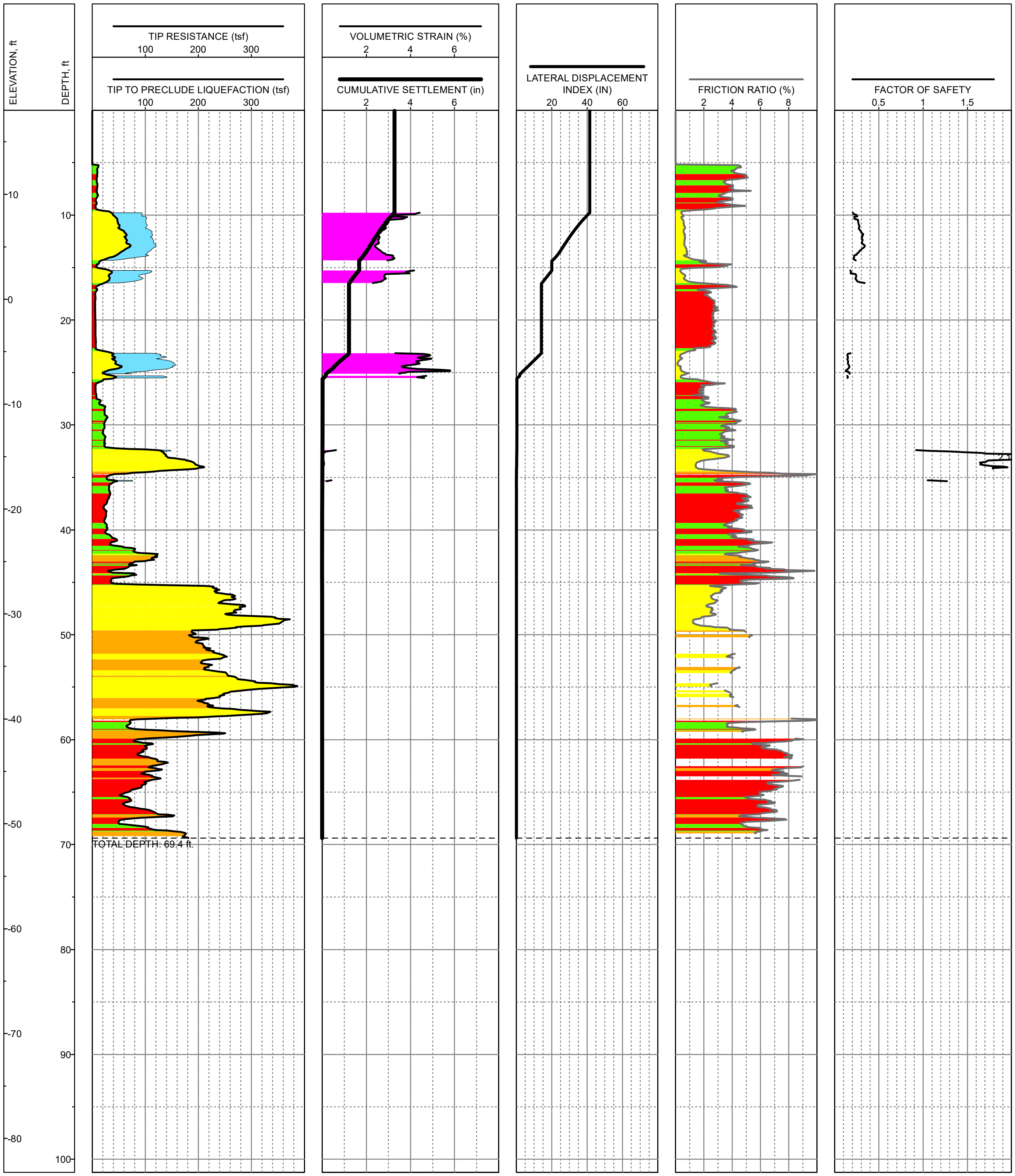


PLATE D-0: KEY TO LIQUEFACTION LOGS



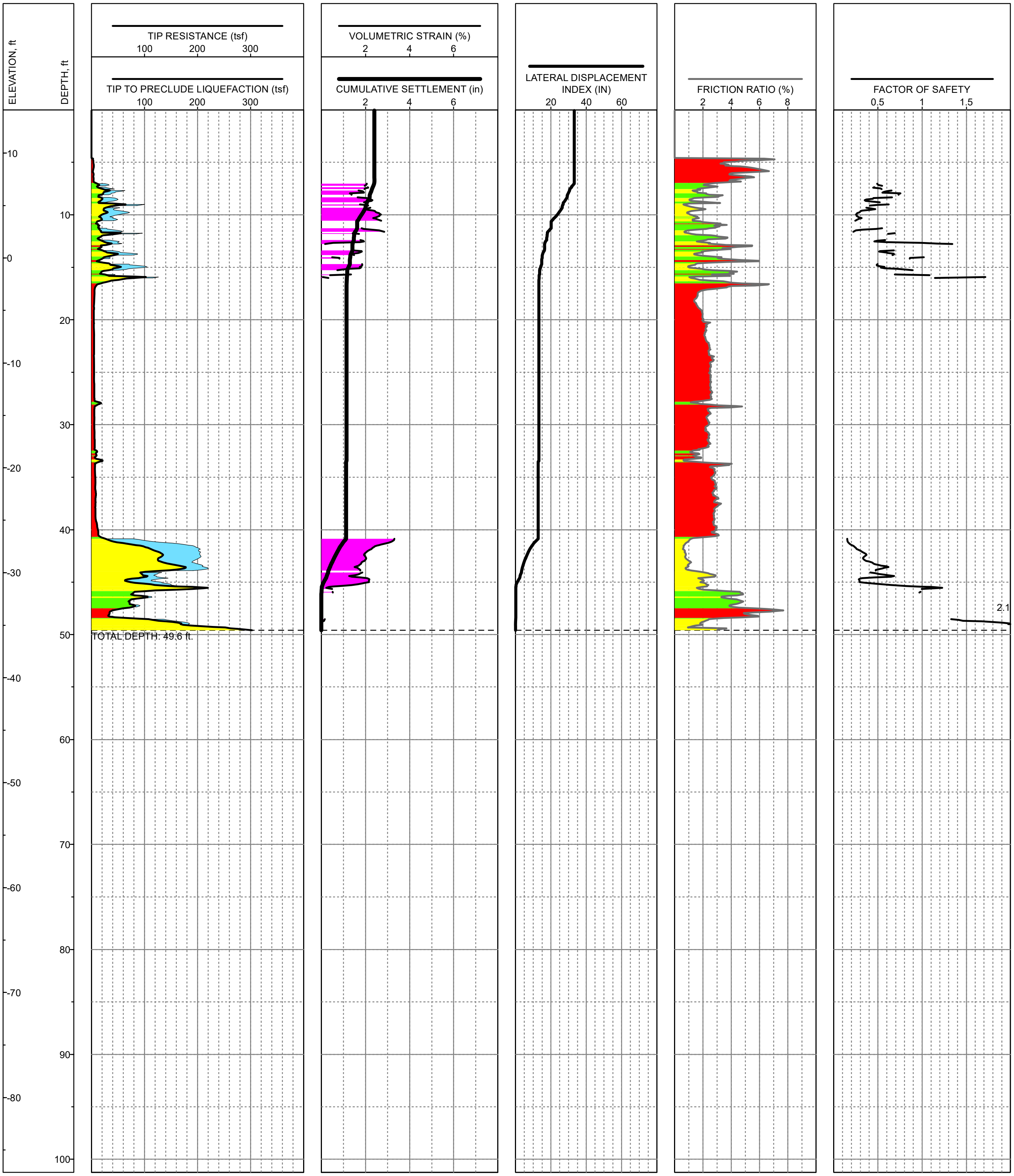
LOCATION: E6,052,365, N2,116,794, NAD83 SP CA Z3 FT
SURFACE EL: 18.02 ft
COMPLETION DEPTH: 69.4 ft
TESTDATE: 3/29/2019

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.59
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\01_Explorations\CPT\2019\Logs\2019_06_18_Logs_M7_0_a0_810_N_TL_TR\MXD\CPT_Logs_M7_0_a0_810_N_TL_TR.6/21/2019.A.Ramirez

PLATE D-1: LOG OF 2019-CPT-01 – M=7.0, PGA=0.810, N, TL, TR

FUGRO

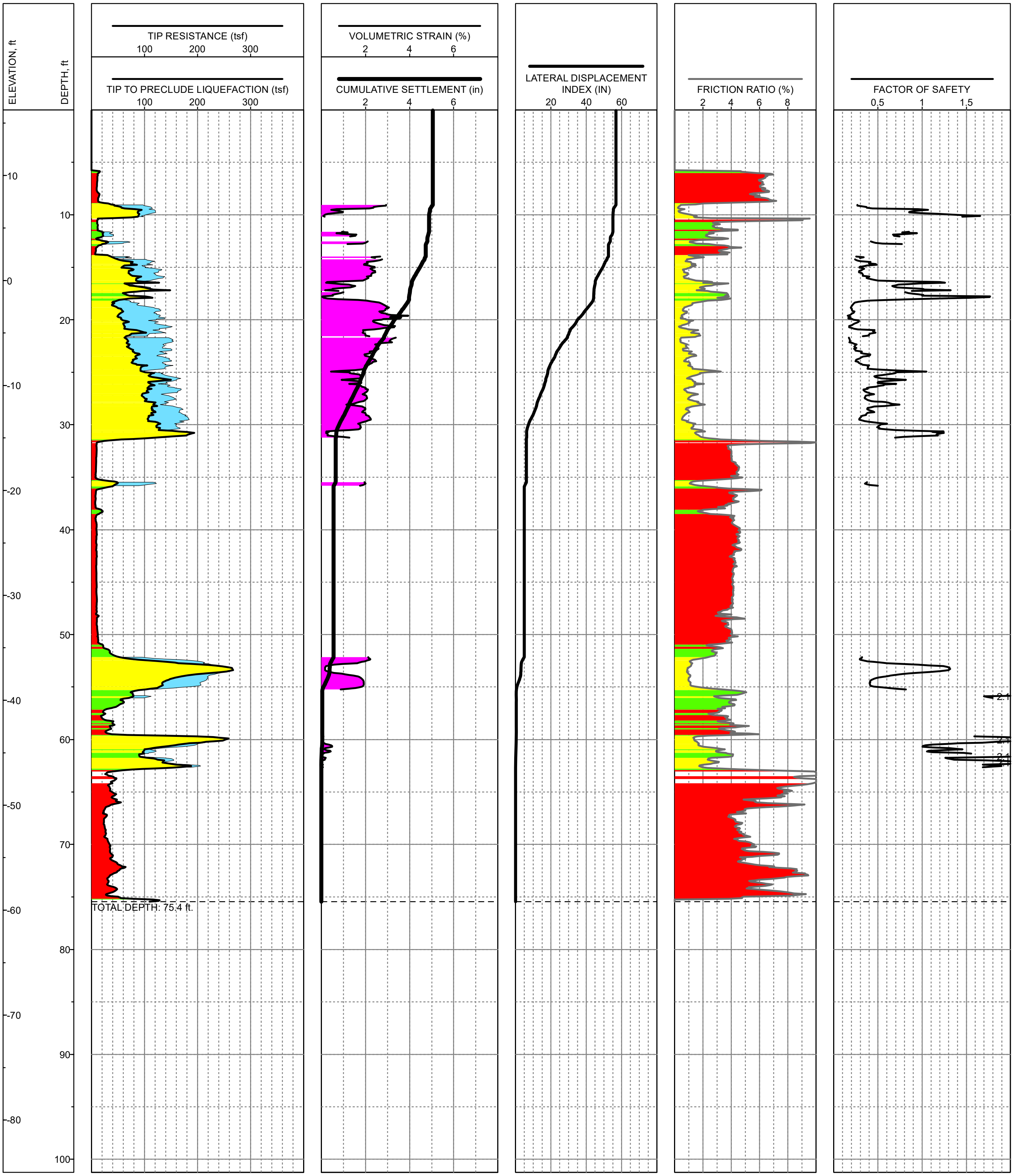


LOCATION: E6,052,593, N2,116,694, NAD83 SP CA Z3 FT
SURFACE EL: 14.11 ft
COMPLETION DEPTH: 49.6 ft
TESTDATE: 3/29/2019

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.59
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

PLATE D-2: LOG OF 2019-CPT-02 – M=7.0, PGA=0.810, N, TL, TR

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\01_Explorations\CPT\2019\Logs\2019_06_18_Logs_M7_0_a0_810_N_TL_TR\MXD\CPT_Logs_M7_0_a0_810_N_TL_TR,6/21/2019,A.Ramirez



LOCATION: E6,052,570, N2,116,535, NAD83 SP CA Z3 FT
SURFACE EL: 16.26 ft
COMPLETION DEPTH: 75.4 ft
TESTDATE: 3/29/2019

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.59
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

PLATE D-3: LOG OF 2019-CPT-03 – M=7.0, PGA=0.810, N, TL, TR

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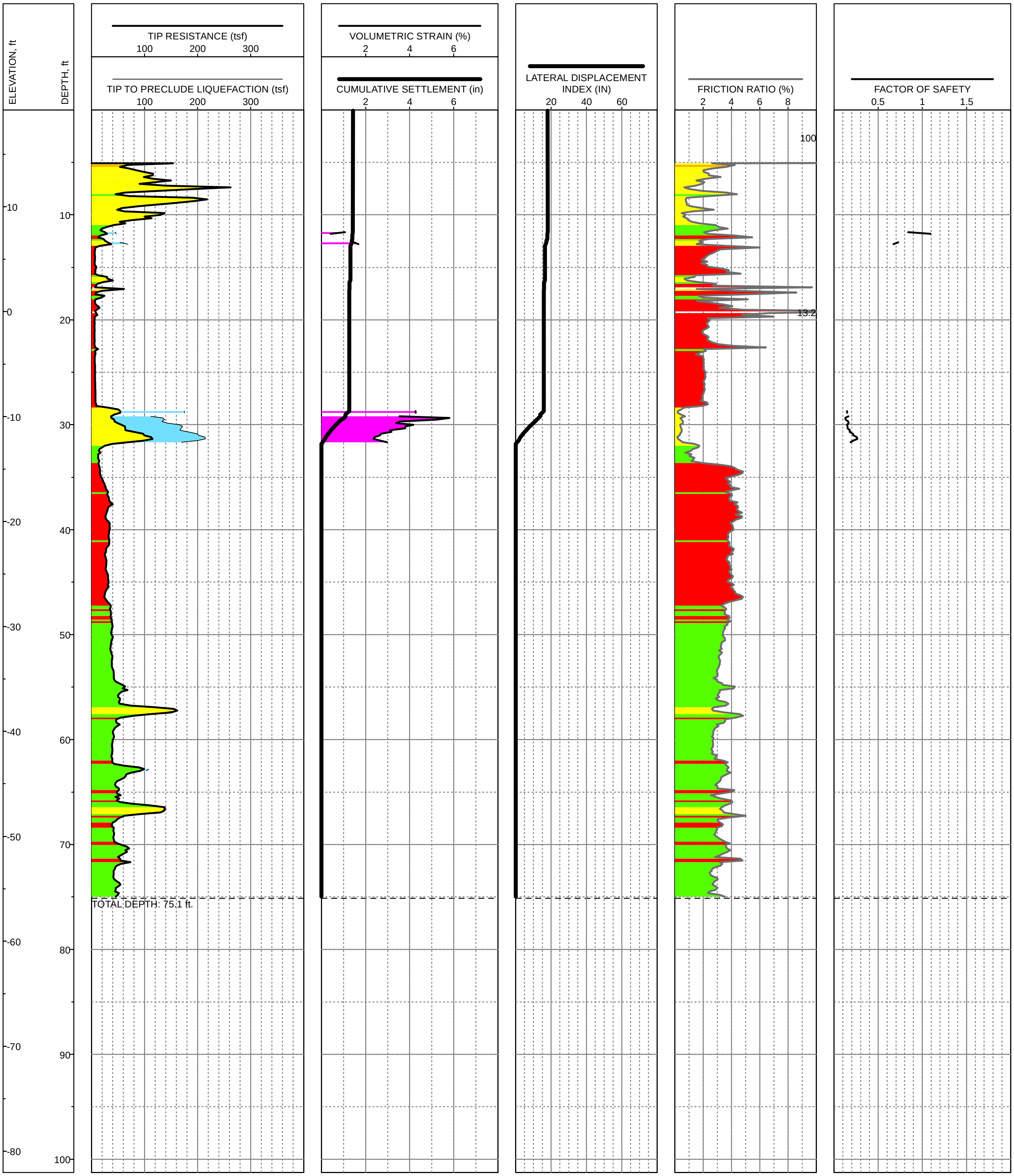
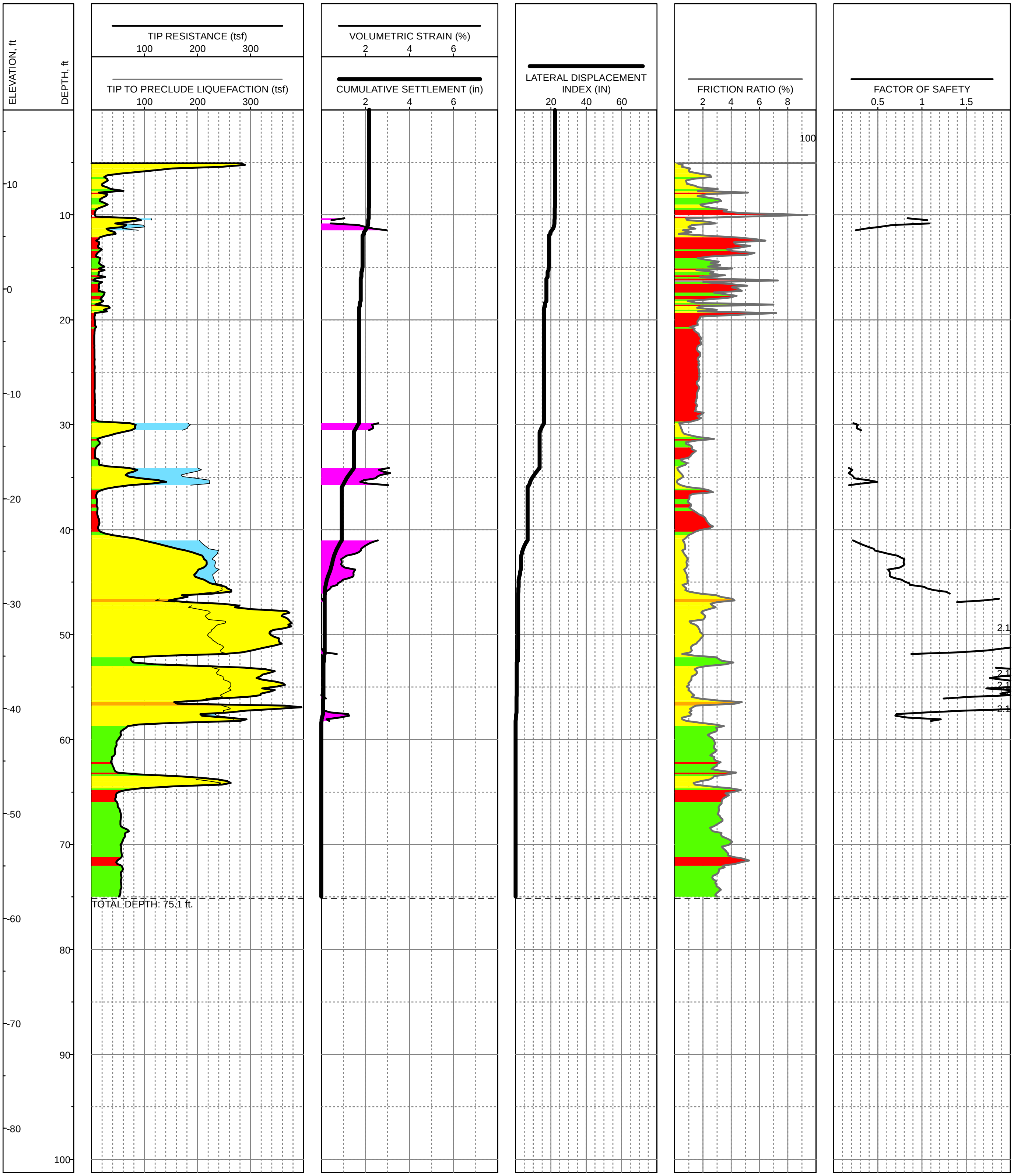


LOCATION: E6,052,490, N2,116,767, NAD83 SP CA Z3 FT
SURFACE EL: 19.2 ft
COMPLETION DEPTH: 75.1 ft
TESTDATE: 3.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

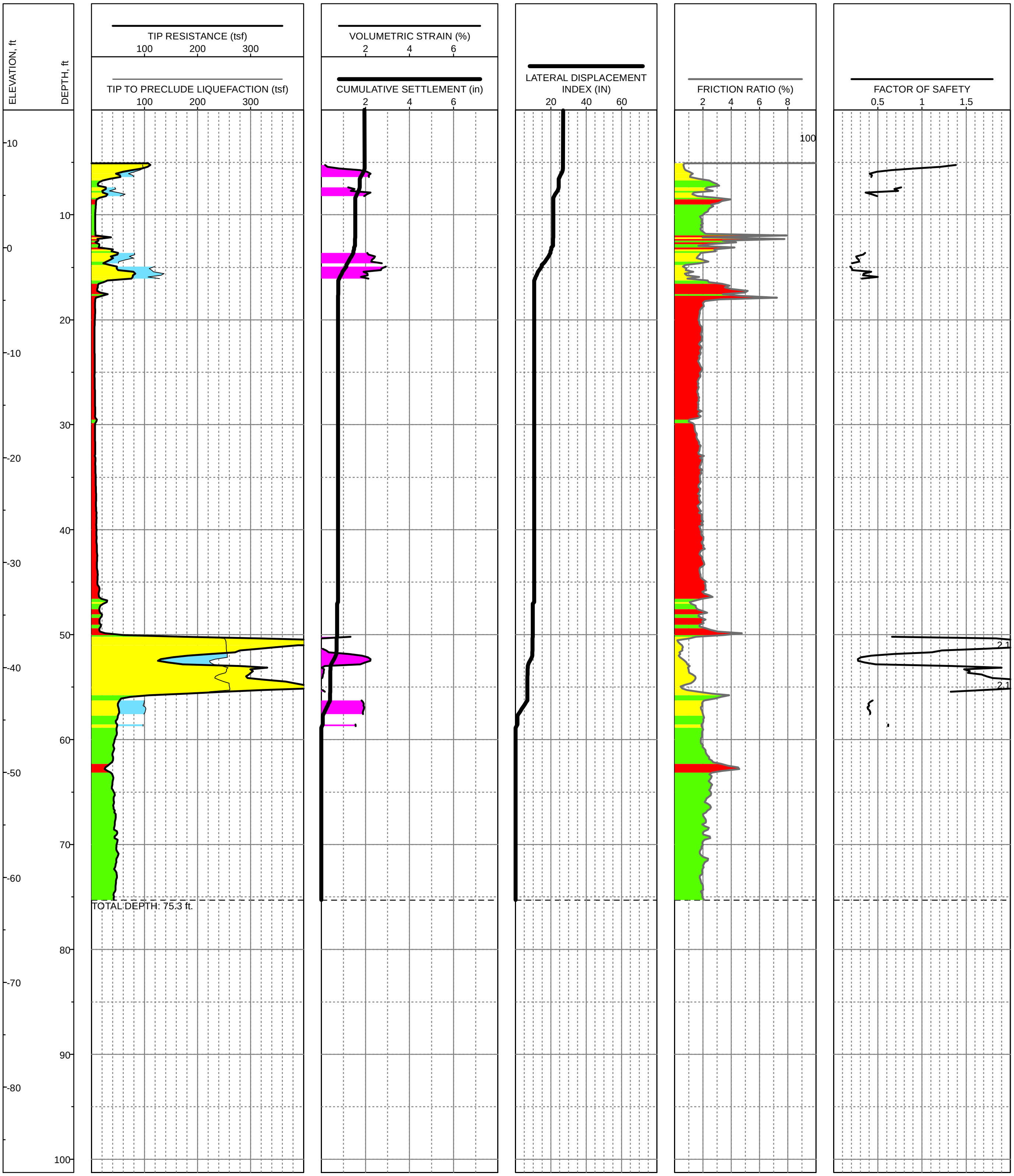
PLATE D-4: LOG OF 2020-CPT-04 – M=7.0, PGA=0.810, N, TL, TR



LOCATION: E6,052,557, N2,116,734, NAD83 SP CA Z3 FT
SURFACE EL: 17.1 ft
COMPLETION DEPTH: 75.1 ft
TESTDATE: 3.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

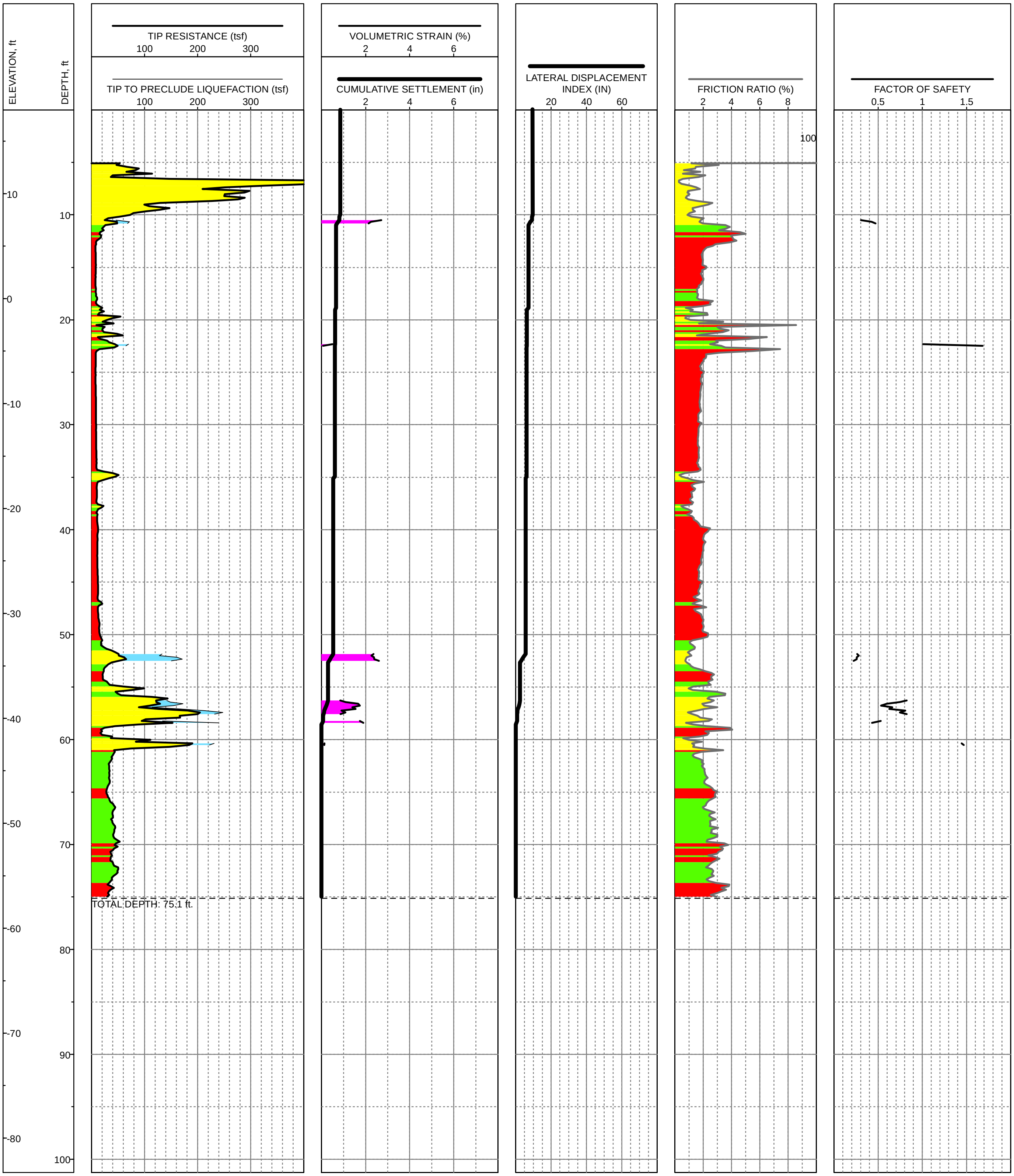
PLATE D-5: LOG OF 2020-CPT-05 – M=7.0, PGA=0.810, N, TL, TR



LOCATION: E6,052,632, N2,116,632, NAD83 SP CA Z3 FT
SURFACE EL: 13.1 ft
COMPLETION DEPTH: 75.3 ft
TESTDATE: 3.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

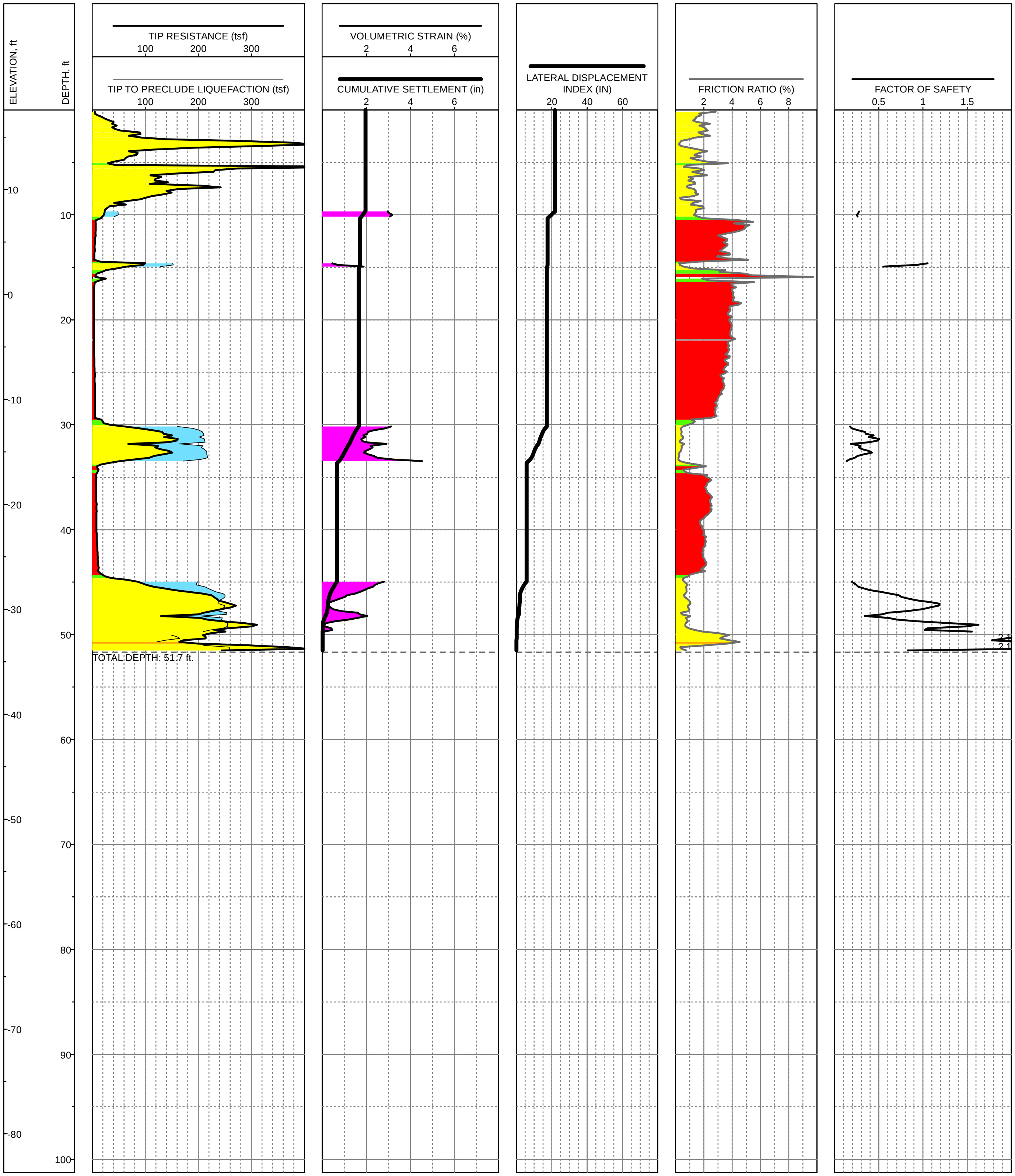
PLATE D-6: LOG OF 2020-CPT-06 – M=7.0, PGA=0.810, N, TL, TR



LOCATION: E6,052,572, N2,116,598, NAD83 SP CA Z3 FT
SURFACE EL: 18.0 ft
COMPLETION DEPTH: 75.1 ft
TESTDATE: 3.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

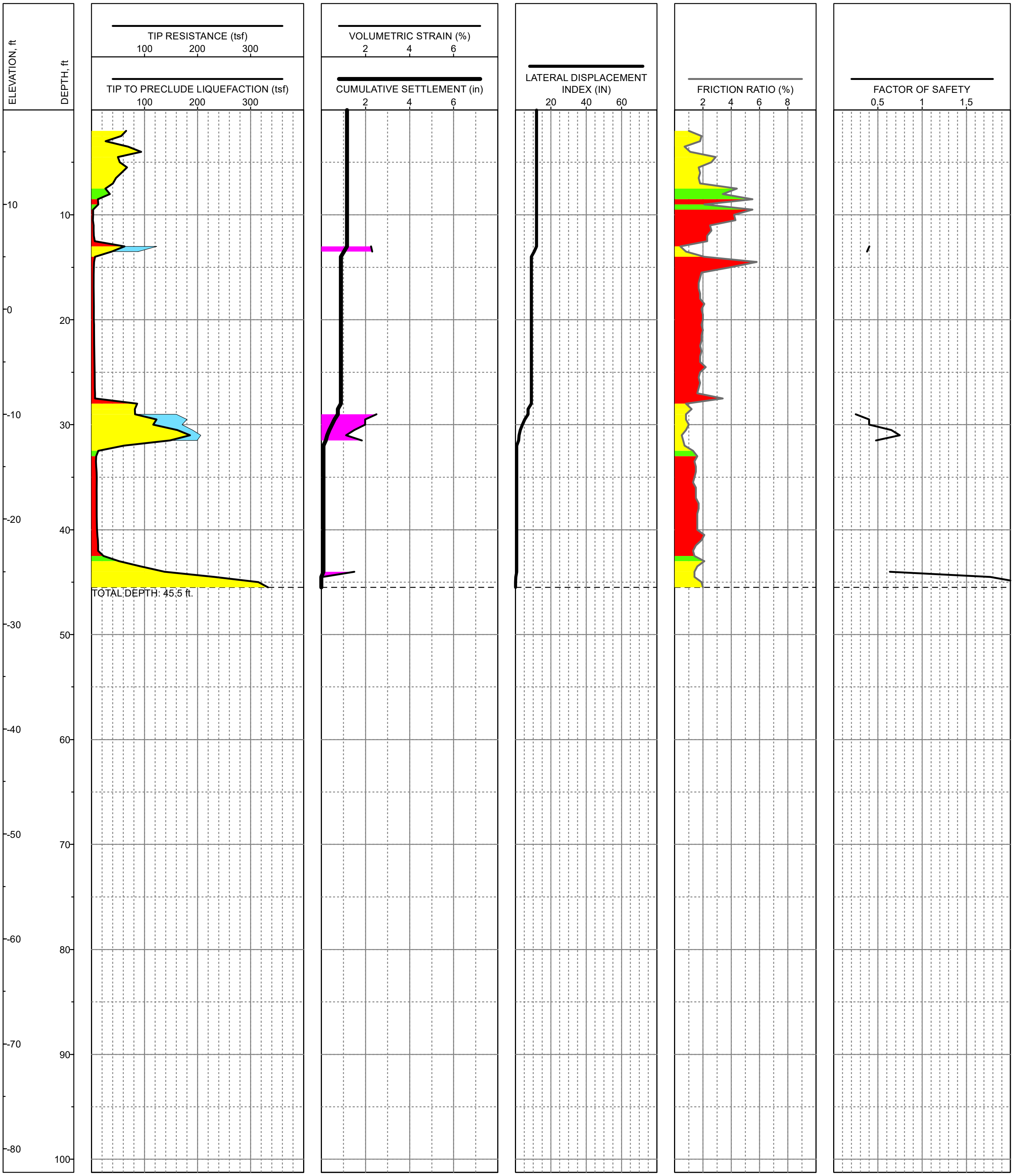
PLATE D-7: LOG OF 2020-CPT-07 – M=7.0, PGA=0.810, N, TL, TR



LOCATION: E6,052,485, N2,116,625, NAD83 SP CA Z3 FT
SURFACE EL: 17.6 ft
COMPLETION DEPTH: 51.7 ft
TESTDATE: 2.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

PLATE D-8: LOG OF 2020-CPT-08 – M=7.0, PGA=0.810, N, TL, TR

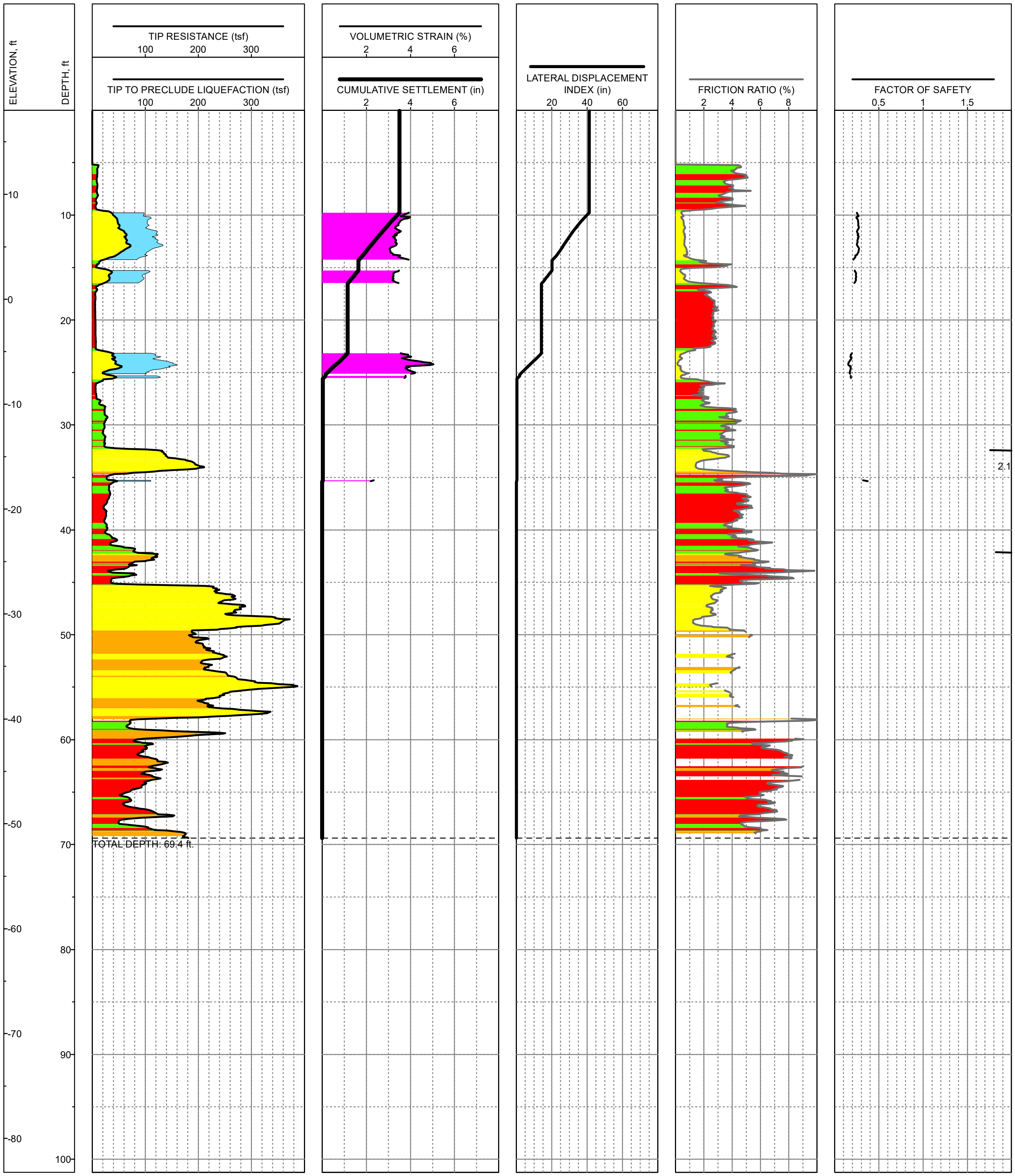


LOCATION: E6,052,497, N2,116,672, NAD83 SP CA Z3 FT
SURFACE EL: 19.00 ft
COMPLETION DEPTH: 45.5 ft
TESTDATE: 2/26/2002

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.59
LIQUEFACTION ANALYSIS GROUNDWATER DEPTH: 8 ft

PLATE D-9: LOG OF 2002-CPT-2 – M=7.0, PGA=0.810, N, TL, TR

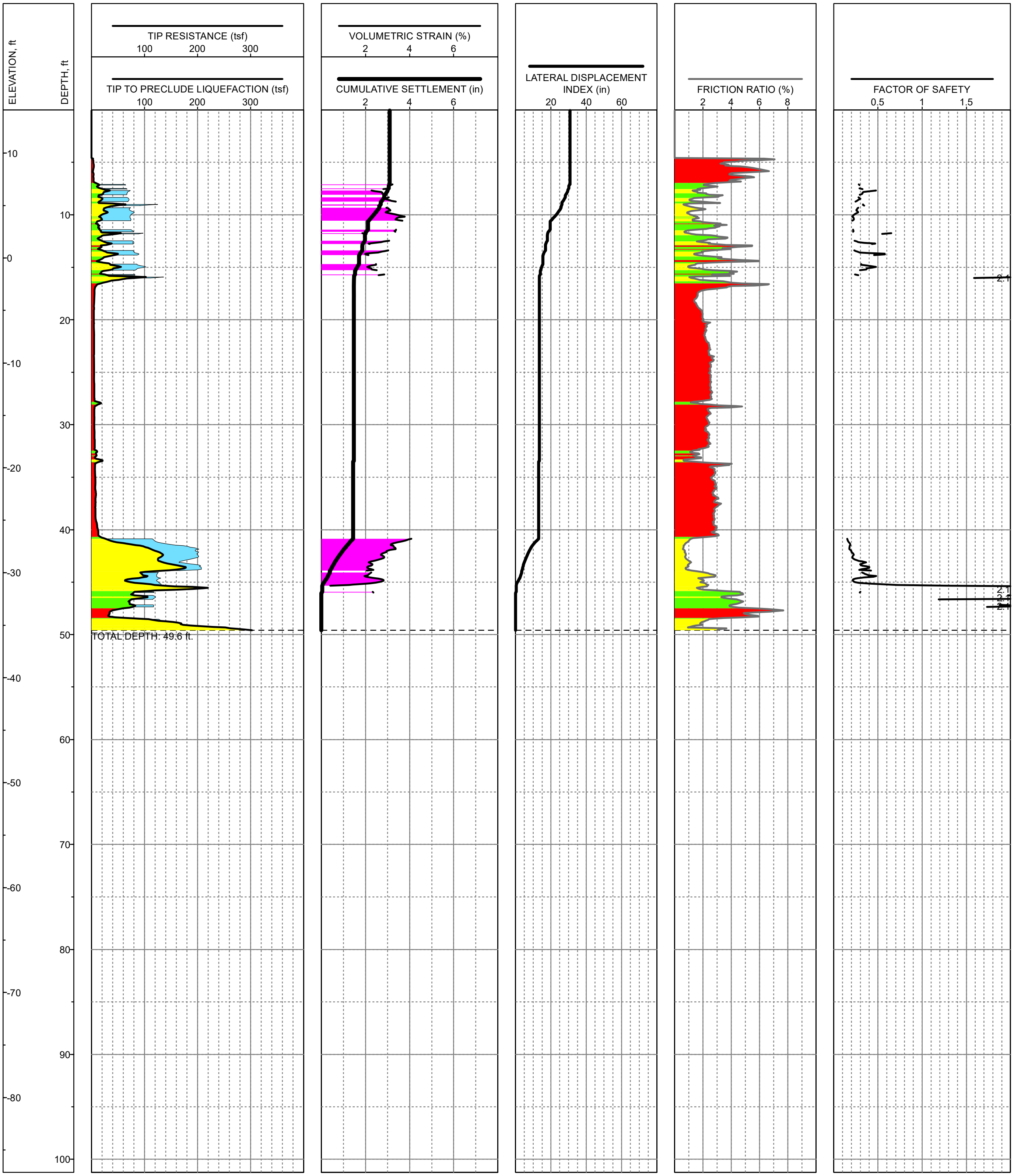
W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\01_Explorations\CPT\2002\Logs\2019_06_18_Logs_M7_0_a0_810_N_TL_TR\MXD\CPT_Logs_M7_0_a0_810_N_TL_TR,6/21/2019,A.Ramirez



LOCATION: E6,052,365, N2,116,794, NAD83 SP CA Z3 FT
SURFACE EL: 18.02 ft
COMPLETION DEPTH: 69.4 ft
TESTDATE: 3/29/2019

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.59
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

PLATE D-10: LOG OF 2019-CPT-01 – M=7.0, PGA=0.810, B, TL, TR

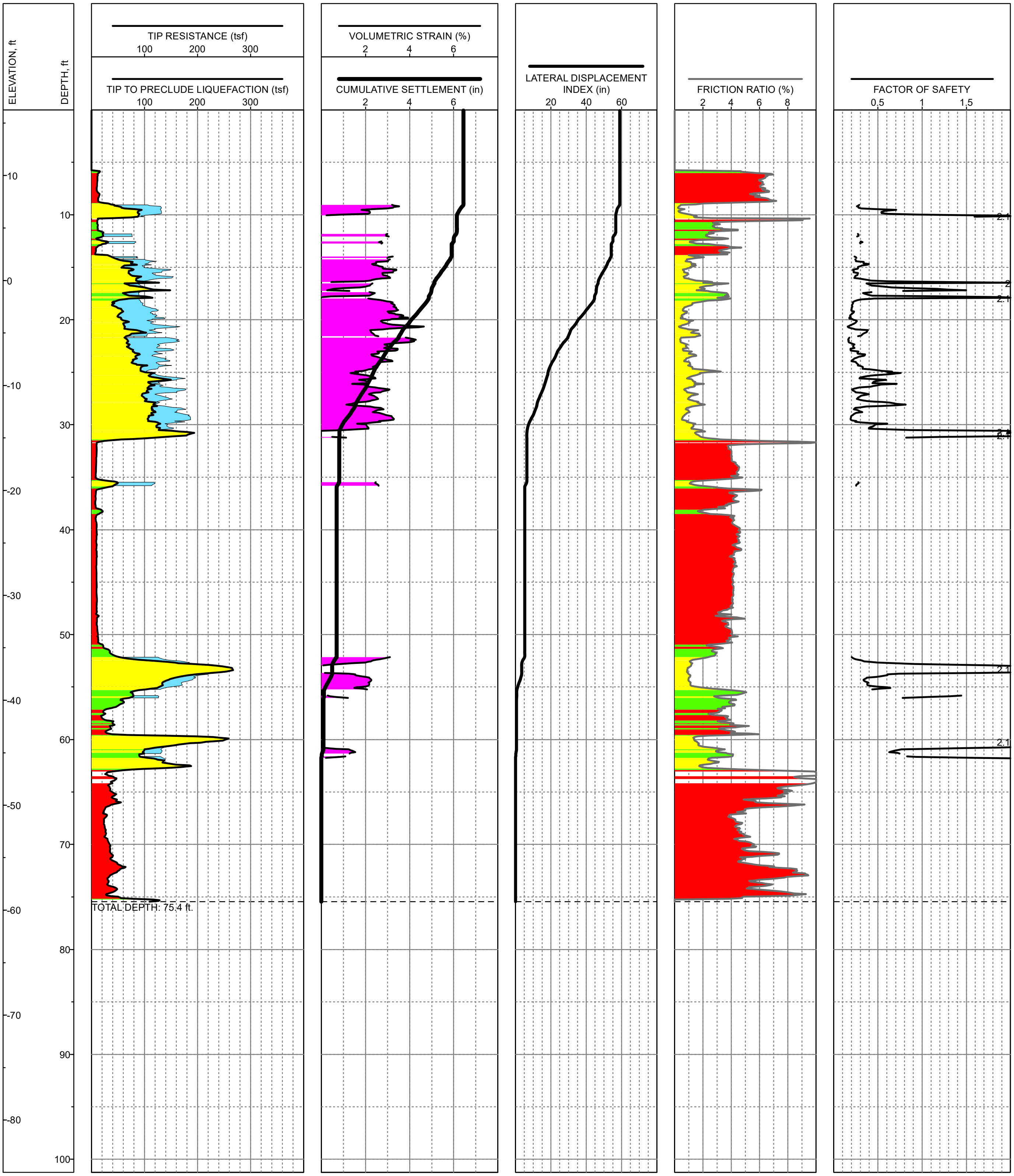


LOCATION: E6,052,593, N2,116,694, NAD83 SP CA Z3 FT
SURFACE EL: 14.11 ft
COMPLETION DEPTH: 49.6 ft
TESTDATE: 3/29/2019

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.59
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

PLATE D-11: LOG OF 2019-CPT-02 – M=7.0, PGA=0.810, B, TL, TR

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\01_Explorations\CPT\2019\Logs\2019_06_18_Logs_M7_0_a0_810_B_IB_TL_TR\MXD\CPT_Logs_M7_0_a0_810_B_IB_TL_TR,6/21/2019,A.Ramirez

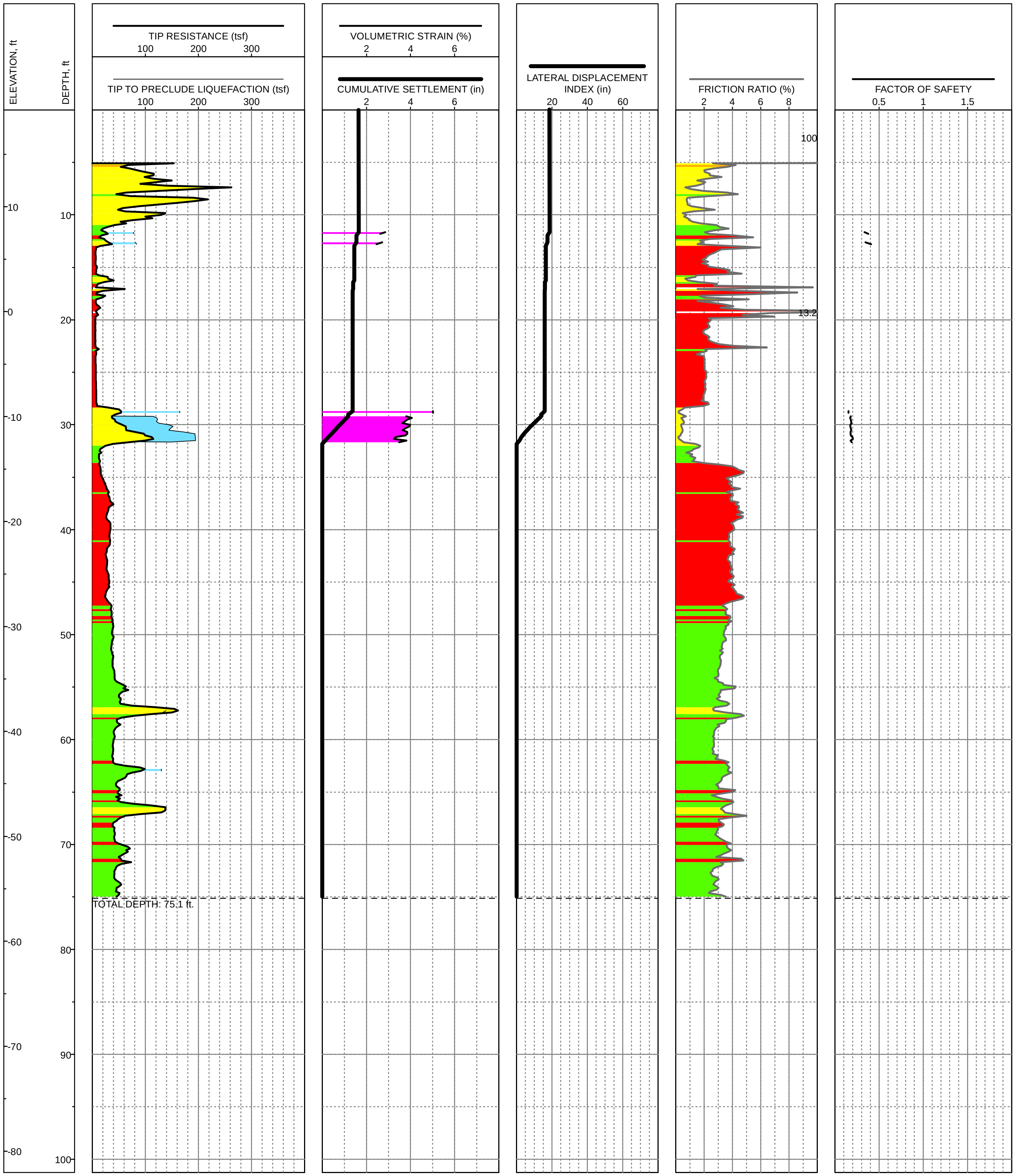


LOCATION: E6,052,570, N2,116,535, NAD83 SP CA Z3 FT
SURFACE EL: 16.26 ft
COMPLETION DEPTH: 75.4 ft
TESTDATE: 3/29/2019

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.59
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\01_Explorations\CPT\2019\Logs\2019_06_18_Logs_M7_0_a0_810_B_IB_TL_TRIMXD\CPT_Logs_M7_0_a0_810_B_IB_TL_TR,6/21/2019,A.Ramirez

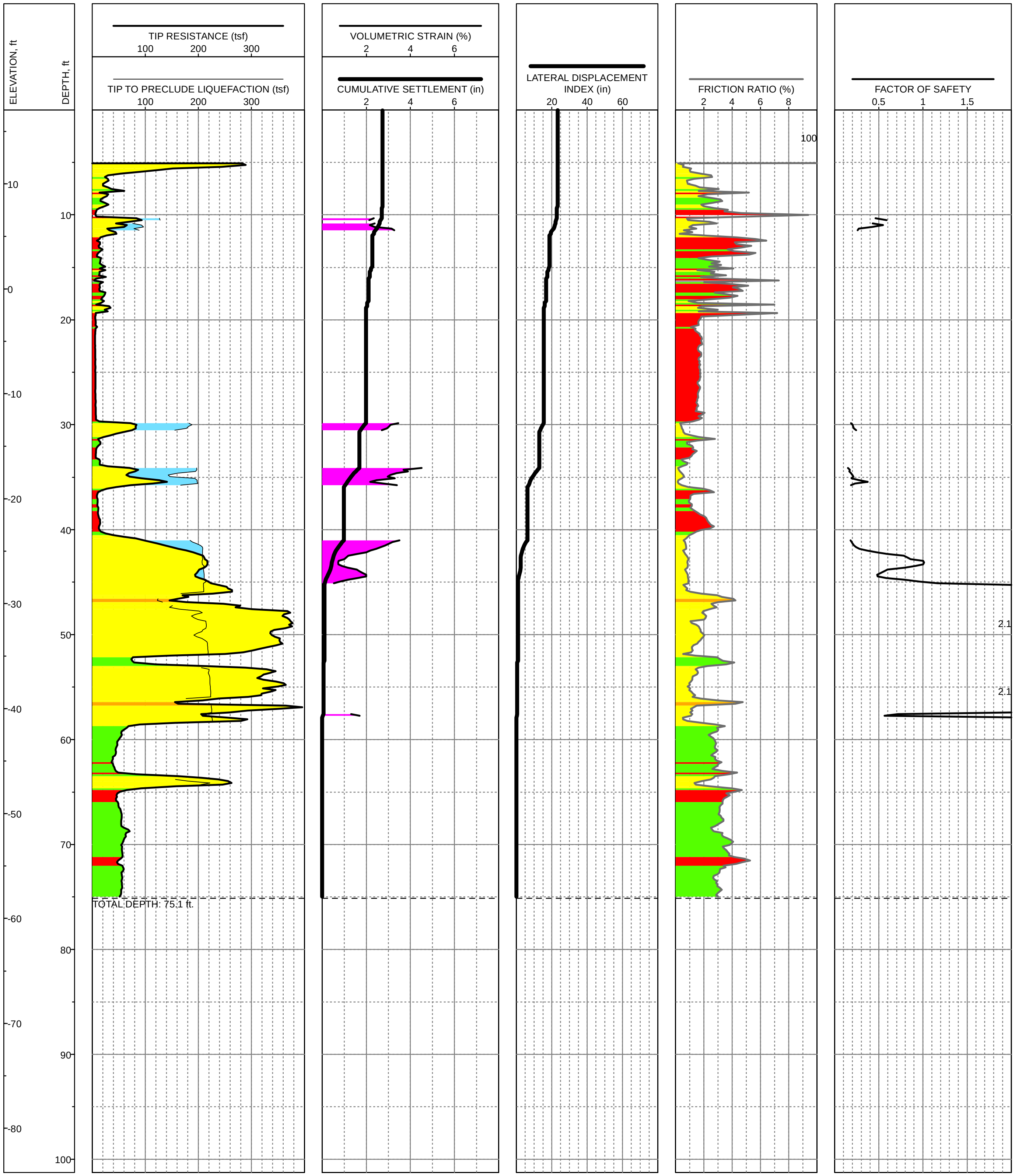
PLATE D-12: LOG OF 2019-CPT-03 – M=7.0, PGA=0.810, B, TL, TR



LOCATION: E6,052,490, N2,116,767, NAD83 SP CA Z3 FT
SURFACE EL: 19.2 ft
COMPLETION DEPTH: 75.1 ft
TESTDATE: 3.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

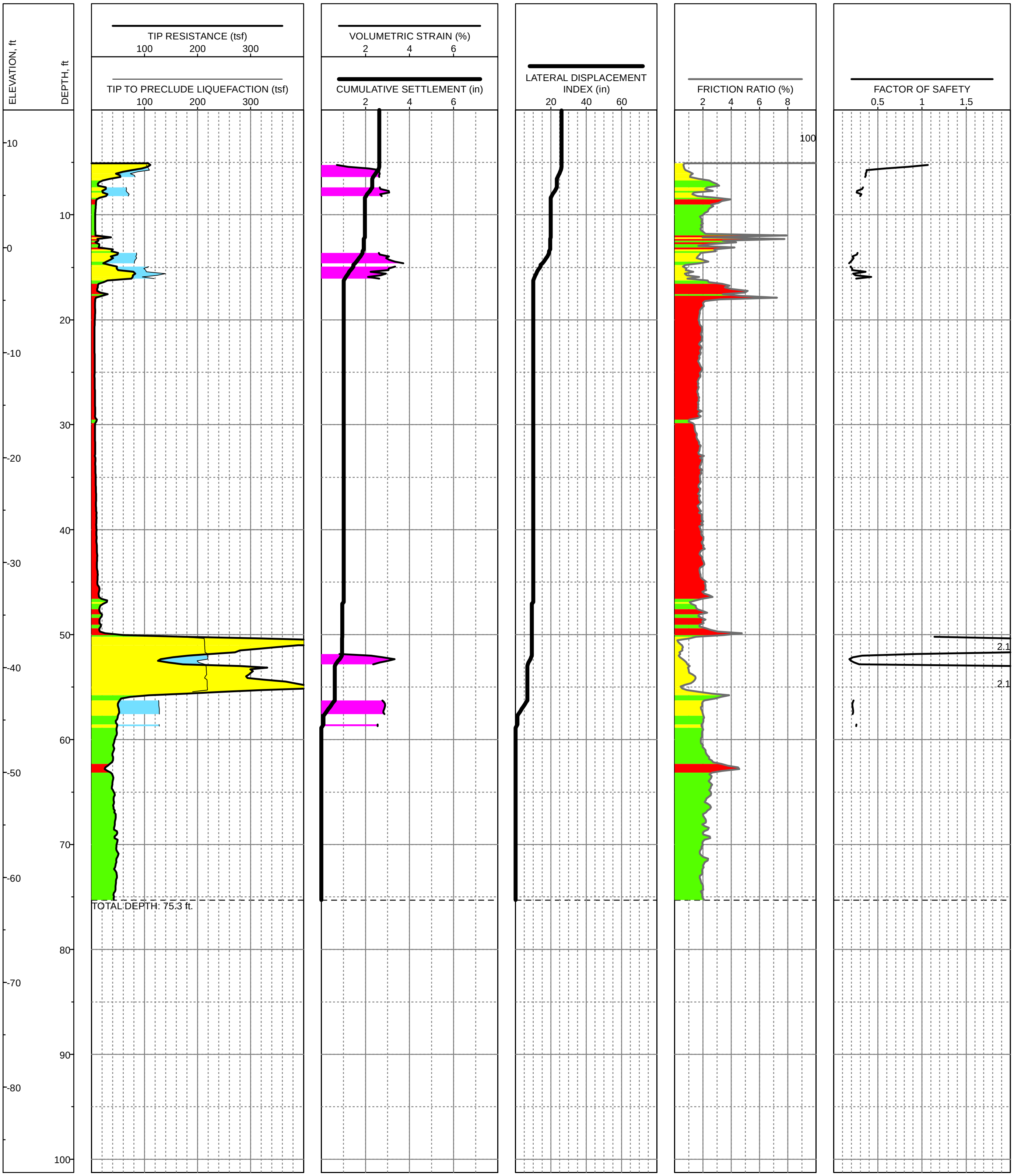
PLATE D-13: LOG OF 2020-CPT-04 – M=7.0, PGA=0.810, B, TL, TR



LOCATION: E6,052,557, N2,116,734, NAD83 SP CA Z3 FT
SURFACE EL: 17.1 ft
COMPLETION DEPTH: 75.1 ft
TESTDATE: 3.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

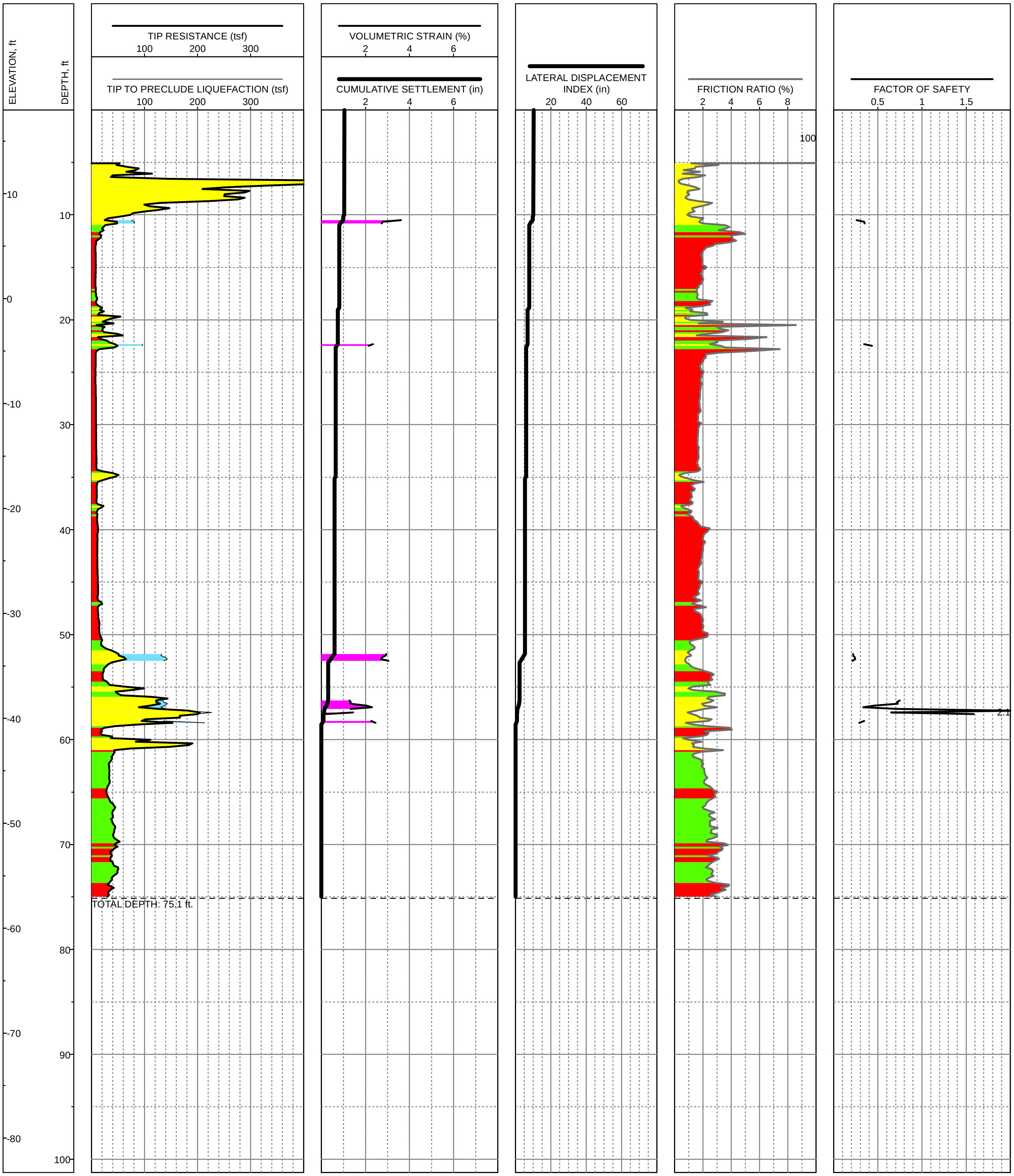
PLATE D-14: LOG OF 2020-CPT-05 – M=7.0, PGA=0.810, B, TL, TR



LOCATION: E6,052,632, N2,116,632, NAD83 SP CA Z3 FT
SURFACE EL: 13.1 ft
COMPLETION DEPTH: 75.3 ft
TESTDATE: 3.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

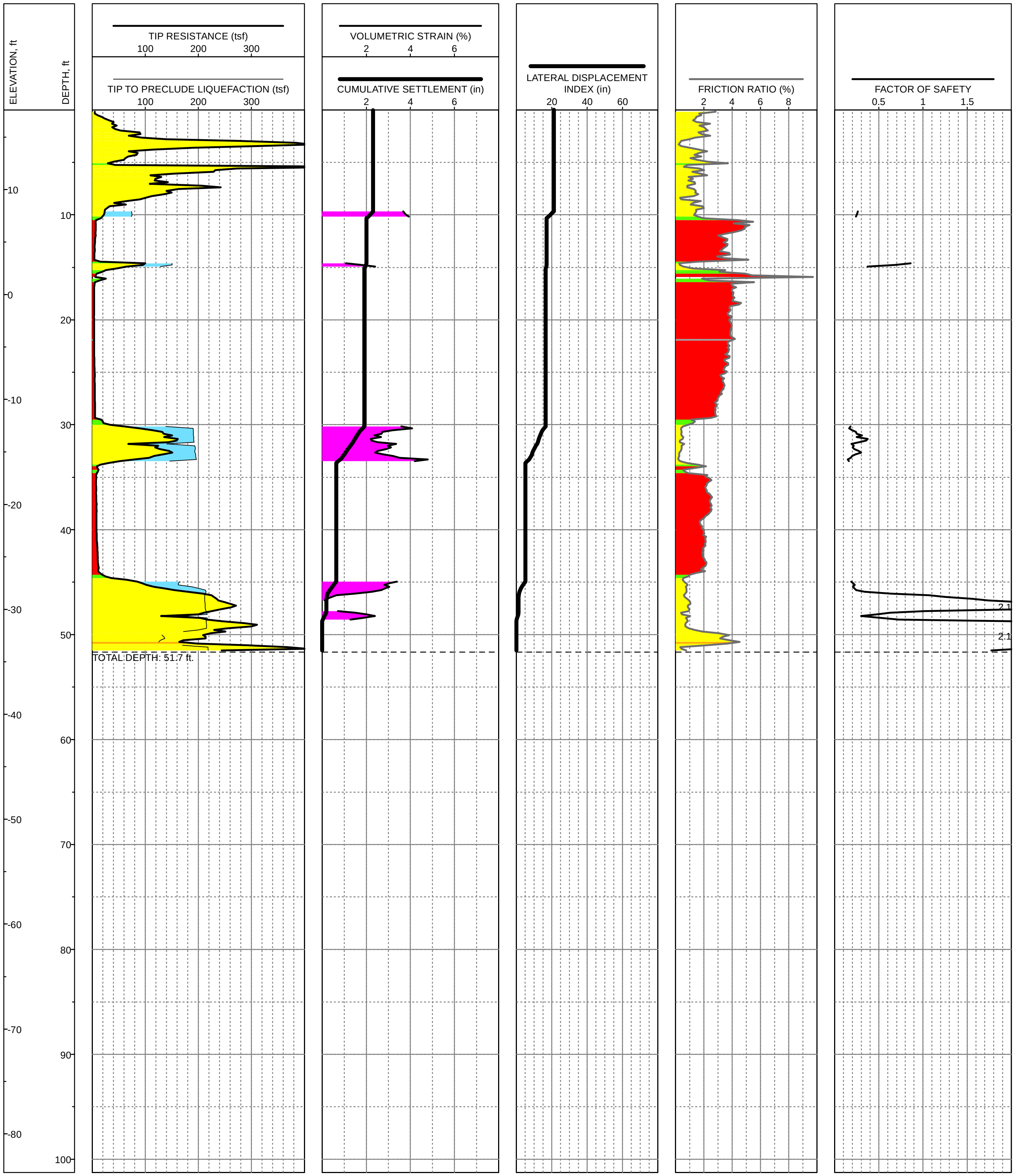
PLATE D-15: LOG OF 2020-CPT-06 – M=7.0, PGA=0.810, B, TL, TR



LOCATION: E6,052,572, N2,116,598, NAD83 SP CA Z3 FT
SURFACE EL: 18.0 ft
COMPLETION DEPTH: 75.1 ft
TESTDATE: 3.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

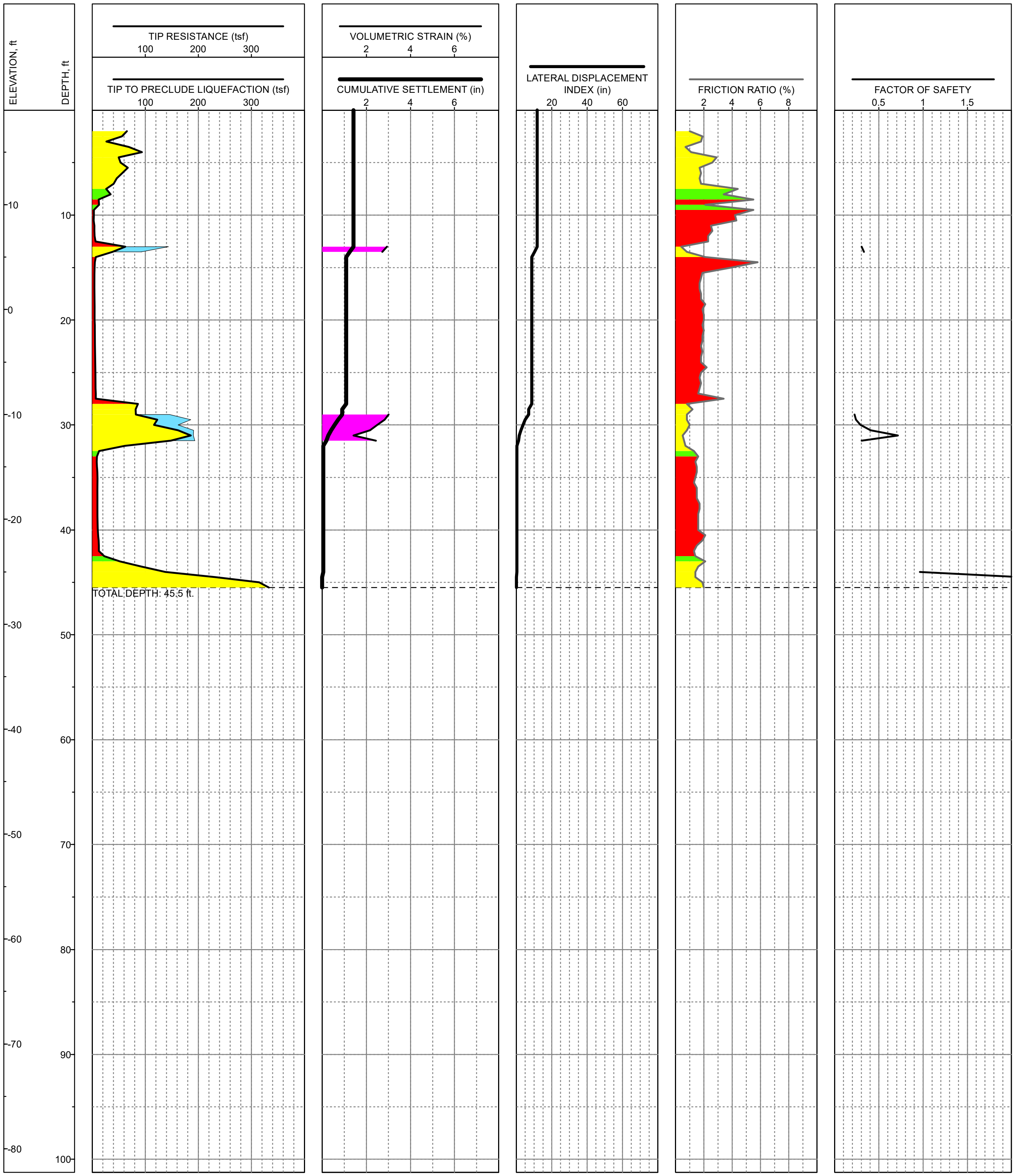
PLATE D-16: LOG OF 2020-CPT-07 – M=7.0, PGA=0.810, B, TL, TR



LOCATION: E6,052,485, N2,116,625, NAD83 SP CA Z3 FT
SURFACE EL: 17.6 ft
COMPLETION DEPTH: 51.7 ft
TESTDATE: 2.01.2020

EXPLORATION METHOD: CPT
PERFORMED BY: GREGG DRILLING
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.80
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

PLATE D-17: LOG OF 2020-CPT-08 – M=7.0, PGA=0.810, B, TL, TR



LOCATION: E6,052,497, N2,116,672, NAD83 SP CA Z3 FT
SURFACE EL: 19.00 ft
COMPLETION DEPTH: 45.5 ft
TESTDATE: 2/26/2002

EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: T. CHEN
CONE AREA RATIO: 0.59
LIQUEFACTION ANALYSIS GROUNDWATER EL: 8 ft

PLATE D-18: LOG OF 2002-CPT-2 – M=7.0, PGA=0.810, B, TL, TR

LIQUEFACTION ANALYSES BASED ON SPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
1/28/20 TC

a_{max} = 0.81 g ASCE 7-16
Mw = 7.0

2020-B-01
Ground Elevation = 17.5 ft
Depth to Ground Water Table = 9.5 ft = EL 8 ft
γ = 110 pcf
γ_{sat} = 120 pcf
Boring Diameter = 4 inch = 101.6 mm
Rod Length Above Ground = 3 ft = 0.9 m

Elevation	Depth	Depth	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N
ft	*ft*	*m*	*blow/ft*	*psf*	*kPa*	*psf*	*kPa*		*Y/N*				
6.5	11.0	3.4	9	1,225.0	58.7	1,131.4	54.2	0.85	N	1.00	1	1.4	1.41
5	12.5	3.8	8	1,405.0	67.4	1,217.8	58.4	0.85	Y	1.10	1	1.4	1.36
1.5	16.0	4.9	16	1,825.0	87.5	1,419.4	68.0	0.95	N	1.00	1	1.4	1.21
-13.5	31.0	9.4	14	3,625.0	173.8	2,283.4	109.5	1.00	N	1.00	1	1.4	0.96
-33.5	51.0	15.5	24	6,025.0	288.8	3,435.4	164.7	1.00	N	1.00	1	1.4	0.82
-37.5	55.0	16.8	96	6,505.0	311.8	3,665.8	175.7	1.00	N	1.00	1	1.4	0.86
-47.5	65.0	19.8	64	7,705.0	369.4	4,241.8	203.4	1.00	N	1.00	1	1.4	0.83

Elevation	$N_{1,60}$	FC	ΔN	$N_{1,60,cs}$	r_d	CSR	MSF	K_σ	$CRR_{M=7.5,1 atm}$	CRR	FS	γlim	$F\alpha$	γmax	ε_v	ΔH (ft)	ΔS (in)	ΔLDI (in)
ft		*%*		*blow/ft*											*%*	*ft*	*in*	
6.5	15	21	5	20	0.97	0.55	1.141	1.08	0.20	0.25	0.5	0.164	0.531	0.164	2.3	1.0	0.3	2.0
5	14	21	5	19	0.96	0.58	1.141	1.07	0.19	0.24	0.4	0.179	0.574	0.179	2.4	2.5	0.7	5.4
1.5	26	6	0	26	0.95	0.64	1.141	1.07	0.31	0.37	0.6	0.081	0.188	0.081	1.8	3.0	0.7	2.9
-13.5	19	8	0	19	0.87	0.72	1.141	0.99	0.20	0.22	0.3	0.174	0.559	0.174	2.4	5.5	1.6	11.5
-33.5	28	16	4	31	0.76	0.70	1.141	0.90	0.56	0.58	0.8	0.040	-0.163	0.040	0.8	3.0	0.3	1.4
-37.5	116	17	4	120	0.74	0.69	1.141	0.84	2.00	1.91	2.0	0.000	-8.012	0.000	0.0	5.0	0.0	0.0
-47.5	75	19	4	79	0.69	0.66	1.141	0.79	2.00	1.81	2.0	0.000	-4.090	0.000	0.0	11.5	0.0	0.0
														Total	2.0	15.0	3.5	23.2

PLATE D-19: BORING 2020-B-01

LIQUEFACTION ANALYSES BASED ON SPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
5/15/19 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2002-EB-1

Ground Elevation =			19.8	ft			
Depth to Ground Water Table =			11.8	ft	= EL	8	ft
γ =	110	pcf					
γ_{sat} =	120	pcf					
Boring Diameter =	8	inch =	203.2	mm			
Rod Length Above Ground =	3	ft =	0.9	m			

Elevation	Depth	Depth	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N	$N_{1,60}$
ft	*ft*	*m*	*blow/ft*	*psf*	*kPa*	*psf*	*kPa*		*Y/N*					
-14.7	34.5	10.5	6	4,022.0	192.8	2,605.5	124.9	1.00	N	1.00	1.15	1	0.88	6
-24.7	44.5	13.6	20	5,222.0	250.3	3,181.5	152.5	1.00	N	1.00	1.15	1	0.83	19
-29.2	49	14.9	100	5,762.0	276.2	3,440.7	164.9	1.00	N	1.00	1.15	1	0.88	101
-34.7	54.5	16.6	37	6,422.0	307.9	3,757.5	180.1	1.00	Y	1.30	1.15	1	0.83	46
-39.7	59.5	18.1	63	7,022.0	336.6	4,045.5	193.9	1.00	Y	1.30	1.15	1	0.84	79
-44.7	64.5	19.7	38	7,622.0	365.4	4,333.5	207.7	1.00	N	1.00	1.15	1	0.80	35
-49.7	69.5	21.2	42	8,222.0	394.2	4,621.5	221.6	1.00	N	1.00	1.15	1	0.80	39
-54.7	74.5	22.7	46	8,822.0	422.9	4,909.5	235.4	1.00	N	1.00	1.15	1	0.80	42

Elevation	Assumed FC	ΔN	$N_{1,60,cs}$	r_d	CSR	MSF	K_σ	$CRR_{M=7.5,1 atm}$	CRR	FS	γlim	$F\alpha$	γmax	ε_v	ΔH (ft)	ΔS (in)	ΔLDI (in)
ft	*%*		*blow/ft*											*%*	*ft*	*in*	
-14.7	30	5	11	0.85	0.69	1.141	0.98	0.13	0.14	0.2	0.404	0.879	0.404	3.4	5.0	2.1	24.2
-24.7	30	5	25	0.80	0.68	1.141	0.93	0.28	0.30	0.4	0.094	0.260	0.094	1.9	5.0	1.2	5.6
-29.2	30	5	106	0.77	0.68	1.141	0.86	2.00	1.95	2.0	0.000	-6.686	0.000	0.0	5.0	0.0	0.0
-34.7	30	5	51	0.74	0.66	1.141	0.83	2.00	1.89	2.0	0.000	-1.704	0.000	0.0	5.0	0.0	0.0
-39.7	30	5	85	0.72	0.65	1.141	0.81	2.00	1.84	2.0	0.000	-4.628	0.000	0.0	5.0	0.0	0.0
-44.7	30	5	40	0.69	0.64	1.141	0.79	2.00	1.80	2.0	0.008	-0.826	0.000	0.0	5.0	0.0	0.0
-49.7	30	5	44	0.67	0.62	1.141	0.77	2.00	1.75	2.0	0.003	-1.106	0.000	0.0	5.0	0.0	0.0
-54.7	30	5	48	0.65	0.61	1.141	0.75	2.00	1.71	2.0	0.001	-1.404	0.000	0.0	5.0	0.0	0.0
													Total	2.7	10.0	3.2	29.9

PLATE D-20: BORING 2002-EB-1

LIQUEFACTION ANALYSES BASED ON SPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
5/15/19 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2002-EB-2

Ground Elevation =			18.2	ft			
Depth to Ground Water Table =			10.2	ft	= EL	8	ft
γ =	110	pcf					
γ_{sat} =	120	pcf					
Boring Diameter =	8	inch =	203.2	mm			
Rod Length Above Ground =	3	ft =	0.9	m			

Elevation	Depth	Depth	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N	$N_{1,60}$	Assumed FC	ΔN	$N_{1,60,cs}$	r_d	CSR	MSF	K_σ	$CRR_{M=7.5,1 atm}$	CRR	FS	γlim	$F\alpha$	γmax	ε_v	ΔH (ft)	ΔS (in)	ΔLDI (in)
ft	ft	m	blow/ft	psf	kPa	psf	kPa		Y/N						%		blow/ft											%	ft	in	
-16.3	34.5	10.5	37	4,038.0	193.6	2,521.7	120.9	1.00	Y	1.30	1.15	1	0.94	52	15	3	56	0.85	0.71	1.141	0.95	2.00	2.00	2.0	0.000	-2.044	0.000	0.0	5.0	0.0	0.0
-21.3	39.5	12.0	32	4,638.0	222.3	2,809.7	134.7	1.00	Y	1.30	1.15	1	0.90	43	15	3	47	0.83	0.71	1.141	0.92	2.00	2.00	2.0	0.002	-1.310	0.000	0.0	5.0	0.0	0.0
-26.3	44.5	13.6	32	5,238.0	251.1	3,097.7	148.5	1.00	Y	1.30	1.15	1	0.87	42	15	3	45	0.80	0.71	1.141	0.89	2.00	2.00	2.0	0.002	-1.195	0.000	0.0	5.0	0.0	0.0
-31.3	49.5	15.1	37	5,838.0	279.9	3,385.7	162.3	1.00	Y	1.30	1.15	1	0.86	48	15	3	51	0.77	0.69	1.141	0.86	2.00	1.96	2.0	0.000	-1.659	0.000	0.0	5.0	0.0	0.0
																											Total	0.0	0.0	0.0	0.0

PLATE D-21: BORING 2002-EB-2

LIQUEFACTION ANALYSES BASED ON SPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
5/15/19 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2002-EB-3

Ground Elevation =			19.2	ft			
Depth to Ground Water Table =			11.2	ft	= EL	8	ft
γ =	110	pcf					
γ_{sat} =	120	pcf					
Boring Diameter =	8	inch =	203.2	mm			
Rod Length Above Ground =	3	ft =	0.9	m			

Elevation	Depth	Depth	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N	$N_{1,60}$	Assumed FC	ΔN	$N_{1,60,cs}$	r_d	CSR	MSF	K_σ	$CRR_{M=7.5,1\ atm}$	CRR	FS	γlim	$F\alpha$	γmax	ε_v	ΔH (ft)	ΔS (in)	ΔLDI (in)
ft	*ft*	*m*	*blow/ft*	*psf*	*kPa*	*psf*	*kPa*		*Y/N*						*%*		*blow/ft*											*%*	*ft*	*in*	
-10.3	29.5	9.0	11	3,428.0	164.3	2,286.1	109.6	1.00	N	1.00	1.15	1	0.96	12	15	3	15	0.88	0.69	1.141	0.99	0.16	0.18	0.3	0.264	0.739	0.264	2.8	1.0	0.3	3.2
-11.8	31.0	9.4	8	3,608.0	173.0	2,372.5	113.7	1.00	Y	1.10	1.15	1	0.93	9	15	3	13	0.87	0.69	1.141	0.99	0.14	0.16	0.2	0.352	0.839	0.352	3.2	5.0	1.9	21.1
-30.3	49.5	15.1	33	5,828.0	279.4	3,438.1	164.8	1.00	N	1.00	1.15	1	0.85	32	15	3	35	0.77	0.68	1.141	0.87	1.19	1.18	1.7	0.021	-0.461	0.006	0.1	5.0	0.1	0.4
																											Total	1.8	11.0	2.3	24.7

PLATE D-22: BORING 2002-EB-3

Supplement E

Dynamic Densification Analyses

DYNAMIC DENSIFICATION ANALYSES BASED ON CPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/2020 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2019-CPT-01					
Ground Elevation =	18.0	ft			
Depth to Ground Water Table =	10.0	ft	= EL	8	ft
γ =	110	pcf			
γ_{sat} =	120	pcf			
Atmospheric pressure	2,116.2	psf			
Cone Area Ratio	0.59				

Depth	q_c	f_s	Pore Pressure	q_c	qt	f_s	Pore Pressure	σ_v	u	σ_v'	rd	τ_{ave}	Fr	n	Qtn	Ic	G0	p	a	b	R	γ	Kc	Qtn,cs	N1(60),cs	Nc	$\varepsilon_{vol(15)}$	ε_{vol}	Δs
ft	tsf	tsf	tsf	psf	psf	psf	psf	psf	psf	psf		psf					psf												in
													Hand Auger from 0 to 6 feet																
6.04	9.75	0.42	0.46	19,501.4	19,874.7	843.0	910.4	664.4	0.0	664.4	0.99	10.7	4.4	1.04	10.3	3.1	2.92E+05	442.9	0.1	16,357.3	3.68E-05	0.0040	7.7	79.2	28.2	10.8	0.0027	0.0023	0.001
6.10	9.46	0.44	0.43	18,925.0	19,279.6	880.0	864.8	671.0	0.0	671.0	0.99	10.8	4.7	1.02	11.1	3.0	3.06E+05	447.3	0.1	16,260.6	3.53E-05	0.0039	7.2	79.8	27.7	10.8	0.0026	0.0023	0.002
6.17	9.42	0.47	0.38	18,839.8	19,154.4	937.8	767.2	678.7	0.0	678.7	0.99	10.9	5.1	1.02	11.5	3.0	3.16E+05	452.5	0.1	16,149.6	3.46E-05	0.0038	7.0	80.7	27.7	10.8	0.0025	0.0022	0.002
6.23	9.55	0.48	0.35	19,098.6	19,382.2	956.6	691.8	685.3	0.0	685.3	0.99	11.1	5.1	1.03	11.5	3.0	3.26E+05	456.9	0.1	16,056.1	3.39E-05	0.0037	7.3	83.6	29.1	10.8	0.0023	0.0020	0.002
6.30	9.52	0.47	0.31	19,045.8	19,302.7	941.8	626.6	693.0	0.0	693.0	0.99	11.2	5.1	1.05	11.0	3.1	3.30E+05	462.0	0.1	15,948.8	3.39E-05	0.0037	8.0	87.3	31.5	10.8	0.0021	0.0018	0.001
6.36	9.27	0.47	0.30	18,537.4	18,786.4	939.2	607.2	699.6	0.0	699.6	0.99	11.3	5.2	1.06	10.6	3.1	3.32E+05	466.4	0.1	15,858.4	3.40E-05	0.0037	8.5	89.7	33.3	10.8	0.0020	0.0017	0.001
6.43	8.88	0.45	0.34	17,767.2	18,047.6	905.6	684.0	707.3	0.0	707.3	0.99	11.4	5.2	1.07	10.5	3.2	3.36E+05	471.5	0.1	15,754.6	3.39E-05	0.0037	8.7	90.5	33.9	10.8	0.0020	0.0017	0.001
6.49	8.62	0.42	0.36	17,241.4	17,539.5	845.2	727.0	713.9	0.0	713.9	0.98	11.5	5.0	1.05	10.9	3.1	3.43E+05	475.9	0.1	15,667.0	3.35E-05	0.0036	8.1	89.0	32.5	10.8	0.0020	0.0018	0.001
6.56	8.41	0.37	0.38	16,825.4	17,141.0	741.2	769.8	721.6	0.0	721.6	0.98	11.6	4.5	1.04	11.3	3.1	3.45E+05	481.1	0.1	15,566.5	3.37E-05	0.0036	7.6	86.0	30.5	10.8	0.0022	0.0019	0.002
6.63	8.31	0.32	0.44	16,610.0	16,966.9	640.2	870.6	729.3	0.0	729.3	0.98	11.7	3.9	1.03	11.1	3.1	3.39E+05	486.2	0.1	15,467.7	3.46E-05	0.0038	7.4	82.6	29.0	10.8	0.0024	0.0021	0.002
6.69	8.30	0.30	0.46	16,595.0	16,975.6	602.2	928.2	735.9	0.0	735.9	0.98	11.9	3.7	1.03	11.0	3.0	3.33E+05	490.6	0.1	15,384.3	3.56E-05	0.0039	7.2	79.2	27.5	10.8	0.0026	0.0023	0.002
6.76	8.52	0.30	0.51	17,030.0	17,446.6	590.2	1,016.2	743.6	0.0	743.6	0.98	12.0	3.5	1.02	10.9	3.0	3.34E+05	495.7	0.1	15,288.5	3.59E-05	0.0039	7.1	78.2	27.0	10.8	0.0027	0.0023	0.002
6.82	8.63	0.29	0.54	17,265.2	17,704.1	586.0	1,070.4	750.2	0.0	750.2	0.98	12.1	3.5	1.04	11.2	3.1	3.57E+05	500.1	0.1	15,207.7	3.39E-05	0.0037	7.7	85.7	30.5	10.8	0.0022	0.0019	0.001
6.89	9.00	0.31	0.52	17,995.2	18,422.9	625.8	1,043.2	757.9	0.0	757.9	0.98	12.2	3.5	0.99	13.5	2.9	3.88E+05	505.3	0.1	15,114.8	3.14E-05	0.0034	6.2	83.5	27.4	10.8	0.0023	0.0020	0.002
6.95	9.41	0.33	0.50	18,828.8	19,237.3	657.6	996.4	764.5	0.0	764.5	0.98	12.3	3.6	0.92	18.9	2.8	4.67E+05	509.7	0.1	15,036.3	2.63E-05	0.0028	4.5	84.9	25.0	10.8	0.0021	0.0018	0.001
7.02	9.84	0.35	0.49	19,684.4	20,085.9	700.2	979.2	772.2	0.0	772.2	0.98	12.4	3.6	0.87	24.1	2.6	5.38E+05	514.8	0.1	14,946.2	2.31E-05	0.0024	3.6	86.1	23.7	10.8	0.0020	0.0017	0.001
7.08	9.97	0.38	0.49	19,941.8	20,340.8	754.2	973.2	778.8	0.0	778.8	0.98	12.5	3.9	0.83	29.3	2.5	5.98E+05	519.2	0.1	14,870.1	2.10E-05	0.0022	2.9	85.8	22.4	10.8	0.0019	0.0016	0.001
7.15	9.80	0.40	0.54	19,603.8	20,048.1	795.2	1,083.6	786.5	0.0	786.5	0.98	12.7	4.1	0.83	30.7	2.5	6.20E+05	524.3	0.1	14,782.6	2.04E-05	0.0021	2.8	86.6	22.4	10.8	0.0019	0.0016	0.001
7.22	9.61	0.39	0.57	19,224.2	19,694.9	781.4	1,148.0	794.2	0.0	794.2	0.98	12.8	4.1	0.87	26.3	2.6	5.88E+05	529.5	0.1	14,696.4	2.17E-05	0.0023	3.4	90.0	24.6	10.8	0.0018	0.0015	0.001
7.28	9.48	0.38	0.60	18,960.6	19,455.5	760.4	1,207.0	800.8	0.0	800.8	0.98	12.9	4.1	0.91	22.2	2.7	5.55E+05	533.9	0.1	14,623.6	2.32E-05	0.0024	4.2	93.8	27.2	10.8	0.0017	0.0015	0.001
7.35	9.39	0.38	0.62	18,787.6	19,295.8	757.4	1,239.6	808.5	0.0	808.5	0.98	13.0	4.1	0.88	25.1	2.6	5.87E+05	539.0	0.1	14,539.9	2.21E-05	0.0023	3.6	91.2	25.3	10.8	0.0017	0.0015	0.001
7.41	9.18	0.35	0.67	18,367.0	18,917.1	694.8	1,341.6	815.1	0.0	815.1	0.98	13.1	3.8	0.85	27.6	2.6	6.13E+05	543.4	0.1	14,469.1	2.14E-05	0.0022	3.2	89.0	23.9	10.8	0.0018	0.0016	0.001
7.48	9.16	0.37	0.67	18,319.6	18,872.0	733.2	1,347.2	822.8	0.0	822.8	0.98	13.2	4.1	0.80	34.3	2.4	6.77E+05	548.5	0.1	14,387.7	1.95E-05	0.0020	2.5	85.8	21.5	10.8	0.0019	0.0016	0.001
7.54	9.10	0.37	0.68	18,192.6	18,747.2	745.4	1,352.8	829.4	0.0	829.4	0.98	13.3	4.2	0.76	41.2	2.3	7.48E+05	552.9	0.1	14,318.9	1.78E-05	0.0018	2.1	86.6	20.8	10.8	0.0018	0.0015	0.001
7.61	9.06	0.37	0.69	18,114.4	18,683.1	746.8	1,387.0	837.1	0.0	837.1	0.98	13.5	4.2	0.75	48.6	2.3	8.70E+05	558.1	0.1	14,239.7	1.55E-05	0.0016	2.0	98.1	23.3	10.8	0.0013	0.0011	0.001
7.68	7.03	0.37	0.74	14,052.6	14,655.6	747.0	1,470.8	844.8	0.0	844.8	0.98	13.6	5.4	0.69	61.3	2.1	9.37E+05	563.2	0.1	14,161.7	1.45E-05	0.0015	1.5	94.2	20.7	10.8	0.0014	0.0012	0.001
7.74	9.04	0.35	0.88	18,086.8	18,808.2	708.6	1,759.4	851.4	0.0	851.4	0.98	13.7	3.9	0.71	53.7	2.2	8.78E+05	567.6	0.1	14,095.8	1.56E-05	0.0016	1.7	90.7	20.5	10.8	0.0016	0.0013	0.001
7.81	8.81	0.33	0.99	17,621.8	18,430.4	650.2	1,972.2	859.1	0.0	859.1	0.98	13.8	3.7	0.76	43.2	2.3	7.95E+05	572.7	0.1	14,019.8	1.74E-05	0.0018	2.1	88.7	21.1	10.8	0.0017	0.0014	0.001
7.87	9.11	0.32	1.11	18,228.8	19,137.1	641.6	2,215.4	865.7	0.0	865.7	0.98	13.9	3.5	0.76	41.7	2.3	7.81E+05	577.1	0.1	13,955.6	1.78E-05	0.0018	2.1	87.4	21.0	10.8	0.0017	0.0015	0.001
7.94	9.41	0.33	1.15	18,812.2	19,751.1	652.6	2,290.0	873.4	0.0	873.4	0.98	14.0	3.5	0.80	41.5	2.4	8.50E+05	582.3	0.1	13,881.6	1.65E-05	0.0017	2.5	101.8	25.4	10.8	0.0013	0.0011	0.001
8.00	9.84	0.33	1.12	19,677.4	20,599.2	651.4	2,248.4	880.0	0.0	880.0	0.98	14.1	3.3	0.80	40.1	2.4	8.36E+05	586.7	0.1	13,819.1	1.69E-05	0.0017	2.5	100.5	25.2	10.8	0.0013	0.0011	0.001
8.07	11.12	0.35	0.83	22,230.0	22,913.6	690.0	1,667.2	887.7	0.0	887.7	0.98	14.3	3.1	0.83	35.7	2.5	7.98E+05	591.8	0.1	13,747.0	1.79E-05	0.0018	2.8	101.1	26.2	10.8	0.0013	0.0012	0.001
8.13	10.85	0.33	0.86	21,707.4	22,413.5	659.0	1,722.2	894.3	0.0	894.3	0.98	14.4	3.1	0.90	26.4	2.7	7.10E+05	596.2	0.1	13,686.1	2.02E-05	0.0021	4.0	106.4	30.4	10.8	0.0013	0.0011	0.001
8.20	10.28	0.32	0.92	20,555.0	21,306.1	637.4	1,832.0	902.0	0.0	902.0	0.98	14.5	3.1	0.90	25.9	2.7	6.93E+05	601.3	0.1	13,615.9	2.09E-05	0.0022	3.9	101.7	28.8	10.8	0.0014	0.0012	0.001
8.27	9.44	0.32	0.91	18,871.4	19,614.2	641.4	1,811.6	909.7	0.0	909.7	0.98	14.6	3.4	0.90	25.4	2.7	6.82E+05	606.5	0.1	13,546.6	2.14E-05	0.0022	3.9	99.2	28.1	10.8	0.0015	0.0013	0.001
8.33	8.27	0.31	0.98	16,536.4	17,341.1	629.6	1,962.8	916.3	0.0	916.3	0.98	14.7	3.8	0.83	31.8	2.5	7.34E+05	610.9	0.1	13,488.0	2.00E-05	0.0021	2.9	91.3	23.7	10.8	0.0017	0.0015	0.001
8.40	7.56	0.31	1.07	15,126.0	16,002.1	610.8	2,136.8	924.0	0.0	924.0	0.98	14.8	4.1	0.77	34.3	2.4	6.89E+05	616.0	0.1	13,420.4	2.15E-05	0.0022	2.2	75.0	18.2	10.8	0.0025	0.0022	0.002
8.46	7.26	0.29	1.09	14,522.8	15,419.1	586.6	2,186.0	930.6	0.0	930.6	0.98	14.9	4.0	0.76	36.4	2.3	7.08E+05	620.4	0.1	13,363.2	2.11E-05	0.0022	2.0	74.3	17.7	10.8	0.0025	0.0022	0.002
8.53	6.86	0.28	1.18	13,716.2	14,679.7	553.6	2,350.0	938.3	0.0	938.3	0.98	15.0	4.0	0.75	33.9	2.3	6.52E+05	625.5	0.1	13,297.3	2.31E-05	0.0024	2.0	67.1	15.8	10.8	0.0032	0.0028	0.002
8.59	6.70	0.25	1.26	13,399.0	14,435.6	496.2	2,528.4	944.9	0.0	944.9	0.98	15.2	3.7	0.76	31.0	2.3	6.18E+05	629.9	0.1	13,241.5	2.45E-05	0.0026	2.1	64.8	15.5	10.8	0.0035	0.0030	0.002
8.66	6.79	0.22	1.31	13,584.4	14,661.8	435.0	2,627.8	952.6	0.0	952.6	0.98	15.3	3.2	0.79	28.7	2.4	6.07E+05	635.1	0.1	13,177.2	2.52E-05	0.0026	2.3	66.0	16.2	10.8	0.0034	0.0029	0.002
8.72	7.43	0.22	1.41	14,852.8	16,011.3	448.2	2,825.6	959.2	0.0	959.2	0.98	15.4	3.0	0.85	25.1	2.6	6.30E+05	639.5	0.1	13,122.7	2.44E-05	0.0025	3.1	78.7	20.9	10.8	0.0024	0.0021	0.002
8.79	8.36	0.24	1.38	16,714.6	17,847.5	488.6	2,763.2	966.9	0.0	966.9	0.98	15.5	2.9	0.89	23.4	2.7	6.44E+05	644.6	0.1	13,059.9	2.41E-05	0.0025	3.7	86.5	24.1	10.8	0.0020	0.0017	0.001
8.86	8.49	0.29	1.26	16,988.8	18,023.0	583.4	2,522.4	974.6	0.0	974.6	0.98	15.6	3.4	0.90	25.5	2.7	7.39E+05	649.7	0.1	12,997.9	2.11E-05	0.0022	4.0	102.3	29.2	10.8	0.0014	0.0012	0.001
8.92	8.39	0.29	0.97	16,778.0	17,570.9	588.0	1,933.8	981.2	0.0	981.2	0.98	15.7	3.5	0.67	62.0	2.1	1.00E+06	654.1	0.1	12,945.4	1.57E-05	0.0016	1.4	89.7	19.4	10.8	0.0017	0.0014	0.001
8.99	7.23	0.29	0.68	14,450.4	15,005.3	583.6	1,353.4	988.9	0.0	988.9	0.98	15.8	4.2	0.56	92.8	1.8	1.14E+06	659.3	0.1	12,884.8	1.38E-05	0.0014	1.1	103.7	20.1	10.8	0.0014	0.0012	0.001
9.05	6.48	0.28	0.59	12,963.2	13,446.5	561.0	1,178.8	995.5	0.0	995.5	0.98	15.9	4.5	0.59	80.3	1.9	1.07E+06	663.7	0.1	12,833.5	1.49E-05	0.0015	1.2	94.8	18.9	10.8	0.0016	0.0014	0.001
9.12	5.77	0.28	0.47	11,546.2	11,934.0	569.6	945.8	1,003.2	0.0	1,003.2	0.98	16.1	5.2	0.63	67.8	2.0	1.00E+06	668.8	0.1	12,774.3	1.60E-05	0.0016	1.3	87.5	18.2	10.8	0.0018	0.0016	0.001
9.18	6.38	0.26	0.57	12,750.6	13,216.7	524.0	1,136.8	1,009.8	0.0	1,009.8	0.98	16.2	4.3	0.69	55.3	2.1	9.46E+05	673.2	0.1	12,724.1	1.71E-05	0.0018	1.5	84.2	18.5	10.8	0.0019	0.0017	0.001
9.25	6.61	0.23	0.72	13,223.0	13,816.0	460.8	1,446.4	1,017.5	0.0	1,017.5	0.98	16.3	3.6	0.72	48.8	2.2	9.04E+05	678.3	0.1	12,666.3	1.80E-05	0.0019	1.7	82.7	18.7	10.8	0.0020	0.0017	0.001
9.32	6.65	0.19	0.71	13,298.0	13,880.4	383.6	1,420.4	1,025.2	0.0	1,025.2	0.98	16.4	3.0	0.76	42.5	2.3	8.76E+05	683.5	0.1	12,609.1	1.87E-05	0.0019	2.0	84.7	20.1	10.8	0.0019	0.0017	0.001
9.38	6.50	0.17	0.62	13,001.2	13,508.9	339.8	1,238.2	1,031.8	0.0	1,031.8	0.98	16.5	2.7	0.79	38.8	2.4	8.87E+05	687.9	0.1	12,560.6	1.86E-05	0.0019	2.4	91.6	22.7	10.8	0.0017	0.0014	0.001
9.45	8.44	0.18	0.67	16,881.4	17,427.4	359.6	1,331.8	1,039.5	0.0	1,039.5	0.98	16.6	2.2	0.81	38.0	2.5	9.13E+05	693.0	0.1	12,504.7	1.82E-05	0.0019	2.6	97.2	24.5	10.8	0.0015	0.0013	0.001
9.51	13.91	0.18	0.78	27,812.6	28,449.7	358.2	1,554.0	1,046.1	0.0	1,046.1	0.98	16.7	1.3	0.80	36.7	2.4	8.69E+05	697.4	0.1	12,457.3	1.93E-05	0.0020	2.5	90.7	22.7	10.8	0.0017	0.0015	0.001
9.58	21.73	0.17	0.80	43,453.0	44,108.3	334.2	1,598.2	1,053.8	0.0	1,053.8	0.98	16.9	0.8	0.77	37.0	2.4	8.08E+05	702.5	0.1	12,402.6	2.09E-05	0.0022	2.1	78.6	18.9	10.8	0.0023	0.0020	0.002
9.64	30.54	0.14	0.58	61,077.0	61,554.5	286.4	1,164.6	1,060.4	0.0	1,060.4	0.98	17.0	0.5	0.74	40.0	2.3	8.15E+05	706.9	0.1	12,356.3	2.08E-05	0.0022	1.9	75.1	17.5	10.8	0.0025	0.0022	0.002

Depth	q_c	f_s	Pore Pressure	q_c	qt	f_s	Pore Pressure	σ_v	u	σ_v'	rd	τ_{ave}	Fr	n
ft	tsf	tsf	tsf	psf	psf	psf	psf	psf	psf	psf		psf		
9.71	34.78	0.14	0.39	69,551.4	69,869.4	279.6	775.6	1,068.1	0.0	1,068.1	0.98	17.1	0.4	0.71
9.77	36.08	0.15	0.32	72,156.2	72,422.3	291.2	649.0	1,074.7	0.0	1,074.7	0.98	17.2	0.4	0.70
9.84	37.82	0.16	0.24	75,636.2	75,829.7	328.0	472.0	1,082.4	0.0	1,082.4	0.98	17.3	0.4	0.71
9.91	38.66	0.18	0.17	77,325.4	77,461.7	362.8	332.4	1,090.1	0.0	1,090.1	0.98	17.4	0.5	0.75
9.97	39.27	0.20	0.13	78,543.6	78,649.9	391.0	259.2	1,096.7	0.0	1,096.7	0.98	17.5	0.5	0.77

Depth	Qtn	Ic	G0	p	a	b	R	γ	Kc	Qtn,cs	N1(60),cs	Nc	$\varepsilon_{vol(15)}$	ε_{vol}	Δs
ft			psf												in
9.71	45.1	2.2	8.41E+05	712.1	0.1	12,302.7	2.03E-05	0.0021	1.6	73.3	16.4	10.8	0.0027	0.0023	0.002
9.77	45.9	2.2	8.37E+05	716.5	0.1	12,257.3	2.05E-05	0.0021	1.6	72.0	15.9	10.8	0.0028	0.0024	0.002
9.84	40.9	2.2	7.81E+05	721.6	0.1	12,205.0	2.22E-05	0.0023	1.7	67.9	15.3	10.8	0.0032	0.0027	0.002
9.91	34.9	2.3	7.35E+05	726.7	0.1	12,153.2	2.37E-05	0.0025	1.9	66.9	15.7	10.8	0.0033	0.0029	0.002
9.97	30.5	2.4	6.90E+05	731.1	0.1	12,109.2	2.54E-05	0.0026	2.1	65.3	15.7	10.8	0.0035	0.0031	0.000
											Total Estimated Settlement			2 x $\Sigma\Delta s$	**0.2**

DYNAMIC DENSIFICATION ANALYSES BASED ON CPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/2020 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2019-CPT-02				
Ground Elevation =	14.1	ft		
Depth to Ground Water Table =	6.1	ft	= EL 8	ft
γ =	110	pcf		
γ_{sat} =	120	pcf		
Atmospheric pressure	2,116.2	psf		
Cone Area Ratio	0.59			

Depth	q_c	f_s	Pore Pressure	q_c	qt	f_s	Pore Pressure	σ_v	u	σ_v'	rd	τ_{ave}	Fr	n	Qtn	Ic	G0	p	a	b	R	γ	Kc	Qtn,cs	N1(60),cs	Nc	$\varepsilon_{vol(15)}$	ε_{vol}	Δs
ft	tsf	tsf	tsf	psf	psf	psf	psf	psf	psf	psf		psf					psf												in

Hand Auger from 0 to 6 feet - Ground Water Table is at 5 feet below ground surface

Total Estimated Settlement 2 x $\Sigma\Delta s$ **0.0**

DYNAMIC DENSIFICATION ANALYSES BASED ON CPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

FUGRO

04.72190021
2/20/2020 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2019-CPT-03					
Ground Elevation =	16.3	ft			
Depth to Ground Water Table =	8.3	ft	= EL	8	ft
γ =	110	pcf			
γ_{sat} =	120	pcf			
Atmospheric pressure	2,116.2	psf			
Cone Area Ratio	0.59				

Depth	q_c	f_s	Pore Pressure	q_c	qt	f_s	Pore Pressure	σ_v	u	σ_v'	rd	τ_{ave}	Fr	n	Qtn	Ic	G0	p	a	b	R	γ	Kc	Qtn,cs	N1(60),cs	Nc	$\varepsilon_{vol(15)}$	ε_{vol}	Δs
ft	tsf	tsf	tsf	psf	psf	psf	psf	psf	psf	psf		psf					psf												in
													Hand Auger from 0 to 5 feet																
5.84	15.47	0.73	0.22	30,930.0	31,112.3	1,450.2	444.6	642.4	0.0	642.4	0.99	10.4	4.8	0.88	41.0	2.7	7.92E+05	428.3	0.1	16,691.1	1.31E-05	0.0013	3.7	150.9	42.0	10.8	0.0006	0.0005	0.000
5.90	15.85	0.74	0.23	31,702.8	31,892.9	1,485.6	463.6	649.0	0.0	649.0	0.99	10.5	4.8	0.88	41.5	2.7	8.07E+05	432.7	0.1	16,589.1	1.30E-05	0.0013	3.7	151.8	42.1	10.8	0.0005	0.0005	0.000
5.97	14.51	0.75	0.02	29,012.8	29,025.9	1,507.2	32.0	656.7	0.0	656.7	0.99	10.6	5.3	0.90	38.3	2.7	7.90E+05	437.8	0.1	16,472.1	1.34E-05	0.0014	4.1	155.9	44.6	10.8	0.0005	0.0005	0.000
6.04	13.42	0.78	0.07	26,840.4	26,895.8	1,563.0	135.2	664.4	0.0	664.4	0.99	10.7	6.0	0.92	36.0	2.8	7.83E+05	442.9	0.1	16,357.3	1.37E-05	0.0014	4.5	161.6	47.6	10.8	0.0005	0.0004	0.000
6.10	11.61	0.79	-0.09	23,226.8	23,151.6	1,578.4	-183.4	671.0	0.0	671.0	0.99	10.8	7.0	0.95	31.7	2.9	7.51E+05	447.3	0.1	16,260.6	1.44E-05	0.0015	5.3	167.0	51.8	10.8	0.0005	0.0004	0.000
6.17	11.34	0.79	-0.06	22,676.8	22,626.9	1,578.6	-121.8	678.7	0.0	678.7	0.99	10.9	7.2	0.96	30.9	2.9	7.48E+05	452.5	0.1	16,149.6	1.46E-05	0.0015	5.4	167.0	52.2	10.8	0.0005	0.0004	0.000
6.23	12.02	0.77	0.07	24,038.4	24,093.8	1,542.0	135.0	685.3	0.0	685.3	0.99	11.1	6.6	0.94	32.1	2.8	7.59E+05	456.9	0.1	16,056.1	1.46E-05	0.0015	5.1	162.1	49.6	10.8	0.0005	0.0004	0.000
6.30	11.37	0.75	0.12	22,735.2	22,833.3	1,492.8	239.2	693.0	0.0	693.0	0.99	11.2	6.7	0.95	30.3	2.9	7.40E+05	462.0	0.1	15,948.8	1.51E-05	0.0016	5.3	159.9	49.6	10.8	0.0005	0.0005	0.000
6.36	11.13	0.72	0.16	22,255.2	22,383.7	1,439.4	313.4	699.6	0.0	699.6	0.99	11.3	6.6	0.96	29.5	2.9	7.28E+05	466.4	0.1	15,858.4	1.55E-05	0.0016	5.3	156.4	48.6	10.8	0.0006	0.0005	0.000
6.43	10.99	0.70	0.25	21,980.6	22,184.3	1,404.4	496.8	707.3	0.0	707.3	0.99	11.4	6.5	0.96	28.9	2.9	7.23E+05	471.5	0.1	15,754.6	1.58E-05	0.0016	5.3	153.8	47.8	10.8	0.0006	0.0005	0.000
6.49	11.04	0.71	0.22	22,079.2	22,261.2	1,415.8	444.0	713.9	0.0	713.9	0.98	11.5	6.6	0.96	28.8	2.9	7.28E+05	475.9	0.1	15,667.0	1.58E-05	0.0016	5.3	153.9	47.9	10.8	0.0006	0.0005	0.000
6.56	11.04	0.69	0.31	22,075.2	22,328.8	1,381.6	618.6	721.6	0.0	721.6	0.98	11.6	6.4	0.96	28.5	2.9	7.25E+05	481.1	0.1	15,566.5	1.60E-05	0.0017	5.3	151.0	46.9	10.8	0.0006	0.0005	0.000
6.63	11.19	0.68	0.39	22,380.0	22,696.5	1,360.0	772.0	729.3	0.0	729.3	0.98	11.7	6.2	0.95	28.6	2.8	7.27E+05	486.2	0.1	15,467.7	1.62E-05	0.0017	5.2	148.5	45.8	10.8	0.0006	0.0005	0.000
6.69	11.25	0.67	0.37	22,498.4	22,804.8	1,339.4	747.4	735.9	0.0	735.9	0.98	11.9	6.1	0.95	28.5	2.8	7.26E+05	490.6	0.1	15,384.3	1.63E-05	0.0017	5.2	146.6	45.1	10.8	0.0006	0.0005	0.000
6.76	11.10	0.66	0.41	22,193.6	22,530.6	1,329.4	822.0	743.6	0.0	743.6	0.98	12.0	6.1	0.95	27.9	2.8	7.24E+05	495.7	0.1	15,288.5	1.65E-05	0.0017	5.2	145.7	45.0	10.8	0.0006	0.0006	0.000
6.82	10.83	0.67	0.26	21,660.8	21,873.0	1,340.0	517.6	750.2	0.0	750.2	0.98	12.1	6.3	0.96	27.1	2.9	7.21E+05	500.1	0.1	15,207.7	1.68E-05	0.0017	5.4	146.6	45.9	10.8	0.0006	0.0006	0.000
6.89	10.68	0.67	0.42	21,354.6	21,698.4	1,331.0	838.6	757.9	0.0	757.9	0.98	12.2	6.4	0.96	26.6	2.9	7.20E+05	505.3	0.1	15,114.8	1.70E-05	0.0018	5.5	145.7	45.7	10.8	0.0006	0.0006	0.000
6.95	10.50	0.65	0.23	20,997.2	21,187.0	1,303.0	463.0	764.5	0.0	764.5	0.98	12.3	6.4	0.97	25.9	2.9	7.11E+05	509.7	0.1	15,036.3	1.73E-05	0.0018	5.6	144.0	45.5	10.8	0.0007	0.0006	0.000
7.02	10.46	0.64	0.19	20,912.6	21,065.8	1,289.0	373.6	772.2	0.0	772.2	0.98	12.4	6.4	0.97	25.5	2.9	7.10E+05	514.8	0.1	14,946.2	1.75E-05	0.0018	5.6	142.7	45.2	10.8	0.0007	0.0006	0.000
7.08	10.29	0.65	0.27	20,582.0	20,803.4	1,295.8	540.0	778.8	0.0	778.8	0.98	12.5	6.5	0.97	25.0	2.9	7.10E+05	519.2	0.1	14,870.1	1.77E-05	0.0018	5.7	142.9	45.5	10.8	0.0007	0.0006	0.000
7.15	10.30	0.63	0.21	20,606.4	20,781.2	1,263.2	426.4	786.5	0.0	786.5	0.98	12.7	6.3	0.97	24.7	2.9	7.06E+05	524.3	0.1	14,782.6	1.79E-05	0.0019	5.7	140.3	44.6	10.8	0.0007	0.0006	0.001
7.22	10.52	0.63	0.22	21,042.0	21,221.9	1,267.2	438.8	794.2	0.0	794.2	0.98	12.8	6.2	0.97	25.0	2.9	7.13E+05	529.5	0.1	14,696.4	1.79E-05	0.0019	5.6	139.6	44.2	10.8	0.0007	0.0006	0.000
7.28	10.42	0.63	0.43	20,842.4	21,192.7	1,256.4	854.4	800.8	0.0	800.8	0.98	12.9	6.2	0.97	24.7	2.9	7.13E+05	533.9	0.1	14,623.6	1.81E-05	0.0019	5.6	138.5	43.8	10.8	0.0007	0.0006	0.000
7.35	10.32	0.62	0.32	20,631.6	20,895.5	1,245.8	643.6	808.5	0.0	808.5	0.98	13.0	6.2	0.97	24.2	2.9	7.10E+05	539.0	0.1	14,539.9	1.83E-05	0.0019	5.7	137.6	43.8	10.8	0.0007	0.0006	0.000
7.41	9.85	0.62	0.41	19,705.8	20,040.3	1,235.0	815.8	815.1	0.0	815.1	0.98	13.1	6.4	0.98	23.2	2.9	7.00E+05	543.4	0.1	14,469.1	1.87E-05	0.0019	5.9	137.4	44.3	10.8	0.0007	0.0006	0.001
7.48	9.65	0.62	0.32	19,300.0	19,566.4	1,239.2	649.8	822.8	0.0	822.8	0.98	13.2	6.6	0.99	22.6	2.9	6.98E+05	548.5	0.1	14,387.7	1.89E-05	0.0020	6.1	137.6	44.9	10.8	0.0007	0.0006	0.001
7.54	9.76	0.60	0.41	19,514.4	19,853.3	1,203.6	826.6	829.4	0.0	829.4	0.98	13.3	6.3	0.98	22.6	2.9	6.96E+05	552.9	0.1	14,318.9	1.91E-05	0.0020	6.0	134.7	43.5	10.8	0.0008	0.0007	0.001
7.61	9.86	0.61	0.16	19,728.8	19,856.1	1,226.4	310.4	837.1	0.0	837.1	0.98	13.5	6.4	0.99	22.5	2.9	7.03E+05	558.1	0.1	14,239.7	1.91E-05	0.0020	6.0	135.6	44.0	10.8	0.0008	0.0007	0.001
7.68	9.97	0.65	0.49	19,943.2	20,348.0	1,308.0	987.2	844.8	0.0	844.8	0.98	13.6	6.7	0.99	22.9	2.9	7.26E+05	563.2	0.1	14,161.7	1.87E-05	0.0019	6.1	139.5	45.5	10.8	0.0007	0.0006	0.000
7.74	10.54	0.71	0.48	21,086.4	21,477.3	1,411.0	953.4	851.4	0.0	851.4	0.98	13.7	6.8	0.99	23.9	2.9	7.60E+05	567.6	0.1	14,095.8	1.80E-05	0.0019	6.0	144.0	46.7	10.8	0.0007	0.0006	0.000
7.81	11.13	0.74	0.43	22,251.4	22,603.4	1,482.4	858.6	859.1	0.0	859.1	0.98	13.8	6.8	0.98	24.9	2.9	7.88E+05	572.7	0.1	14,019.8	1.75E-05	0.0018	5.9	146.5	47.1	10.8	0.0006	0.0006	0.000
7.87	12.28	0.76	0.5	24,550.6	24,924.2	1,519.6	911.2	865.7	0.0	865.7	0.98	13.9	6.3	0.96	26.9	2.9	8.21E+05	577.1	0.1	13,955.6	1.69E-05	0.0017	5.4	145.9	45.7	10.8	0.0006	0.0006	0.000
7.94	13.90	0.77	0.57	27,809.2	28,278.5	1,545.2	1,144.6	873.4	0.0	873.4	0.98	14.0	5.6	0.94	29.7	2.8	8.61E+05	582.3	0.1	13,881.6	1.63E-05	0.0017	4.8	143.7	43.4	10.8	0.0007	0.0006	0.000
8.00	15.32	0.81	0.78	30,635.0	31,271.2	1,628.8	1,551.8	880.0	0.0	880.0	0.98	14.1	5.4	0.92	32.3	2.8	9.07E+05	586.7	0.1	13,819.1	1.56E-05	0.0016	4.5	145.1	42.8	10.8	0.0006	0.0006	0.000
																									Total Estimated Settlement			2 x $\Sigma\Delta$s	**0.0**

DYNAMIC DENSIFICATION ANALYSES BASED ON CPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/2020 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2020-CPT-04

Ground Elevation =	19.2	ft			
Water Table Depth from ground surface	11.2	ft	= EL	8	ft
γ =	110	pcf			
γ_{sat} =	120	pcf			
Atmospheric pressure	2,116.2	psf			
Cone Area Ratio	0.8				

Hand Auger from 0 to 5 feet

Depth	q_c	f_s	Pore Pressure	q_c	qt	f_s	Pore Pressure	σ_v	u	σ_v'	rd	τ_{ave}	Fr	n	Qtn
ft	tsf	tsf	tsf	psf	psf	psf	psf	psf	psf	psf		psf			
5.09	153.74	4.04	0.10	307,472.0	307,511.9	8,084.0	199.7	559.4	0.0	559.4	0.99	9.0	2.6	1.04	513.9
5.25	68.61	2.91	0.18	137,216.0	137,287.9	5,812.0	359.6	577.4	0.0	577.4	0.99	9.3	4.3	1.02	11.1
5.41	53.82	1.96	0.62	107,634.0	107,883.0	3,922.0	1,245.0	595.5	0.0	595.5	0.99	9.6	3.7	1.02	11.5
5.58	70.64	1.86	0.40	141,284.0	141,442.5	3,722.0	792.3	613.5	0.0	613.5	0.99	9.9	2.6	1.03	11.5
5.74	83.60	1.68	1.31	167,190.0	167,715.9	3,368.0	2,629.7	631.6	0.0	631.6	0.99	10.2	2.0	1.05	11.0
5.91	97.58	1.98	1.93	195,156.0	195,926.9	3,954.0	3,854.7	649.6	0.0	649.6	0.99	10.5	2.0	1.06	10.6
6.07	116.10	2.78	1.01	232,204.0	232,606.1	5,554.0	2,010.7	667.7	0.0	667.7	0.99	10.8	2.4	1.07	10.5
6.23	115.46	2.77	1.46	230,924.0	231,509.9	5,548.0	2,929.4	685.7	0.0	685.7	0.99	11.1	2.4	1.05	10.9
6.40	98.72	3.19	1.73	197,442.0	198,133.1	6,378.0	3,455.3	703.7	0.0	703.7	0.99	11.3	3.2	1.04	11.3
6.56	116.88	2.48	1.43	233,766.0	234,336.6	4,956.0	2,852.8	721.8	0.0	721.8	0.98	11.6	2.1	1.03	11.1
6.73	149.89	2.29	1.27	299,782.0	300,288.6	4,570.0	2,533.2	739.8	0.0	739.8	0.98	11.9	1.5	1.03	11.0
6.89	119.92	2.50	0.81	239,838.0	240,162.9	5,004.0	1,624.5	757.9	0.0	757.9	0.98	12.2	2.1	1.02	10.9
7.05	90.70	1.74	0.68	181,396.0	181,667.0	3,484.0	1,354.9	775.9	0.0	775.9	0.98	12.5	1.9	1.04	11.2
7.22	140.06	2.02	1.87	280,116.0	280,865.7	4,042.0	3,748.3	794.0	0.0	794.0	0.98	12.8	1.4	0.99	13.5
7.38	262.07	1.75	1.52	524,132.0	524,739.2	3,492.0	3,036.0	812.0	0.0	812.0	0.98	13.1	0.7	0.92	18.9
7.55	194.71	1.99	0.83	389,422.0	389,753.5	3,988.0	1,657.7	830.1	0.0	830.1	0.98	13.3	1.0	0.87	24.1
7.71	127.33	2.48	0.57	254,656.0	254,883.7	4,968.0	1,138.5	848.1	0.0	848.1	0.98	13.6	2.0	0.83	29.3
7.87	63.85	2.33	0.48	127,690.0	127,883.1	4,664.0	965.4	866.1	0.0	866.1	0.98	13.9	3.7	0.83	30.7
8.04	45.35	1.99	1.57	90,698.0	91,327.2	3,978.0	3,145.8	884.2	0.0	884.2	0.98	14.2	4.4	0.87	26.3
8.20	72.17	1.74	5.35	144,348.0	146,487.1	3,476.0	10,695.6	902.2	0.0	902.2	0.98	14.5	2.4	0.91	22.2
8.37	190.78	1.58	3.87	381,566.0	383,112.6	3,168.0	7,732.9	920.3	0.0	920.3	0.98	14.8	0.8	0.88	25.1
8.53	217.78	1.66	1.97	435,552.0	436,338.3	3,318.0	3,931.3	938.3	0.0	938.3	0.98	15.0	0.8	0.85	27.6
8.69	194.85	1.52	1.17	389,700.0	390,168.0	3,048.0	2,340.1	956.4	0.0	956.4	0.98	15.3	0.8	0.80	34.3
8.86	159.11	1.32	0.76	318,224.0	318,528.2	2,634.0	1,521.2	974.4	0.0	974.4	0.98	15.6	0.8	0.76	41.2
9.02	123.48	1.03	0.51	246,968.0	247,171.7	2,052.0	1,018.7	992.5	0.0	992.5	0.98	15.9	0.8	0.75	48.6
9.19	88.89	0.99	0.37	177,774.0	177,923.8	1,970.0	748.9	1,010.5	0.0	1,010.5	0.98	16.2	1.1	0.69	61.3
9.35	57.52	1.12	0.11	115,044.0	115,087.3	2,248.0	216.4	1,028.5	0.0	1,028.5	0.98	16.5	2.0	0.71	53.7
9.51	48.55	1.34	0.11	97,106.0	97,148.6	2,674.0	213.1	1,046.6	0.0	1,046.6	0.98	16.7	2.8	0.76	43.2
9.68	62.20	0.78	0.45	124,404.0	124,582.4	1,558.0	892.1	1,064.6	0.0	1,064.6	0.98	17.0	1.3	0.76	41.7
9.84	137.80	0.66	0.94	275,604.0	275,979.5	1,310.0	1,877.5	1,082.7	0.0	1,082.7	0.98	17.3	0.5	0.80	41.5
10.01	129.81	0.97	0.29	259,614.0	259,731.8	1,942.0	589.2	1,100.7	0.0	1,100.7	0.98	17.6	0.8	0.80	40.1
10.17	100.17	0.59	0.00	200,338.0	200,336.0	1,180.0	-9.9	1,118.8	0.0	1,118.8	0.98	17.9	0.6	0.83	35.7
10.33	113.29	0.97	0.13	226,578.0	226,630.6	1,940.0	262.9	1,136.8	0.0	1,136.8	0.98	18.2	0.9	0.90	26.4
10.50	76.72	0.73	0.12	153,430.0	153,477.9	1,450.0	239.6	1,154.9	0.0	1,154.9	0.98	18.4	1.0	0.90	25.9
10.66	53.37	0.64	-0.05	106,742.0	106,722.7	1,286.0	-96.5	1,172.9	0.0	1,172.9	0.98	18.7	1.2	0.90	25.4
10.83	64.01	1.14	-0.17	128,024.0	127,956.8	2,278.0	-336.2	1,190.9	0.0	1,190.9	0.97	19.0	1.8	0.83	31.8
10.99	41.70	1.20	-0.02	83,400.0	83,391.3	2,406.0	-43.3	1,209.0	0.0	1,209.0	0.97	19.3	2.9	0.77	34.3
11.15	28.75	0.89	0.02	57,494.0	57,502.0	1,784.0	40.0	1,227.0	0.0	1,227.0	0.97	19.6	3.2	0.76	36.4

Depth	Ic	G0	p	a	b	R	γ	Kc	Qtn,cs	N1(60),cs	Nc	$\varepsilon_{vol(15)}$	ε_{vol}	Δs
ft		psf												in
5.09	3.1	2.92E+05	372.9	0.1	18,136.1	3.68E-05	0.0041	7.7	79.2	28.2	10.8	0.0027	0.0023	0.001
5.25	3.0	3.06E+05	385.0	0.1	17,793.9	3.53E-05	0.0039	7.2	79.8	27.7	10.8	0.0026	0.0023	0.002
5.41	3.0	3.16E+05	397.0	0.1	17,468.3	3.46E-05	0.0038	7.0	80.7	27.7	10.8	0.0025	0.0022	0.002
5.58	3.0	3.26E+05	409.0	0.1	17,158.2	3.39E-05	0.0037	7.3	83.6	29.1	10.8	0.0023	0.0020	0.002
5.74	3.1	3.30E+05	421.0	0.1	16,862.4	3.39E-05	0.0037	8.0	87.3	31.5	10.8	0.0021	0.0018	0.001
5.91	3.1	3.32E+05	433.1	0.1	16,579.8	3.40E-05	0.0037	8.5	89.7	33.3	10.8	0.0020	0.0017	0.001
6.07	3.2	3.36E+05	445.1	0.1	16,309.4	3.39E-05	0.0037	8.7	90.5	33.9	10.8	0.0020	0.0017	0.001
6.23	3.1	3.43E+05	457.1	0.1	16,050.6	3.35E-05	0.0036	8.1	89.0	32.5	10.8	0.0020	0.0018	0.001
6.40	3.1	3.45E+05	469.2	0.1	15,802.3	3.37E-05	0.0036	7.6	86.0	30.5	10.8	0.0022	0.0019	0.002
6.56	3.1	3.39E+05	481.2	0.1	15,564.1	3.46E-05	0.0038	7.4	82.6	29.0	10.8	0.0024	0.0021	0.002
6.73	3.0	3.33E+05	493.2	0.1	15,335.2	3.56E-05	0.0039	7.2	79.2	27.5	10.8	0.0026	0.0023	0.002
6.89	3.0	3.34E+05	505.2	0.1	15,115.1	3.59E-05	0.0039	7.1	78.2	27.0	10.8	0.0027	0.0023	0.002
7.05	3.1	3.57E+05	517.3	0.1	14,903.2	3.39E-05	0.0037	7.7	85.7	30.5	10.8	0.0022	0.0019	0.001
7.22	2.9	3.88E+05	529.3	0.1	14,699.0	3.14E-05	0.0034	6.2	83.5	27.4	10.8	0.0023	0.0020	0.002
7.38	2.8	4.67E+05	541.3	0.1	14,502.2	2.63E-05	0.0028	4.5	84.9	25.0	10.8	0.0021	0.0018	0.001
7.55	2.6	5.38E+05	553.4	0.1	14,312.2	2.31E-05	0.0024	3.6	86.1	23.7	10.8	0.0020	0.0017	0.001
7.71	2.5	5.98E+05	565.4	0.1	14,128.7	2.10E-05	0.0022	2.9	85.8	22.4	10.8	0.0019	0.0016	0.001
7.87	2.5	6.20E+05	577.4	0.1	13,951.3	2.04E-05	0.0021	2.8	86.6	22.4	10.8	0.0019	0.0016	0.001
8.04	2.6	5.88E+05	589.5	0.1	13,779.8	2.17E-05	0.0023	3.4	90.0	24.6	10.8	0.0018	0.0015	0.001
8.20	2.7	5.55E+05	601.5	0.1	13,613.8	2.32E-05	0.0024	4.2	93.8	27.2	10.8	0.0017	0.0015	0.001
8.37	2.6	5.87E+05	613.5	0.1	13,453.0	2.21E-05	0.0023	3.6	91.2	25.3	10.8	0.0017	0.0015	0.001
8.53	2.6	6.13E+05	625.5	0.1	13,297.1	2.14E-05	0.0022	3.2	89.0	23.9	10.8	0.0018	0.0016	0.001
8.69	2.4	6.77E+05	637.6	0.1	13,146.0	1.95E-05	0.0020	2.5	85.8	21.5	10.8	0.0019	0.0016	0.001
8.86	2.3	7.48E+05	649.6	0.1	12,999.4	1.78E-05	0.0018	2.1	86.6	20.8	10.8	0.0018	0.0015	0.001
9.02	2.3	8.70E+05	661.6	0.1	12,857.1	1.55E-05	0.0016	2.0	98.1	23.3	10.8	0.0013	0.0011	0.001
9.19	2.1	9.37E+05	673.7	0.1	12,718.8	1.45E-05	0.0015	1.5	94.2	20.7	10.8	0.0014	0.0012	0.001
9.35	2.2	8.78E+05	685.7	0.1	12,584.5	1.56E-05	0.0016	1.7	90.7	20.5	10.8	0.0016	0.0013	0.001
9.51	2.3	7.95E+05	697.7	0.1	12,453.8	1.74E-05	0.0018	2.1	88.7	21.1	10.8	0.0017	0.0014	0.001
9.68	2.3	7.81E+05	709.8	0.1	12,326.8	1.78E-05	0.0018	2.1	87.4	21.0	10.8	0.0017	0.0015	0.001
9.84	2.4	8.50E+05	721.8	0.1	12,203.1	1.65E-05	0.0017	2.5	101.8	25.4	10.8	0.0013	0.0011	0.001
10.01	2.4	8.36E+05	733.8	0.1	12,082.7	1.69E-05	0.0017	2.5	100.5	25.2	10.8	0.0013	0.0011	0.001
10.17	2.5	7.98E+05	745.8	0.1	11,965.3	1.79E-05	0.0018	2.8	101.1	26.2	10.8	0.0013	0.0012	0.001
10.33	2.7	7.10E+05	757.9	0.1	11,851.0	2.02E-05	0.0021	4.0	106.4	30.4	10.8	0.0013	0.0011	0.001
10.50	2.7	6.93E+05	769.9	0.1	11,739.6	2.09E-05	0.0022	3.9	101.7	28.8	10.8	0.0014	0.0012	0.001
10.66	2.7	6.82E+05	781.9	0.1	11,630.9	2.14E-05	0.0022	3.9	99.2	28.1	10.8	0.0015	0.0013	0.001
10.83	2.5	7.34E+05	794.0	0.1	11,524.8	2.00E-05	0.0021	2.9	91.3	23.7	10.8	0.0017	0.0015	0.001
10.99	2.4	6.89E+05	806.0	0.1	11,421.3	2.15E-05	0.0022	2.2	75.0	18.2	10.8	0.0025	0.0022	0.002
11.15	2.3	7.08E+05	818.0	0.1	11,320.2	2.11E-05	0.0022	2.0	74.3	17.7	10.8	0.0025	0.0022	0.000
											Total Estimated Settlement		2 x $\Sigma\Delta$s	**0.1**

DYNAMIC DENSIFICATION ANALYSES BASED ON CPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/2020 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2020-CPT-05					
Ground Elevation =	17.1	ft			
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γ_{sat} =	120	pcf			
Atmospheric pressure	2,116.2	psf			
Cone Area Ratio	0.8				

Depth	q_c	f_s	Pore Pressure	q_c	qt	f_s	Pore Pressure	σ_v	u	σ_v'	rd	τ_{ave}	Fr	n	Qtn	Ic	G0	p	a	b	R	γ	Kc	Qtn,cs	N1(60),cs	Nc	$\varepsilon_{vol(15)}$	ε_{vol}	Δs
ft	tsf	tsf	tsf	psf	psf	psf	psf	psf	psf	psf		psf					psf												in
														Hand Auger from 0 to 5 feet															
5.09	282.09	0.50	0.20	564,188.0	564,269.2	1,000.0	406.1	559.4	0.0	559.4	0.99	9.0	0.2	1.04	10.3	3.1	2.92E+05	372.9	0.1	18,136.1	3.68E-05	0.0040	7.7	79.2	28.2	10.8	0.0027	0.0023	0.001
5.25	289.53	1.63	2.63	579,064.0	580,116.6	3,266.0	5,262.9	577.4	0.0	577.4	0.99	9.3	0.6	1.02	11.1	3.0	3.06E+05	385.0	0.1	17,793.9	3.53E-05	0.0039	7.2	79.8	27.7	10.8	0.0026	0.0023	0.002
5.41	243.79	1.24	1.54	487,584.0	488,199.2	2,484.0	3,075.8	595.5	0.0	595.5	0.99	9.6	0.5	1.02	11.5	3.0	3.16E+05	397.0	0.1	17,468.3	3.46E-05	0.0038	7.0	80.7	27.7	10.8	0.0025	0.0022	0.002
5.58	152.12	1.67	0.24	304,240.0	304,335.9	3,330.0	479.4	613.5	0.0	613.5	0.99	9.9	1.1	1.03	11.5	3.0	3.26E+05	409.0	0.1	17,158.2	3.39E-05	0.0037	7.3	83.6	29.1	10.8	0.0023	0.0020	0.002
5.74	122.82	1.23	0.39	245,632.0	245,788.4	2,452.0	782.2	631.6	0.0	631.6	0.99	10.2	1.0	1.05	11.0	3.1	3.30E+05	421.0	0.1	16,862.4	3.39E-05	0.0037	8.0	87.3	31.5	10.8	0.0021	0.0018	0.001
5.91	90.84	0.96	0.71	181,674.0	181,959.6	1,920.0	1,428.0	649.6	0.0	649.6	0.99	10.5	1.1	1.06	10.6	3.1	3.32E+05	433.1	0.1	16,579.8	3.40E-05	0.0037	8.5	89.7	33.3	10.8	0.0020	0.0017	0.001
6.07	53.54	0.96	0.40	107,078.0	107,237.8	1,916.0	798.9	667.7	0.0	667.7	0.99	10.8	1.8	1.07	10.5	3.2	3.36E+05	445.1	0.1	16,309.4	3.39E-05	0.0037	8.7	90.5	33.9	10.8	0.0020	0.0017	0.001
6.23	29.72	0.75	0.23	59,444.0	59,537.2	1,494.0	466.0	685.7	0.0	685.7	0.99	11.1	2.5	1.05	10.9	3.1	3.43E+05	457.1	0.1	16,050.6	3.35E-05	0.0036	8.1	89.0	32.5	10.8	0.0020	0.0018	0.001
6.40	24.07	0.62	0.12	48,134.0	48,183.9	1,240.0	249.7	703.7	0.0	703.7	0.99	11.3	2.6	1.04	11.3	3.1	3.45E+05	469.2	0.1	15,802.3	3.37E-05	0.0036	7.6	86.0	30.5	10.8	0.0022	0.0019	0.002
6.56	29.50	0.39	0.09	58,998.0	59,033.3	774.0	176.4	721.8	0.0	721.8	0.98	11.6	1.3	1.03	11.1	3.1	3.39E+05	481.2	0.1	15,564.1	3.46E-05	0.0038	7.4	82.6	29.0	10.8	0.0024	0.0021	0.002
6.73	31.14	0.25	0.07	62,284.0	62,311.3	500.0	136.5	739.8	0.0	739.8	0.98	11.9	0.8	1.03	11.0	3.0	3.33E+05	493.2	0.1	15,335.2	3.56E-05	0.0039	7.2	79.2	27.5	10.8	0.0026	0.0023	0.002
6.89	25.82	0.22	0.03	51,644.0	51,658.0	432.0	69.8	757.9	0.0	757.9	0.98	12.2	0.8	1.02	10.9	3.0	3.34E+05	505.2	0.1	15,115.1	3.59E-05	0.0039	7.1	78.2	27.0	10.8	0.0027	0.0023	0.002
7.05	20.50	0.18	0.01	41,004.0	41,008.7	352.0	23.3	775.9	0.0	775.9	0.98	12.5	0.9	1.04	11.2	3.1	3.57E+05	517.3	0.1	14,903.2	3.39E-05	0.0037	7.7	85.7	30.5	10.8	0.0022	0.0019	0.001
7.22	20.36	0.29	0.04	40,726.0	40,742.6	576.0	83.2	794.0	0.0	794.0	0.98	12.8	1.4	0.99	13.5	2.9	3.88E+05	529.3	0.1	14,699.0	3.14E-05	0.0034	6.2	83.5	27.4	10.8	0.0023	0.0020	0.002
7.38	29.86	0.51	0.06	59,722.0	59,744.6	1,016.0	113.2	812.0	0.0	812.0	0.98	13.1	1.7	0.92	18.9	2.8	4.67E+05	541.3	0.1	14,502.2	2.63E-05	0.0028	4.5	84.9	25.0	10.8	0.0021	0.0018	0.001
7.55	36.38	1.11	0.01	72,760.0	72,766.0	2,220.0	30.0	830.1	0.0	830.1	0.98	13.3	3.1	0.87	24.1	2.6	5.38E+05	553.4	0.1	14,312.2	2.31E-05	0.0024	3.6	86.1	23.7	10.8	0.0020	0.0017	0.001
7.71	60.59	0.99	-0.01	121,172.0	121,168.7	1,986.0	-16.7	848.1	0.0	848.1	0.98	13.6	1.7	0.83	29.3	2.5	5.98E+05	565.4	0.1	14,128.7	2.10E-05	0.0022	2.9	85.8	22.4	10.8	0.0019	0.0016	0.001
7.87	13.98	0.73	-0.02	27,966.0	27,956.7	1,450.0	-46.7	866.1	0.0	866.1	0.98	13.9	5.4	0.83	30.7	2.5	6.20E+05	577.4	0.1	13,951.3	2.04E-05	0.0021	2.8	86.6	22.4	10.8	0.0019	0.0016	0.001
8.04	30.17	0.69	0.09	60,336.0	60,373.9	1,374.0	189.6	884.2	0.0	884.2	0.98	14.2	2.3	0.87	26.3	2.6	5.88E+05	589.5	0.1	13,779.8	2.17E-05	0.0023	3.4	90.0	24.6	10.8	0.0018	0.0015	0.001
8.20	27.35	0.44	0.01	54,708.0	54,713.3	886.0	26.6	902.2	0.0	902.2	0.98	14.5	1.6	0.91	22.2	2.7	5.55E+05	601.5	0.1	13,613.8	2.32E-05	0.0024	4.2	93.8	27.2	10.8	0.0017	0.0015	0.001
8.37	20.98	0.51	0.04	41,950.0	41,966.0	1,018.0	79.9	920.3	0.0	920.3	0.98	14.8	2.5	0.88	25.1	2.6	5.87E+05	613.5	0.1	13,453.0	2.21E-05	0.0023	3.6	91.2	25.3	10.8	0.0017	0.0015	0.001
8.53	15.91	0.50	0.07	31,812.0	31,838.6	1,006.0	133.1	938.3	0.0	938.3	0.98	15.0	3.3	0.85	27.6	2.6	6.13E+05	625.5	0.1	13,297.1	2.14E-05	0.0022	3.2	89.0	23.9	10.8	0.0018	0.0016	0.001
8.69	15.74	0.52	0.26	31,476.0	31,578.5	1,040.0	512.6	956.4	0.0	956.4	0.98	15.3	3.4	0.80	34.3	2.4	6.77E+05	637.6	0.1	13,146.0	1.95E-05	0.0020	2.5	85.8	21.5	10.8	0.0019	0.0016	0.001
8.86	24.32	0.63	0.25	48,636.0	48,735.2	1,250.0	496.1	974.4	0.0	974.4	0.98	15.6	2.6	0.76	41.2	2.3	7.48E+05	649.6	0.1	12,999.4	1.78E-05	0.0018	2.1	86.6	20.8	10.8	0.0018	0.0015	0.001
9.02	30.67	0.56	0.04	61,338.0	61,355.3	1,110.0	86.5	992.5	0.0	992.5	0.98	15.9	1.8	0.75	48.6	2.3	8.70E+05	661.6	0.1	12,857.1	1.55E-05	0.0016	2.0	98.1	23.3	10.8	0.0013	0.0011	0.000
																										Total Estimated Settlement		2 x $\Sigma\Delta s$	**0.1**

DYNAMIC DENSIFICATION ANALYSES BASED ON CPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/2020 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2020-CPT-06					
Ground Elevation =	13.1	ft			
Water Table Depth from ground surface	5.1	ft	= EL	8	ft
γ =	110	pcf			
γ_{sat} =	120	pcf			
Atmospheric pressure	2,116.2	psf			
Cone Area Ratio	0.8				

Depth	q_c	f_s	Pore Pressure	q_c	qt	f_s	Pore Pressure	σ_v	u	σ_v'	rd	τ_{ave}	Fr	n	Qtn	Ic	G0	p	a	b	R	γ	Kc	Qtn,cs	N1(60),cs	Nc	$\varepsilon_{vol(15)}$	ε_{vol}	Δs
ft	tsf	tsf	tsf	psf	psf	psf	psf	psf	psf	psf		psf					psf												in
													Hand Auger from 0 to 5 feet																
5.09	107.30	0.75	0.41	214,600.0	214,765.1	1,506.0	825.6	559.4	0.0	559.4	0.99	9.0	0.7	1.04	10.3	3.1	2.92E+05	372.9	0.1	18,136.1	3.68E-05	0.0040	7.7	79.2	28.2	10.8	0.0027	0.0023	0.001
5.25	110.92	0.68	0.36	221,842.0	221,985.1	1,352.0	715.7	578.9	9.3	569.6	0.99	9.4	0.6	1.02	11.1	3.0	3.06E+05	385.9	0.1	17,766.3	3.53E-05	0.0039	7.2	79.8	27.7	10.8	0.0026	0.0023	0.000
																									Total Estimated Settlement			2 x $\Sigma\Delta$s	**0.1**

DYNAMIC DENSIFICATION ANALYSES BASED ON CPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

FUGRO

04.72190021
2/20/2020 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2020-CPT-07

Ground Elevation =	18.0	ft			
Water Table Depth from ground surface	10.0	ft	= EL	8	ft
γ =	110	pcf			
γ_{sat} =	120	pcf			
Atmospheric pressure	2,116.2	psf			
Cone Area Ratio	0.8				

Depth	q_c	f_s	Pore Pressure	q_c	qt	f_s	Pore Pressure	σ_v	u	σ_v'	rd	τ_{ave}	Fr	n	Qtn	Ic	G0	p	a	b	R	γ	Kc	Qtn,cs	N1(60),cs	Nc	$\varepsilon_{vol(15)}$	ε_{vol}	Δs
ft	tsf	tsf	tsf	psf	psf	psf	psf	psf	psf	psf		psf					psf												in
													Hand Auger from 0 to 5 feet																
5.09	53.37	0.62	0.23	106,742.0	106,832.5	1,246.0	452.7	559.4	0.0	559.4	0.99	9.0	1.2	1.04	10.3	3.1	2.92E+05	372.9	0.1	18,136.1	3.68E-05	0.0040	7.7	79.2	28.2	10.8	0.0027	0.0023	0.001
5.25	47.10	1.46	-0.12	94,208.0	94,158.7	2,924.0	-246.4	577.4	0.0	577.4	0.99	9.3	3.1	1.02	11.1	3.0	3.06E+05	385.0	0.1	17,793.9	3.53E-05	0.0039	7.2	79.8	27.7	10.8	0.0026	0.0023	0.002
5.41	67.27	1.00	-0.14	134,542.0	134,484.7	1,996.0	-286.3	595.5	0.0	595.5	0.99	9.6	1.5	1.02	11.5	3.0	3.16E+05	397.0	0.1	17,468.3	3.46E-05	0.0038	7.0	80.7	27.7	10.8	0.0025	0.0022	0.002
5.58	89.19	1.27	0.03	178,388.0	178,400.0	2,544.0	59.9	613.5	0.0	613.5	0.99	9.9	1.4	1.03	11.5	3.0	3.26E+05	409.0	0.1	17,158.2	3.39E-05	0.0037	7.3	83.6	29.1	10.8	0.0023	0.0020	0.002
5.74	83.04	0.52	0.04	166,076.0	166,094.0	1,040.0	89.9	631.6	0.0	631.6	0.99	10.2	0.6	1.05	11.0	3.1	3.30E+05	421.0	0.1	16,862.4	3.39E-05	0.0037	8.0	87.3	31.5	10.8	0.0021	0.0018	0.001
5.91	65.68	1.18	0.23	131,368.0	131,461.9	2,364.0	469.3	649.6	0.0	649.6	0.99	10.5	1.8	1.06	10.6	3.1	3.32E+05	433.1	0.1	16,579.8	3.40E-05	0.0037	8.5	89.7	33.3	10.8	0.0020	0.0017	0.001
6.07	114.29	0.65	0.34	228,584.0	228,721.8	1,290.0	689.0	667.7	0.0	667.7	0.99	10.8	0.6	1.07	10.5	3.2	3.36E+05	445.1	0.1	16,309.4	3.39E-05	0.0037	8.7	90.5	33.9	10.8	0.0020	0.0017	0.001
6.23	39.92	0.86	-0.07	79,834.0	79,806.0	1,716.0	-139.8	685.7	0.0	685.7	0.99	11.1	2.2	1.05	10.9	3.1	3.43E+05	457.1	0.1	16,050.6	3.35E-05	0.0036	8.1	89.0	32.5	10.8	0.0020	0.0018	0.001
6.40	36.27	0.61	-0.06	72,536.0	72,513.4	1,218.0	-113.2	703.7	0.0	703.7	0.99	11.3	1.7	1.04	11.3	3.1	3.45E+05	469.2	0.1	15,802.3	3.37E-05	0.0036	7.6	86.0	30.5	10.8	0.0022	0.0019	0.002
6.56	146.91	0.82	0.03	293,822.0	293,834.0	1,648.0	59.9	721.8	0.0	721.8	0.98	11.6	0.6	1.03	11.1	3.1	3.39E+05	481.2	0.1	15,564.1	3.46E-05	0.0038	7.4	82.6	29.0	10.8	0.0024	0.0021	0.002
6.73	446.81	1.28	1.50	893,610.0	894,209.2	2,554.0	2,995.9	739.8	0.0	739.8	0.98	11.9	0.3	1.03	11.0	3.0	3.33E+05	493.2	0.1	15,335.2	3.56E-05	0.0039	7.2	79.2	27.5	10.8	0.0026	0.0023	0.002
6.89	471.79	1.31	1.08	943,582.0	944,012.1	2,622.0	2,150.5	757.9	0.0	757.9	0.98	12.2	0.3	1.02	10.9	3.0	3.34E+05	505.2	0.1	15,115.1	3.59E-05	0.0039	7.1	78.2	27.0	10.8	0.0027	0.0023	0.002
7.05	419.87	1.99	0.33	839,736.0	839,869.8	3,970.0	669.2	775.9	0.0	775.9	0.98	12.5	0.5	1.04	11.2	3.1	3.57E+05	517.3	0.1	14,903.2	3.39E-05	0.0037	7.7	85.7	30.5	10.8	0.0022	0.0019	0.001
7.22	326.25	3.58	0.40	652,492.0	652,651.1	7,166.0	795.6	794.0	0.0	794.0	0.98	12.8	1.1	0.99	13.5	2.9	3.88E+05	529.3	0.1	14,699.0	3.14E-05	0.0034	6.2	83.5	27.4	10.8	0.0023	0.0020	0.002
7.38	251.04	3.79	1.33	502,070.0	502,600.6	7,586.0	2,653.1	812.0	0.0	812.0	0.98	13.1	1.5	0.92	18.9	2.8	4.67E+05	541.3	0.1	14,502.2	2.63E-05	0.0028	4.5	84.9	25.0	10.8	0.0021	0.0018	0.001
7.55	209.17	3.69	4.16	418,336.0	419,999.7	7,374.0	8,318.7	830.1	0.0	830.1	0.98	13.3	1.8	0.87	24.1	2.6	5.38E+05	553.4	0.1	14,312.2	2.31E-05	0.0024	3.6	86.1	23.7	10.8	0.0020	0.0017	0.001
7.71	297.86	2.80	2.87	595,720.0	596,867.8	5,598.0	5,739.0	848.1	0.0	848.1	0.98	13.6	0.9	0.83	29.3	2.5	5.98E+05	565.4	0.1	14,128.7	2.10E-05	0.0022	2.9	85.8	22.4	10.8	0.0019	0.0016	0.001
7.87	288.31	2.54	0.72	576,612.0	576,899.6	5,070.0	1,438.1	866.1	0.0	866.1	0.98	13.9	0.9	0.83	30.7	2.5	6.20E+05	577.4	0.1	13,951.3	2.04E-05	0.0021	2.8	86.6	22.4	10.8	0.0019	0.0016	0.001
8.04	250.90	2.60	0.33	501,792.0	501,923.2	5,202.0	655.8	884.2	0.0	884.2	0.98	14.2	1.0	0.87	26.3	2.6	5.88E+05	589.5	0.1	13,779.8	2.17E-05	0.0023	3.4	90.0	24.6	10.8	0.0018	0.0015	0.001
8.20	250.34	2.14	0.65	500,678.0	500,937.0	4,272.0	1,294.8	902.2	0.0	902.2	0.98	14.5	0.9	0.91	22.2	2.7	5.55E+05	601.5	0.1	13,613.8	2.32E-05	0.0024	4.2	93.8	27.2	10.8	0.0017	0.0015	0.001
8.37	288.84	2.20	0.81	577,670.0	577,992.2	4,392.0	1,611.2	920.3	0.0	920.3	0.98	14.8	0.8	0.88	25.1	2.6	5.87E+05	613.5	0.1	13,453.0	2.21E-05	0.0023	3.6	91.2	25.3	10.8	0.0017	0.0015	0.001
8.53	276.66	2.62	1.37	553,326.0	553,873.3	5,240.0	2,736.3	938.3	0.0	938.3	0.98	15.0	0.9	0.85	27.6	2.6	6.13E+05	625.5	0.1	13,297.1	2.14E-05	0.0022	3.2	89.0	23.9	10.8	0.0018	0.0016	0.001
8.69	226.36	3.69	1.37	452,710.0	453,257.3	7,374.0	2,736.3	956.4	0.0	956.4	0.98	15.3	1.6	0.80	34.3	2.4	6.77E+05	637.6	0.1	13,146.0	1.95E-05	0.0020	2.5	85.8	21.5	10.8	0.0019	0.0016	0.001
8.86	129.14	3.43	1.27	258,278.0	258,786.0	6,862.0	2,539.9	974.4	0.0	974.4	0.98	15.6	2.7	0.76	41.2	2.3	7.48E+05	649.6	0.1	12,999.4	1.78E-05	0.0018	2.1	86.6	20.8	10.8	0.0018	0.0015	0.001
9.02	99.58	2.34	3.16	199,168.0	200,431.6	4,682.0	6,318.1	992.5	0.0	992.5	0.98	15.9	2.3	0.75	48.6	2.3	8.70E+05	661.6	0.1	12,857.1	1.55E-05	0.0016	2.0	98.1	23.3	10.8	0.0013	0.0011	0.001
9.19	108.75	1.99	2.91	217,498.0	218,661.8	3,976.0	5,818.8	1,010.5	0.0	1,010.5	0.98	16.2	1.8	0.69	61.3	2.1	9.37E+05	673.7	0.1	12,718.8	1.45E-05	0.0015	1.5	94.2	20.7	10.8	0.0014	0.0012	0.001
9.35	146.97	1.83	0.69	293,934.0	294,210.3	3,668.0	1,381.5	1,028.5	0.0	1,028.5	0.98	16.5	1.3	0.71	53.7	2.2	8.78E+05	685.7	0.1	12,584.5	1.56E-05	0.0016	1.7	90.7	20.5	10.8	0.0016	0.0013	0.001
9.51	129.36	1.65	0.05	258,724.0	258,744.6	3,292.0	103.2	1,046.6	0.0	1,046.6	0.98	16.7	1.3	0.76	43.2	2.3	7.95E+05	697.7	0.1	12,453.8	1.74E-05	0.0018	2.1	88.7	21.1	10.8	0.0017	0.0014	0.001
9.68	101.79	1.44	-0.02	203,570.0	203,562.0	2,870.0	-40.0	1,064.6	0.0	1,064.6	0.98	17.0	1.4	0.76	41.7	2.3	7.81E+05	709.8	0.1	12,326.8	1.78E-05	0.0018	2.1	87.4	21.0	10.8	0.0017	0.0015	0.001
9.84	79.03	0.81	-0.02	158,054.0	158,047.3	1,624.0	-33.3	1,082.7	0.0	1,082.7	0.98	17.3	1.0	0.80	41.5	2.4	8.50E+05	721.8	0.1	12,203.1	1.65E-05	0.0017	2.5	101.8	25.4	10.8	0.0013	0.0011	0.001
10.01	73.01	0.65	-0.06	146,020.0	145,994.0	1,300.0	-129.9	1,100.8	0.4	1,100.4	0.98	17.6	0.9	0.80	40.1	2.4	8.36E+05	733.9	0.1	12,082.2	1.69E-05	0.0017	2.5	100.5	25.2	10.8	0.0013	0.0011	0.000
																									Total Estimated Settlement			2 x $\Sigma\Delta$**s**	**0.1**

DYNAMIC DENSIFICATION ANALYSES BASED ON CPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/2020 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2020-CPT-08					
Ground Elevation =	17.6	ft			
Water Table Depth from ground surface	9.6	ft	= EL	8	ft
γ =	110	pcf			
γ_{sat} =	120	pcf			
Atmospheric pressure	2,116.2	psf			
Cone Area Ratio	0.8				

Depth	q_c	f_s	Pore Pressure	q_c	qt	f_s	Pore Pressure	σ_v	u	σ_v'	rd	τ_{ave}	Fr	n	Qtn	Ic	G0	p	a	b	R	γ	Kc	Qtn,cs	N1(60),cs	Nc	$\varepsilon_{vol(15)}$	ε_{vol}	Δs
ft	tsf	tsf	tsf	psf	psf	psf	psf	psf	psf	psf		psf					psf												in
0.16	4.98	0.14	0.18	9,950.0	10,023.5	282.0	367.5	18.0	0.0	18.0	1.00	0.3	2.8	0.00	4.2	0.0	0.00E+00	12.0	0.1	142,350.9	0.00E+00	0.0000	0.0	0.0	0.0	10.8	0.0000	0.0000	0.000
0.33	4.98	0.08	0.18	9,950.0	10,022.4	166.0	362.2	36.1	0.0	36.1	1.00	0.6	1.7	1.02	4.2	3.0	3.06E+05	24.1	0.1	93,916.5	3.53E-05	0.0039	7.2	79.8	27.7	10.8	0.0026	0.0023	0.002
0.49	10.37	0.19	0.07	20,746.0	20,773.0	376.0	135.1	54.1	0.0	54.1	1.00	0.9	1.8	1.02	4.2	3.0	3.16E+05	36.1	0.1	73,635.5	3.46E-05	0.0038	7.0	80.7	27.7	10.8	0.0025	0.0022	0.002
0.66	17.67	0.26	0.13	35,330.0	35,381.9	518.0	259.5	72.2	0.0	72.2	1.00	1.2	1.5	1.03	4.2	3.0	3.26E+05	48.1	0.1	61,961.8	3.39E-05	0.0037	7.3	83.6	29.1	10.8	0.0023	0.0020	0.002
0.82	23.43	0.33	0.19	46,862.0	46,937.7	660.0	378.4	90.2	0.0	90.2	1.00	1.5	1.4	1.05	4.2	3.1	3.30E+05	60.1	0.1	54,197.4	3.39E-05	0.0037	8.0	87.3	31.5	10.8	0.0021	0.0018	0.001
0.98	31.88	0.39	0.23	63,764.0	63,854.8	786.0	454.0	108.3	0.0	108.3	1.00	1.8	1.2	1.06	0.0	3.1	3.32E+05	72.2	0.1	48,581.3	3.40E-05	0.0037	8.5	89.7	33.3	10.8	0.0020	0.0017	0.001
1.15	40.45	0.61	0.23	80,892.0	80,983.9	1,210.0	459.5	126.3	0.0	126.3	1.00	2.1	1.5	1.07	4.2	3.2	3.36E+05	84.2	0.1	44,289.6	3.39E-05	0.0037	8.7	90.5	33.9	10.8	0.0020	0.0017	0.001
1.31	37.48	0.91	0.23	74,958.0	75,048.8	1,822.0	454.0	144.4	0.0	144.4	1.00	2.4	2.4	1.05	4.2	3.1	3.43E+05	96.2	0.1	40,879.6	3.35E-05	0.0036	8.1	89.0	32.5	10.8	0.0020	0.0018	0.001
1.48	46.21	0.78	0.21	92,424.0	92,506.2	1,550.0	410.8	162.4	0.0	162.4	1.00	2.6	1.7	1.04	4.2	3.1	3.45E+05	108.3	0.1	38,090.3	3.37E-05	0.0036	7.6	86.0	30.5	10.8	0.0022	0.0019	0.002
1.64	37.68	0.76	0.16	75,354.0	75,418.9	1,520.0	324.3	180.4	0.0	180.4	1.00	2.9	2.0	1.03	4.2	3.1	3.39E+05	120.3	0.1	35,756.9	3.46E-05	0.0038	7.4	82.6	29.0	10.8	0.0024	0.0021	0.002
1.80	42.45	0.91	0.17	84,906.0	84,973.0	1,812.0	335.1	198.5	0.0	198.5	1.00	3.2	2.1	1.03	4.2	3.0	3.33E+05	132.3	0.1	33,769.5	3.56E-05	0.0039	7.2	79.2	27.5	10.8	0.0026	0.0023	0.002
1.97	52.94	1.21	0.20	105,878.0	105,959.1	2,414.0	405.4	216.5	0.0	216.5	1.00	3.5	2.3	1.02	4.2	3.0	3.34E+05	144.4	0.1	32,051.7	3.59E-05	0.0039	7.1	78.2	27.0	10.8	0.0027	0.0023	0.002
2.13	89.12	1.44	0.29	178,236.0	178,351.7	2,872.0	578.4	234.6	0.0	234.6	1.00	3.8	1.6	1.04	4.2	3.1	3.57E+05	156.4	0.1	30,548.8	3.39E-05	0.0037	7.7	85.7	30.5	10.8	0.0022	0.0019	0.001
2.30	91.29	1.70	0.25	182,588.0	182,687.4	3,392.0	497.2	252.6	0.0	252.6	0.99	4.1	1.9	0.99	4.2	2.9	3.88E+05	168.4	0.1	29,220.2	3.14E-05	0.0034	6.2	83.5	27.4	10.8	0.0023	0.0020	0.002
2.46	68.85	1.69	0.29	137,704.0	137,818.6	3,370.0	573.0	270.7	0.0	270.7	0.99	4.4	2.5	0.92	4.2	2.8	4.67E+05	180.4	0.1	28,035.3	2.63E-05	0.0028	4.5	84.9	25.0	10.8	0.0021	0.0018	0.001
2.62	93.24	1.29	0.26	186,488.0	186,594.0	2,582.0	529.8	288.7	0.0	288.7	0.99	4.7	1.4	0.87	4.2	2.6	5.38E+05	192.5	0.1	26,970.4	2.31E-05	0.0024	3.6	86.1	23.7	10.8	0.0020	0.0017	0.001
2.79	139.51	1.36	0.31	279,026.0	279,149.2	2,720.0	616.2	306.8	0.0	306.8	0.99	5.0	1.0	0.83	4.2	2.5	5.98E+05	204.5	0.1	26,007.0	2.10E-05	0.0022	2.9	85.8	22.4	10.8	0.0019	0.0016	0.001
2.95	275.86	1.11	0.26	551,722.0	551,828.0	2,224.0	529.8	324.8	0.0	324.8	0.99	5.3	0.4	0.83	4.2	2.5	6.20E+05	216.5	0.1	25,130.2	2.04E-05	0.0021	2.8	86.6	22.4	10.8	0.0019	0.0016	0.001
3.12	383.61	1.15	0.21	767,210.0	767,293.3	2,302.0	416.3	342.8	0.0	342.8	0.99	5.6	0.3	0.87	4.2	2.6	5.88E+05	228.6	0.1	24,328.1	2.17E-05	0.0023	3.4	90.0	24.6	10.8	0.0018	0.0015	0.001
3.28	412.77	0.90	0.24	825,546.0	825,641.1	1,796.0	475.6	360.9	0.0	360.9	0.99	5.9	0.2	0.91	4.2	2.7	5.55E+05	240.6	0.1	23,590.8	2.32E-05	0.0024	4.2	93.8	27.2	10.8	0.0017	0.0015	0.001
3.44	310.65	1.26	0.27	621,308.0	621,416.1	2,514.0	540.4	378.9	0.0	378.9	0.99	6.2	0.4	0.88	4.2	2.6	5.87E+05	252.6	0.1	22,910.2	2.21E-05	0.0023	3.6	91.2	25.3	10.8	0.0017	0.0015	0.001
3.61	190.87	1.91	0.22	381,738.0	381,827.7	3,822.0	448.6	397.0	0.0	397.0	0.99	6.4	1.0	0.85	4.2	2.6	6.13E+05	264.7	0.1	22,279.6	2.14E-05	0.0022	3.2	89.0	23.9	10.8	0.0018	0.0016	0.001
3.77	112.69	1.90	0.17	225,380.0	225,447.0	3,802.0	335.1	415.0	0.0	415.0	0.99	6.7	1.7	0.80	4.2	2.4	6.77E+05	276.7	0.1	21,693.2	1.95E-05	0.0020	2.5	85.8	21.5	10.8	0.0019	0.0016	0.001
3.94	68.77	1.54	0.29	137,534.0	137,649.7	3,082.0	578.4	433.1	0.0	433.1	0.99	7.0	2.2	0.76	4.2	2.3	7.48E+05	288.7	0.1	21,146.3	1.78E-05	0.0018	2.1	86.6	20.8	10.8	0.0018	0.0015	0.001
4.10	85.98	1.44	0.38	171,960.0	172,113.5	2,878.0	767.5	451.1	0.0	451.1	0.99	7.3	1.7	0.75	4.2	2.3	8.70E+05	300.7	0.1	20,634.6	1.55E-05	0.0016	2.0	98.1	23.3	10.8	0.0013	0.0011	0.001
4.27	83.55	1.12	0.31	167,100.0	167,224.3	2,230.0	621.6	469.2	0.0	469.2	0.99	7.6	1.3	0.69	4.2	2.1	9.37E+05	312.8	0.1	20,154.7	1.45E-05	0.0015	1.5	94.2	20.7	10.8	0.0014	0.0012	0.001
4.43	67.81	1.23	0.28	135,612.0	135,722.3	2,454.0	551.4	487.2	0.0	487.2	0.99	7.9	1.8	0.71	4.2	2.2	8.78E+05	324.8	0.1	19,703.4	1.56E-05	0.0016	1.7	90.7	20.5	10.8	0.0016	0.0013	0.001
4.59	62.66	0.66	0.24	125,324.0	125,419.1	1,310.0	475.6	505.2	0.0	505.2	0.99	8.2	1.0	0.76	4.2	2.3	7.95E+05	336.8	0.1	19,278.1	1.74E-05	0.0018	2.1	88.7	21.1	10.8	0.0017	0.0014	0.001
4.76	60.03	0.90	0.22	120,066.0	120,153.6	1,804.0	437.9	523.3	0.0	523.3	0.99	8.5	1.5	0.76	4.2	2.3	7.81E+05	348.9	0.1	18,876.5	1.78E-05	0.0018	2.1	87.4	21.0	10.8	0.0017	0.0015	0.001
4.92	40.67	0.91	0.10	81,344.0	81,384.0	1,824.0	200.0	541.3	0.0	541.3	0.99	8.8	2.3	0.80	4.2	2.4	8.50E+05	360.9	0.1	18,496.4	1.65E-05	0.0017	2.5	101.8	25.4	10.8	0.0013	0.0011	0.001
5.09	29.48	1.09	0.12	58,960.0	59,007.6	2,186.0	237.9	559.4	0.0	559.4	0.99	9.0	3.7	0.80	4.2	2.4	8.36E+05	372.9	0.1	18,136.1	1.69E-05	0.0017	2.5	100.5	25.2	10.8	0.0013	0.0011	0.001
5.25	43.10	0.35	0.14	86,206.0	86,263.3	708.0	286.6	577.4	0.0	577.4	0.99	9.3	0.8	0.83	4.2	2.5	7.98E+05	385.0	0.1	17,793.9	1.79E-05	0.0018	2.8	101.1	26.2	10.8	0.0013	0.0012	0.001
5.41	516.36	3.05	0.21	#########	#########	6,094.0	427.0	595.5	0.0	595.5	0.99	9.6	0.6	0.90	4.2	2.7	7.10E+05	397.0	0.1	17,468.3	2.02E-05	0.0021	4.0	106.4	30.4	10.8	0.0013	0.0011	0.001
5.58	273.35	4.62	0.16	546,690.0	546,756.0	9,234.0	329.8	613.5	0.0	613.5	0.99	9.9	1.7	0.90	4.2	2.7	6.93E+05	409.0	0.1	17,158.2	2.09E-05	0.0022	3.9	101.7	28.8	10.8	0.0014	0.0012	0.001
5.74	231.85	4.67	-0.17	463,706.0	463,636.8	9,336.0	-345.9	631.6	0.0	631.6	0.99	10.2	2.0	0.90	4.2	2.7	6.82E+05	421.0	0.1	16,862.4	2.14E-05	0.0022	3.9	99.2	28.1	10.8	0.0015	0.0013	0.001
5.91	228.94	2.71	0.02	457,884.0	457,893.7	5,422.0	48.7	649.6	0.0	649.6	0.99	10.5	1.2	0.83	4.2	2.5	7.34E+05	433.1	0.1	16,579.8	2.00E-05	0.0021	2.9	91.3	23.7	10.8	0.0017	0.0015	0.001
6.07	152.74	2.72	0.11	305,482.0	305,524.2	5,440.0	210.8	667.7	0.0	667.7	0.99	10.8	1.8	0.77	4.2	2.4	6.89E+05	445.1	0.1	16,309.4	2.15E-05	0.0022	2.2	75.0	18.2	10.8	0.0025	0.0022	0.002
6.23	109.64	2.46	0.04	219,276.0	219,293.3	4,914.0	86.5	685.7	0.0	685.7	0.99	11.1	2.2	0.76	4.2	2.3	7.08E+05	457.1	0.1	16,050.6	2.11E-05	0.0022	2.0	74.3	17.7	10.8	0.0025	0.0022	0.002
6.40	130.38	1.19	0.02	260,768.0	260,775.6	2,384.0	37.9	703.7	0.0	703.7	0.99	11.3	0.9	0.75	4.2	2.3	6.52E+05	469.2	0.1	15,802.3	2.31E-05	0.0024	2.0	67.1	15.8	10.8	0.0032	0.0028	0.002
6.56	119.19	1.48	0.02	238,382.0	238,391.7	2,954.0	48.7	721.8	0.0	721.8	0.98	11.6	1.2	0.76	4.2	2.3	6.18E+05	481.2	0.1	15,564.1	2.45E-05	0.0026	2.1	64.8	15.5	10.8	0.0035	0.0030	0.002
6.73	117.98	1.06	0.05	235,952.0	235,970.4	2,128.0	91.9	739.8	0.0	739.8	0.98	11.9	0.9	0.79	4.2	2.4	6.07E+05	493.2	0.1	15,335.2	2.52E-05	0.0026	2.3	66.0	16.2	10.8	0.0034	0.0029	0.002
6.89	142.65	1.93	0.06	285,302.0	285,327.9	3,854.0	129.7	757.9	0.0	757.9	0.98	12.2	1.4	0.85	4.2	2.6	6.30E+05	505.2	0.1	15,115.1	2.44E-05	0.0025	3.1	78.7	20.9	10.8	0.0024	0.0021	0.002
7.05	108.28	1.49	0.19	216,562.0	216,638.8	2,982.0	383.8	775.9	0.0	775.9	0.98	12.5	1.4	0.89	4.2	2.7	6.44E+05	517.3	0.1	14,903.2	2.41E-05	0.0025	3.7	86.5	24.1	10.8	0.0020	0.0017	0.001
7.22	205.85	1.72	0.31	411,700.0	411,822.2	3,434.0	610.8	794.0	0.0	794.0	0.98	12.8	0.8	0.90	4.2	2.7	7.39E+05	529.3	0.1	14,699.0	2.11E-05	0.0022	4.0	102.3	29.2	10.8	0.0014	0.0012	0.001
7.38	242.34	1.99	0.22	484,678.0	484,767.7	3,980.0	448.6	812.0	0.0	812.0	0.98	13.1	0.8	0.67	4.2	2.1	1.00E+06	541.3	0.1	14,502.2	1.57E-05	0.0016	1.4	89.7	19.4	10.8	0.0017	0.0014	0.001
7.55	162.46	2.11	0.25	324,928.0	325,029.6	4,210.0	508.2	830.1	0.0	830.1	0.98	13.3	1.3	0.56	4.2	1.8	1.14E+06	553.4	0.1	14,312.2	1.38E-05	0.0014	1.1	103.7	20.1	10.8	0.0014	0.0012	0.001
7.71	139.91	2.00	0.26	279,818.0	279,922.9	4,000.0	524.3	848.1	0.0	848.1	0.98	13.6	1.4	0.59	4.2	1.9	1.07E+06	565.4	0.1	14,128.7	1.49E-05	0.0015	1.2	94.8	18.9	10.8	0.0016	0.0014	0.001
7.87	149.80	2.07	0.26	299,602.0	299,706.9	4,138.0	524.3	866.1	0.0	866.1	0.98	13.9	1.4	0.63	4.2	2.0	1.00E+06	577.4	0.1	13,951.3	1.60E-05	0.0016	1.3	87.5	18.2	10.8	0.0018	0.0016	0.001
8.04	135.56	2.17	0.20	271,112.0	271,190.9	4,342.0	394.6	884.2	0.0	884.2	0.98	14.2	1.6	0.69	4.2	2.1	9.46E+05	589.5	0.1	13,779.8	1.71E-05	0.0018	1.5	84.2	18.5	10.8	0.0019	0.0017	0.001
8.20	112.32	1.44	0.07	224,646.0	224,674.1	2,882.0	140.5	902.2	0.0	902.2	0.98	14.5	1.3	0.72	4.2	2.2	9.04E+05	601.5	0.1	13,613.8	1.80E-05	0.0019	1.7	82.7	18.7	10.8	0.0020	0.0017	0.001
8.37	100.76	0.32	0.16	201,526.0	201,590.9	646.0	324.3	920.3	0.0	920.3	0.98	14.8	0.3	0.76	4.2	2.3	8.76E+05	613.5	0.1	13,453.0	1.87E-05	0.0019	2.0	84.7	20.1	10.8	0.0019	0.0017	0.001
8.53	89.63	0.36	0.06	179,252.0	179,275.8	718.0	118.9	938.3	0.0	938.3	0.98	15.0	0.4	0.79	4.2	2.4	8.87E+05	625.5	0.1	13,297.1	1.86E-05	0.0019	2.4	91.6	22.7	10.8	0.0017	0.0014	0.001
8.69	65.23	1.16	0.06	130,468.0	130,490.7	2,320.0	113.5	956.4	0.0	956.4	0.98	15.3	1.8	0.81	4.2	2.5	9.13E+05	637.6	0.1	13,146.0	1.82E-05	0.0019	2.6	97.2	24.5	10.8	0.0015	0.0013	0.001
8.86	41.13	0.51	0.03	82,250.0	82,261.9	1,018.0	59.5	974.4	0.0	974.4	0.98	15.6	1.3	0.80	4.2	2.4	8.69E+05	649.6	0.1	12,999.4	1.93E-05	0.0020	2.5	90.7	22.7	10.8	0.0017	0.0015	0.001
9.02	63.43	0.68	0.02	126,850.0	126,858.6	1,356.0	43.2	992.5	0.0	992.5	0.98	15.9	1.1	0.77	4.2	2.4	8.08E+05	661.6	0.1	12,857.1	2.09E-05	0.0022	2.1	78.6	18.9	10.8	0.0023	0.0020	0.002
9.19	32.67	0.62	0.04	65,348.0	65,364.2	1,244.0	81.1	1,010.5	0.0	1,010.5	0.98	16.2	1.9	0.74	4.2	2.3	8.15E+05	673.7	0.1	12,718.8	2.08E-05	0.0022	1.9	75.1	17.5	10.8	0.0025	0.0022	0.002
9.35	28.01	0.56	0.29	56,020.0	56,134.6	1,116.0	573.0	1,028.5	0.0	1,028.5	0.98	16.5	2.0	0.71	4.2	2.2	8.41E+05	685.7	0.1	12,584.5	2.03E-05	0.0021	1.6	73.3	16.4	10.8	0.0027	0.0023	0.002
9.51	24.05	0.35	0.17	48,106.0	48,175.2	698.0	345.9	1,046.6	0.0	1,046.6	0.98	16.7	1.5	0.70	4.2	2.2	8.37E+05	697.7	0.1	12,453.8	2.05E-05	0.0021	1.6	72.0	15.9	10.8	0.0028	0.0024	0.002
9.68	23.60	0.35	0.14	47,202.0	47,257.1	690.0	275.6	1,065.4	4.9	1,060.5	0.98	17.0	1.5	0.71	4.2	2.2	7.81E+05	710.3	0.1	12,321.3	2.22E-05	0.0023	1.7	67.9	15.3	10.8	0.0032	0.0027	0.000
																									Total Estimated Settlement			$2 \times \Sigma\Delta s$	0.2

DYNAMIC DENSIFICATION ANALYSES BASED ON SPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/20 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2002-B-1

Ground Elevation =			19.8	ft			
Depth to Groumd Water Table =			11.8	ft	= EL	8	ft
γ =	110	pcf					
γ_{sat} =	120	pcf					
Boring Diameter =	8	inch =	203.2	mm			
Rod Length Above Ground =	3	ft =	0.9	m			
$\varepsilon_{C,N}/\varepsilon_{C,N=15}$ =	0.925						
ϕ	35	degree					

Elevation	Depth	Depth	ΔH (ft)	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N	$N_{1,60}$	σ_m'	σ_m'	$K_{2(max)}$	G_{max}	r_d	$r_{eff}*G_{eff}/G_{max}$	Sand	r_{eff}	r_{eff}	$\varepsilon_{C,N=15}$	$\varepsilon_{C,N}$	ΔS (in)
ft	*ft*	*m*	*ft*	*blow/ft*	*psf*	*kPa*	*psf*	*kPa*		*Y/N*						*psf*	*tsf*		*psf*			*Y/N*		*%*	*%*	*%*	*in*
17.8	2	0.6	2.5	12	220.0	10.5	220.0	10.5	0.75	N	1.00	1.15	1	1.70	18	135.9	0.07	52.0	6.1E+05	1.00	1.9E-04	Y	0.00035	0.035	0.036	0.03	0.01
15.8	4	1.2	2.0	13	440.0	21.1	440.0	21.1	0.75	Y	1.13	1.15	1	1.70	22	271.8	0.14	55.6	9.2E+05	1.00	2.5E-04	Y	0.00036	0.036	0.030	0.03	0.01
13.8	6	1.8	2.5	13	660.0	31.6	660.0	31.6	0.80	N	1.00	1.15	1	1.70	20	407.6	0.20	54.6	1.1E+06	0.99	3.1E-04	Y	0.00500	0.500	0.500	0.46	0.14
10.3	9.5	2.9	2.5	31	1,045.0	50.1	1,045.0	50.1	0.85	N	1.00	1.15	1	1.28	39	645.4	0.32	67.8	1.7E+06	0.98	3.1E-04	Y	0.00250	0.250	0.075	0.07	0.02
																										Total	0.2
																										Multi-directional Shaking Total	0.4

DYNAMIC DENSIFICATION ANALYSES BASED ON SPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/20 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2002-B-2

Ground Elevation =			18.2	ft				
Depth to Groumd Water Table =			10.2	ft	= EL	8	ft	
γ =	110	pcf						
γ_{sat} =	120	pcf						
Boring Diameter =	8	inch =	203.2	mm				
Rod Length Above Ground =	3	ft =	0.9	m				
$\varepsilon_{C,N}/\varepsilon_{C,N=15}$ =	0.925							
ϕ	35	degree						

Elevation	Depth	Depth	ΔH (ft)	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N	$N_{1,60}$	σ_m'	σ_m'	$K_{2(max)}$	G_{max}	r_d	$r_{eff}*G_{eff}/G_{max}$	Sand	r_{eff}	r_{eff}	$\varepsilon_{C,N=15}$	$\varepsilon_{C,N}$	ΔS (in)
ft	ft	m	ft	blow/ft	psf	kPa	psf	kPa		Y/N						psf	tsf		psf			Y/N		%	%	%	in
16.2	2	0.6	1.0	9	220.0	10.5	220.0	10.5	0.75	N	1.00	1.15	1	1.70	13	135.9	0.07	47.3	5.5E+05	1.00	2.1E-04	Y	0.010	1.0	1.3	1.2	0.14
14.2	4	1.2	5.0	23	440.0	21.1	440.0	21.1	0.75	Y	1.23	1.15	1	1.70	41	271.8	0.14	69.2	1.1E+06	1.00	2.0E-04	Y	0.002	0.2	0.1	0.1	0.03
12.2	6	1.8	1.0	6	660.0	31.6	660.0	31.6	0.80	N	1.00	1.15	1	1.70	9	407.6	0.20	42.2	8.5E+05	0.99	4.0E-04	Y	0.010	1.0	1.0	0.9	0.11
																										Total	0.3
																										Multi-directional Shaking Total	0.6

DYNAMIC DENSIFICATION ANALYSES BASED ON SPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/20 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2002-B-3

Ground Elevation =			19.2	ft		
Depth to Groumd Water Table =			11.2	ft	= EL 8	ft
γ =	110	pcf				
γ_{sat} =	120	pcf				
Boring Diameter =	8	inch =	203.2	mm		
Rod Length Above Ground =	3	ft =	0.9	m		
$\varepsilon_{C,N}/\varepsilon_{C,N=15}$ =	0.925					
ϕ	35	degree				

Elevation	Depth	Depth	ΔH (ft)	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N	$N_{1,60}$	σ_m'	σ_m'	$K_{2(max)}$	G_{max}	r_d	$r_{eff}*G_{eff}/G_{max}$	Sand	r_{eff}	r_{eff}	$\varepsilon_{C,N=15}$	$\varepsilon_{C,N}$	ΔS (in)
ft	*ft*	*m*	*ft*	*blow/ft*	*psf*	*kPa*	*psf*	*kPa*		*Y/N*						*psf*	*tsf*		*psf*			*Y/N*		*%*	*%*	*%*	*in*
14.7	4.5	1.4	5.0	21	495.0	23.7	495.0	23.7	0.75	N	1.00	1.15	1	1.70	31	305.7	0.15	62.7	1.1E+06	0.99	2.4E-04	Y	0.003	0.3	0.2	0.1	0.09
																										Total	0.1
																										Multi-directional Shaking Total	0.2

DYNAMIC DENSIFICATION ANALYSES BASED ON SPT DATA
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER, OAKLAND, CALIFORNIA

04.72190021
2/20/20 TC

a_{max} =	0.81	g	ASCE 7-16
Mw =	7.0		

2020-B-01

Ground Elevation =			17.5	ft		
Depth to Ground Water Table =			9.5	ft	= EL	8 ft
γ =	110	pcf				
γ_{sat} =	120	pcf				
Boring Diameter =	4	inch =	101.6	mm		
Rod Length Above Ground =	3	ft =	0.9	m		
$\varepsilon_{C,N}/\varepsilon_{C,N=15}$ =	0.925					
ϕ	35	degree				
Energy Ratio =	84%					

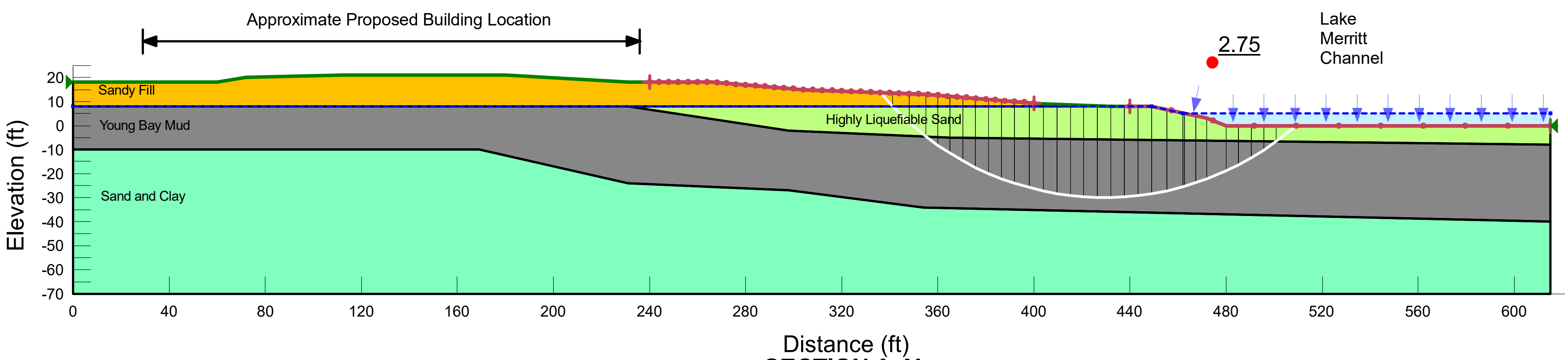
Elevation	Depth	Depth	ΔH (ft)	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N	$N_{1,60}$	σ_m'	σ_m'	$K_{2(max)}$	G_{max}	r_d	$r_{eff}*G_{eff}/G_{max}$	Sand	r_{eff}	r_{eff}	$\varepsilon_{C,N=15}$	$\varepsilon_{C,N}$	ΔS (in)
ft	*ft*	*m*	*ft*	*blow/ft*	*psf*	*kPa*	*psf*	*kPa*		*Y/N*						*psf*	*tsf*		*psf*			*Y/N*		*%*	*%*	*%*	*in*
13.5	4.0	1.2	6.5	19	440.0	21.1	440.0	21.1	0.75	N	1.00	1	1.4	1.70	34	271.8	0.14	64.7	1.1E+06	1.00	2.2E-04	Y	0.0025	0.250	0.100	0.09	0.07
9.5	8.0	2.4	3.0	4	880.0	42.2	880.0	42.2	0.80	Y	1.10	1	1.4	1.70	8	543.5	0.27	40.6	9.5E+05	0.98	4.8E-04	N	-	-	-	-	-
																										Total	0.1
																										Multi-directional Shaking Total	0.1

Supplement F

Slope Stability Analyses

Title: Laney College Library Learning Resource Center
File Name: Section A-A'.gsz
Description: Case 1 - Static Long Term
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		

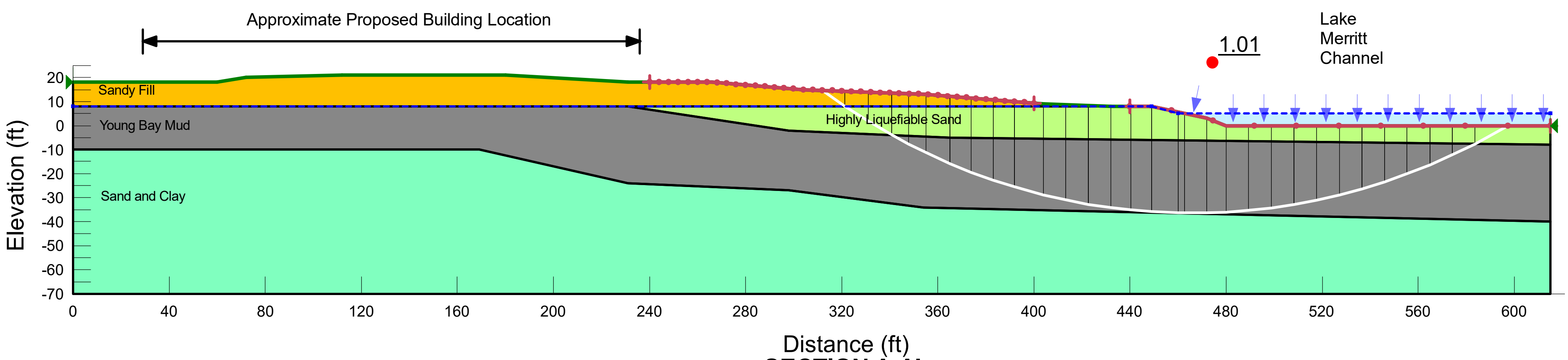


SECTION A-A'

PLATE F-1

Title: Laney College Library Learning Resource Center
File Name: Section A-A'.gsz
Description: Case 2 - Pseudo-Static Yield Acceleration
Horz Seismic Coef.: 0.12
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		

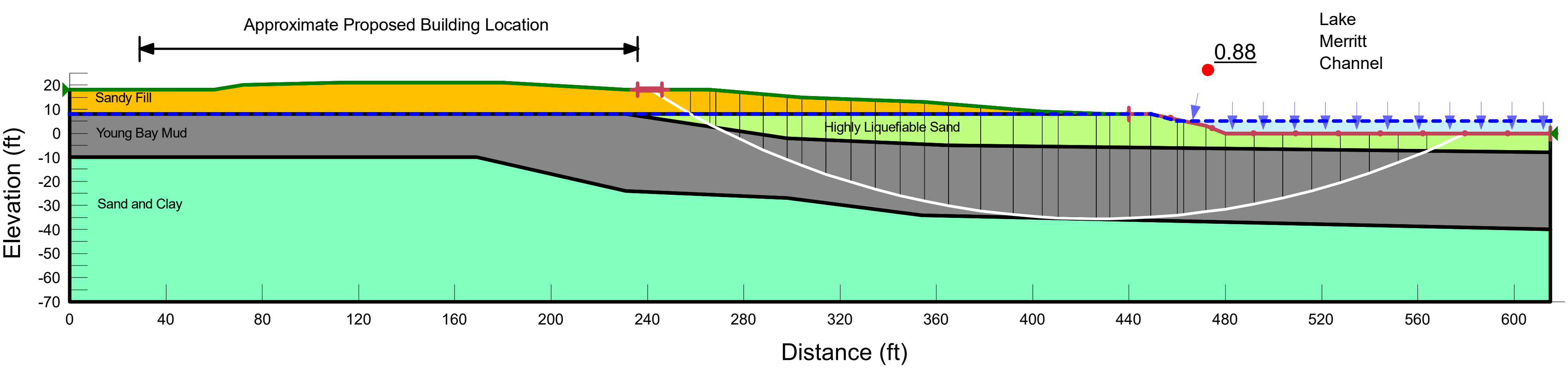


SECTION A-A'

PLATE F-2

Title: Laney College Library Learning Resource Center
File Name: Section A-A'.gsz
Description: Case 3 - Pseudo-Static k = 0.15g; Fixed Slip Surface at Edge of Building
Horz Seismic Coef.: 0.15
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		

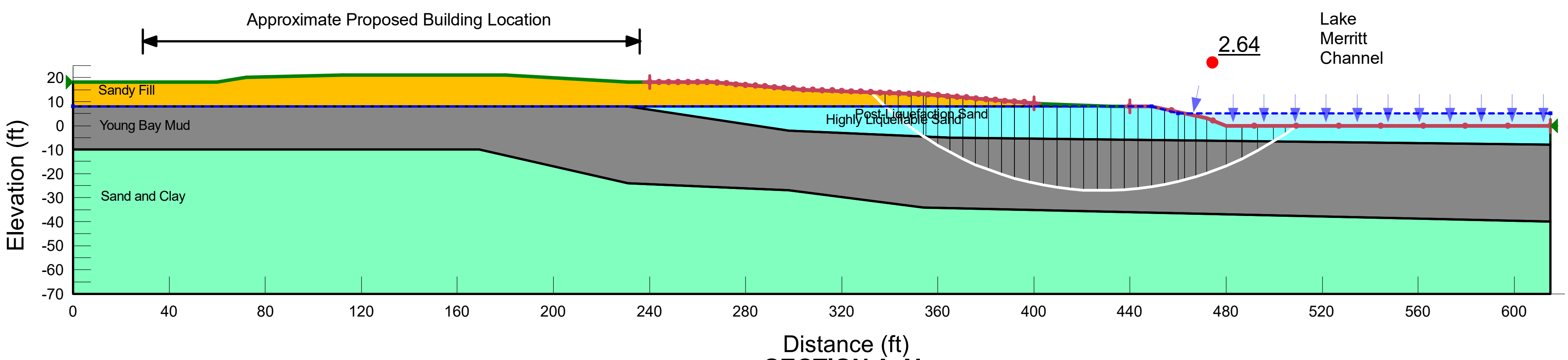


SECTION A-A'

PLATE F-3

Title: Laney College Library Learning Resource Center
File Name: Section A-A'.gsz
Description: Case 4 - Post-Liquefaction
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)	C-Datum (psf)	C-Rate of Change ((lbs/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)
	Sandy Fill	Mohr-Coulomb	120	0	35	1						
	Young Bay Mud	S=f(overburden)	90			1	0.35	350				
	Sand and Clay	Mohr-Coulomb	130	0	40	1						
	Post-Liquefaction Sand	S=f(datum)	110			1			100	20	500	8

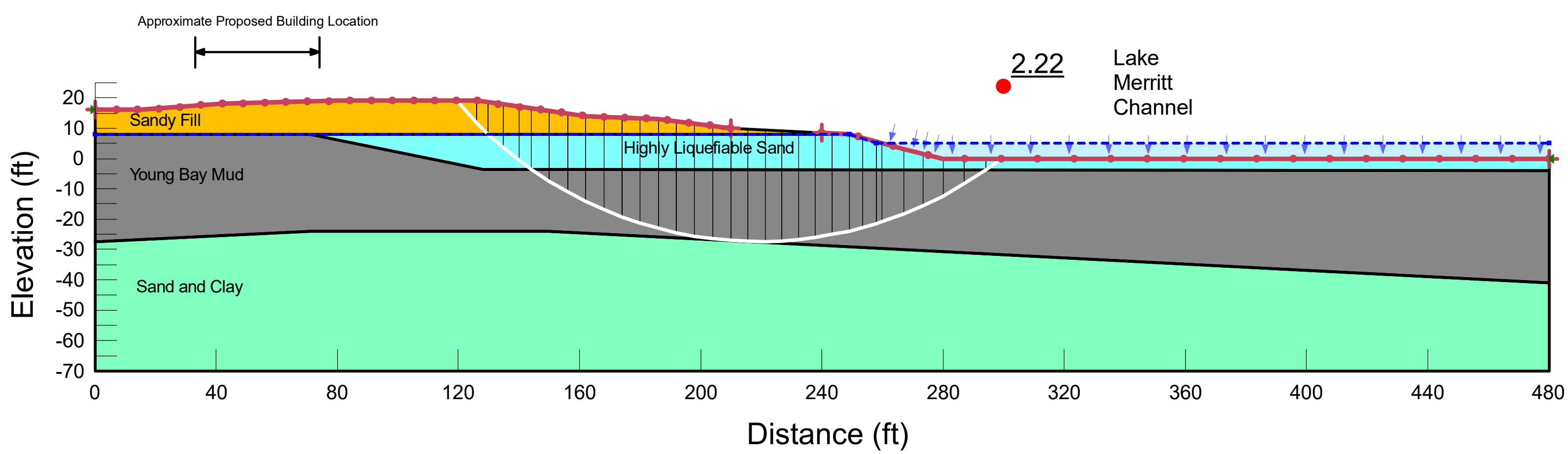


SECTION A-A'

PLATE F-4

Title: Laney College Library Learning Resource Center
File Name: Section D-D'.gsz
Description: Case 1 - Static Long Term
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		

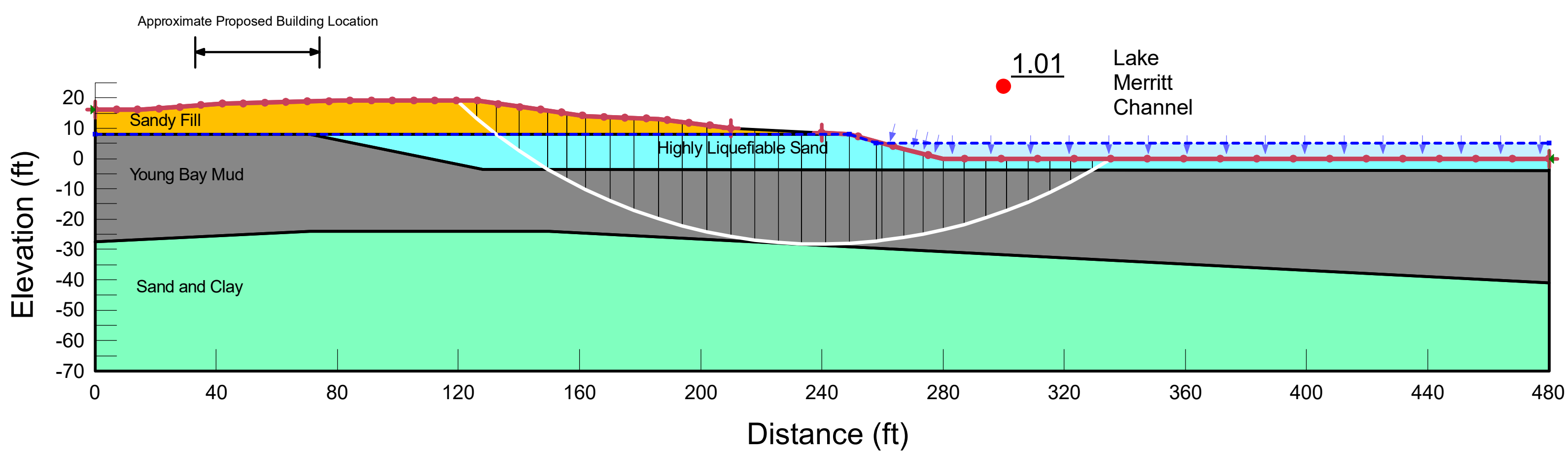


SECTION D-D'

PLATE F-5

Title: Laney College Library Learning Resource Center
File Name: Section D-D'.gsz
Description: Case 2 - Pseudo-Static Yield Acceleration
Horz Seismic Coef.: 0.12
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		

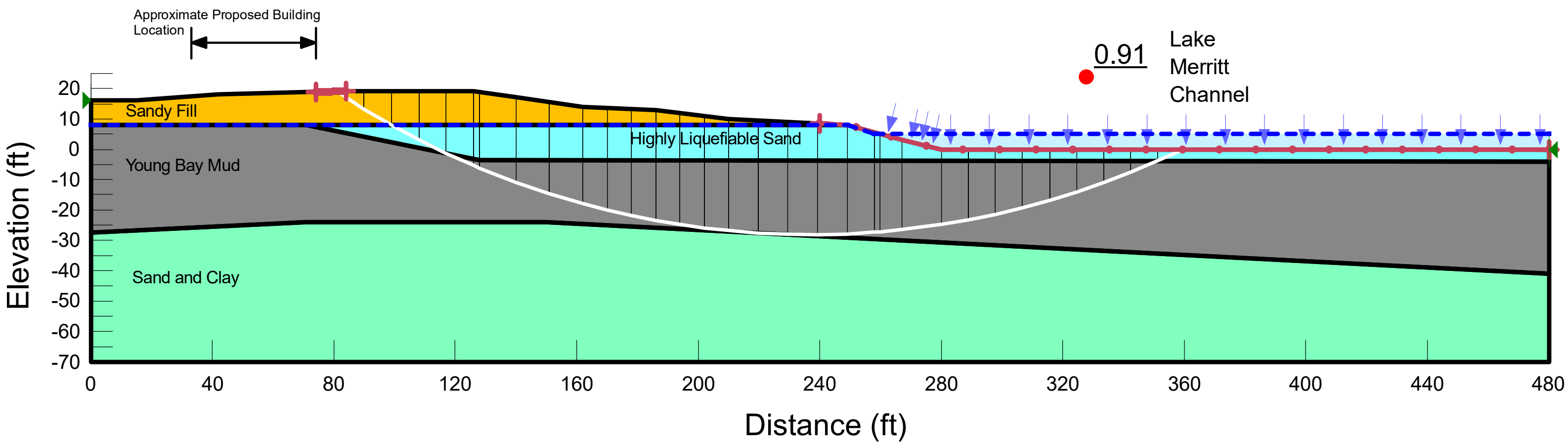


SECTION D-D'

PLATE F-6

Title: Laney College Library Learning Resource Center
File Name: Section D-D'.gsz
Description: Case 3 - Pesudo-Static k = 0.15g; Fixed Slip Surface at Edge of Building
Horz Seismic Coef.: 0.15
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		

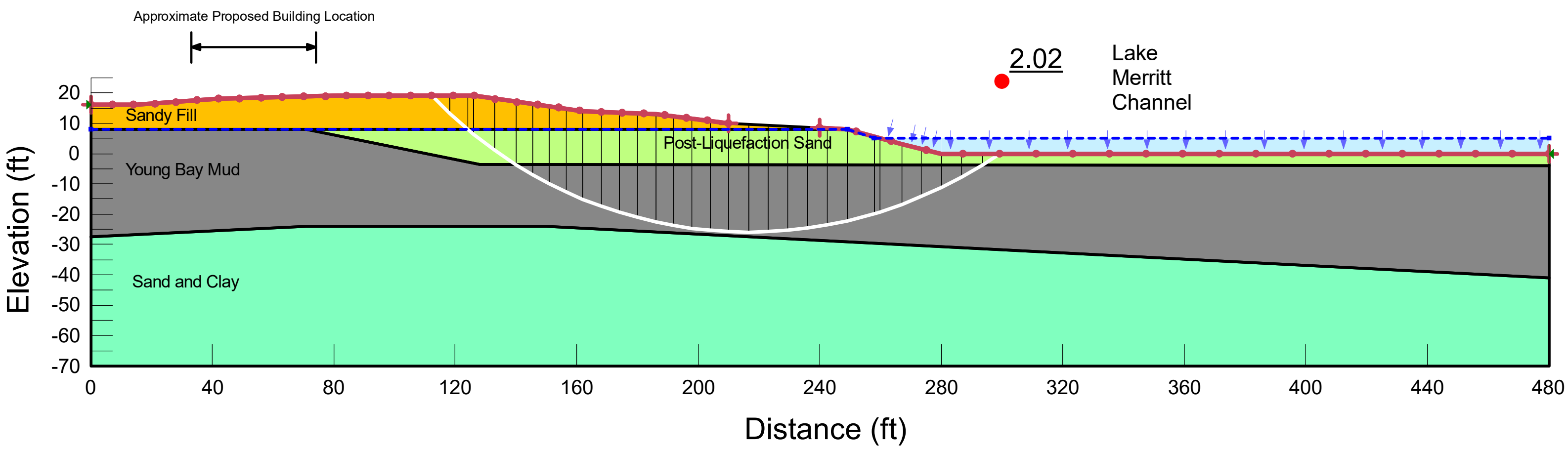


SECTION D-D'

PLATE F-7

Title: Laney College Library Learning Resource Center
File Name: Section D-D'.gsz
Description: Case 4 - Post-Liquefaction
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	C-Datum (psf)	C-Rate of Change ((lbs/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1						
	Post-Liquefaction Sand	S=f(datum)	110			1	100	20	500	8		
	Young Bay Mud	S=f(overburden)	90			1					0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1						

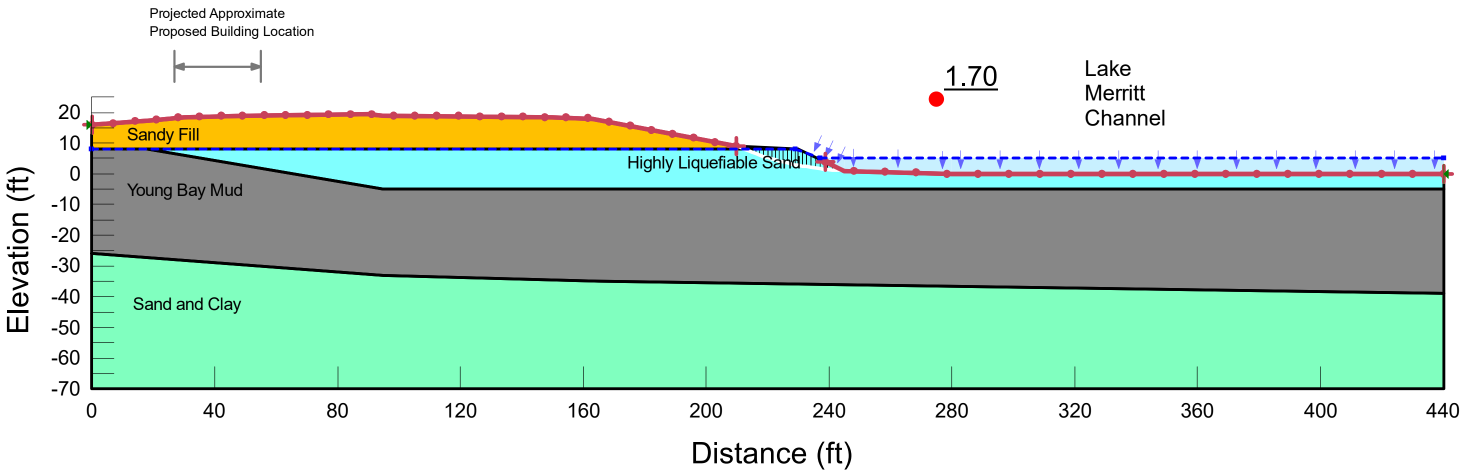


SECTION D-D'

PLATE F-8

Title: Laney College Library Learning Resource Center
File Name: Section E-E'.gsz
Description: Case 1 - Static Long Term
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		

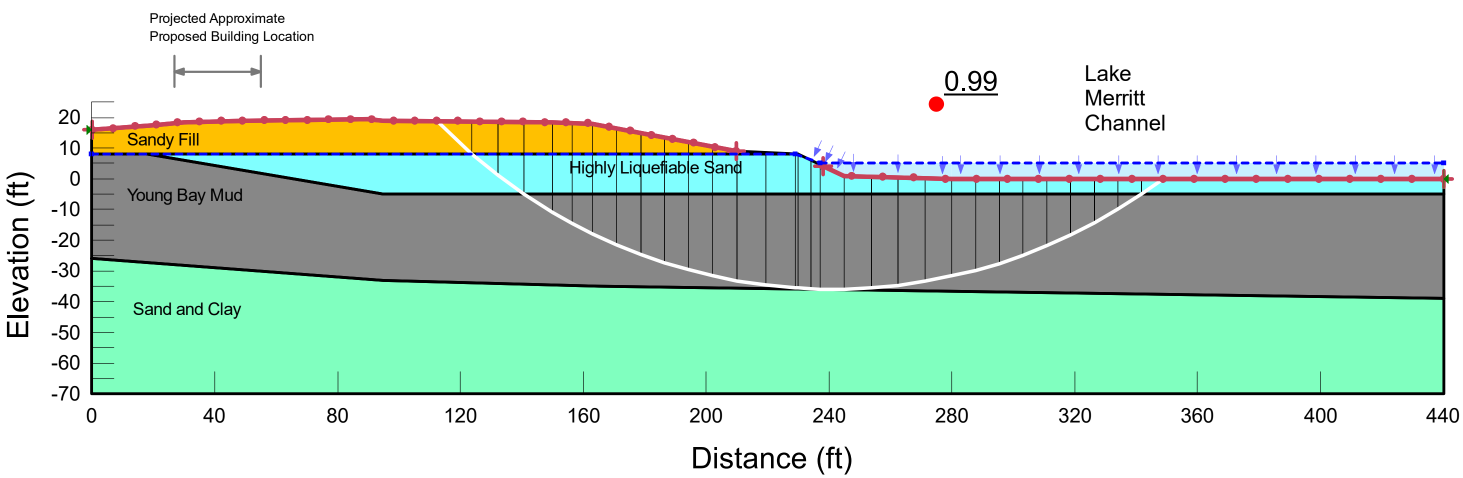


SECTION E-E'

PLATE F-9

Title: Laney College Library Learning Resource Center
File Name: Section E-E'.gsz
Description: Case 2 - Pseudo-Static Yield Acceleration
Horz Seismic Coef.: 0.11
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		

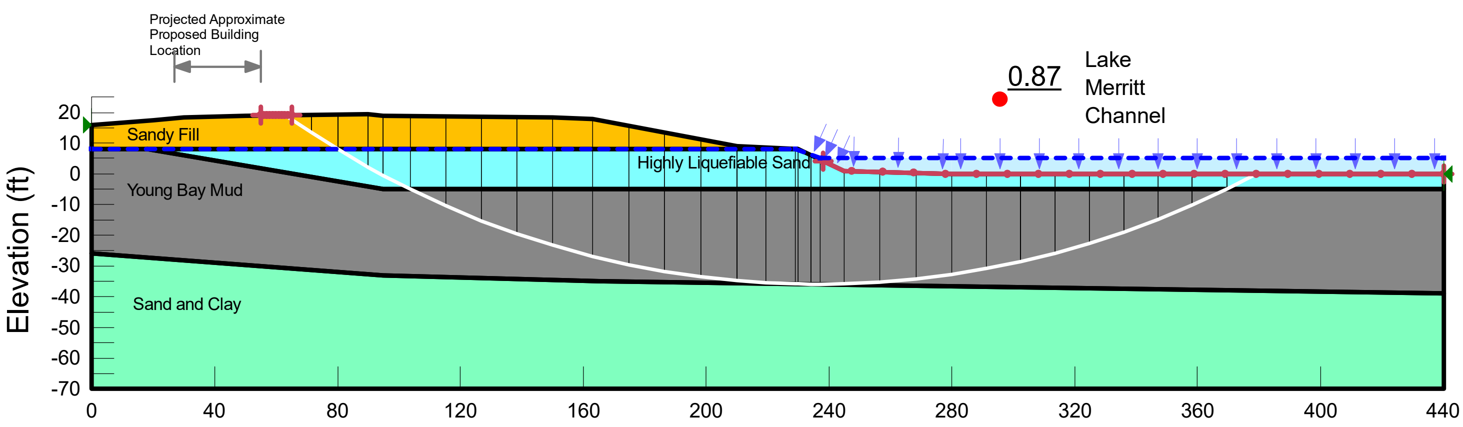


SECTION E-E'

PLATE F-10

Title: Laney College Library Learning Resource Center
File Name: Section E-E'.gsz
Description: Case 3 - Pesudo-Static k = 0.15g; Fixed Slip Surface at Edge of Building
Horz Seismic Coef.: 0.15
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		

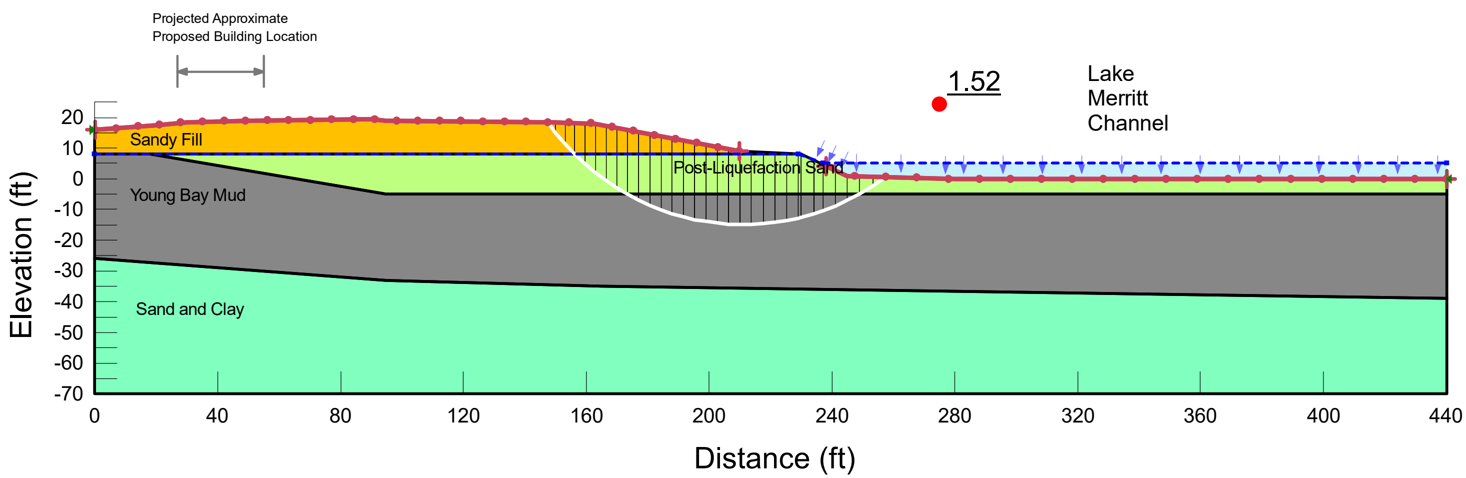


SECTION E-E'

PLATE F-11

Title: Laney College Library Learning Resource Center
File Name: Section E-E'.gsz
Description: Case 4 - Post-Liquefaction
Horz Seismic Coef.: 0
Method: Spencer

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	C-Datum (psf)	C-Rate of Change ((lbs/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1						
	Post-Liquefaction Sand	S=f(datum)	110			1	100	20	500	8		
	Young Bay Mud	S=f(overburden)	90			1					0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1						



SECTION E-E'

PLATE F-12

Supplement G

Site-Specific Ground

Motion Analyses

Contents

Tables in the Main Text

G.1 Introduction

This appendix summarizes a site-specific seismic hazard assessment and site response analyses conducted to estimate the severity of ground motions that may affect the project site for specific design levels of hazard. The seismic hazard assessment was conducted using the seismic source model adopted by the United States Geological Survey (USGS) to develop the 2014 National Seismic Hazard Map Project (NSHMP) (Petersen et al., 2014), and the NGA West 2 Ground Motion Models (Bozorgnia et al., 2014).

A liquefaction triggering hazard assessment indicated that the soils at the site are potentially liquefiable. Therefore, according to ASCE 7-16, the site is classified as Site Class F, and site response analyses are required to calculate the design ground motions at the ground surface. These site response analyses were performed using the commercial finite-difference program FLAC (Itasca, 2016) and evaluated the effect of nonlinear dynamic response of the soft and liquefiable soils at the site on the surface ground motions. The design ground motion parameters were calculated following the site-specific ground motion procedures defined in Chapter 21 of ASCE 7-16 (ASCE, 2016; 2018) as required by the 2019 California Building Code (CBC) (CBSC, 2019).

G.2 Subsurface Conditions for the Seismic Hazard Assessment

Subsurface conditions at the project site generally consist of approximately 10 feet (ft) of sandy fill overlaying approximately 20 to 30 ft of soft Young Bay Mud (YBM) overlaying denser sands and stiffer clays (e.g., see **Plates 7 and 9** of the main text). Liquefiable sand seams on the order of 5 ft in thickness exist within the YBM (these sands are referred to as YBM Sand herein). Bedrock at the project site is expected to exist at depths greater than approximately 500 ft (Rodgers and Figuers, 1991). Idealization of subsurface conditions for the seismic hazard assessment was based primarily on data from geotechnical borings (including standard penetration test [SPT] and laboratory test data) and cone penetration test (CPT) soundings performed at the project site. Locations of the project explorations and interpreted cross sections are shown on **Plate 3** of the main text.

Free-field site response analyses were performed for a one-dimensional soil column extending from the ground surface to the base of the YBM. The denser sands and stiffer clays underlying the YBM are considered competent (Site Class D), and consequently their effect on seismic wave propagation at the site is captured reasonably well by the ground motion models used in the seismic hazard assessment.

G.2.1 Shear Wave Velocity

The time-weighted average shear wave velocity (Vs) in the top 100 ft (30 meters [m]) (Vs30) is an important input parameter to include the local site conditions in the seismic hazard assessment.

Similarly, characterization of the small-strain stiffness, G (where $G = \rho V_s^2$ and ρ is density) is important for site response analysis. In-situ Vs measurements were conducted by Gregg Drilling and Testing for the seismic CPT-07 located between the building footprint and the Lake Merritt Channel (2020-CPT-07 on **Plate 3** of the main text; data presented in **Appendix A**). These measurements are shown on **Figure G.2-1** alongside Vs values calculated from empirical correlations between Vs and CPT data using the same CPT sounding. Two CPT-based shear wave velocity correlations are shown on **Figure G.2-1**; the Mayne and Rix (1995) correlation for clays is shown within the YBM and the Andrus et al. (2007) correlation is shown for all other strata. The correlations are consistent with the seismic measurements for this CPT sounding in the YBM and competent clays and sands underlying the YBM. Strata demarcations for CPT-07 consistent with the interpreted cross sections (e.g., **Plate 8** of the main text) are also shown on this figure. **Figure G.2-2** shows correlated Vs values for all project CPT soundings, where Mayne and Rix (1995) is shown for YBM and Andrus et al. (2007) is shown for all other strata. This range of data approximately represents the variability of Vs across the site. The relatively small range of correlated Vs values in YBM across all CPT soundings is similar to the range of measured values for CPT-07. Idealized shear wave velocities within the YBM and competent sands and clays underlying the YBM are shown on **Figure G.2-3**. Measured and correlated Vs values for CPT-07 are also shown on this figure. Extrapolation of shear wave velocities in the competent soils underlying the YBM was based on review of data from (1) local Fugro projects and (2) near the former Cypress Structure (Rogers and Figuers, 1991). A Vs30 from the base of the YBM of approximately 860 ft/s (260 m/s), corresponding to Site Class D per ASCE 7-16, was computed using the idealization shown on **Figure G.2-3** and was used for the seismic hazard assessment to develop input ground motions for the site response analyses. Vs30 from the ground surface was estimated to be approximately 560 ft/s (170 m/s), corresponding to Site Class E per ASCE 7-16; however, Site Class F was assigned because of the presence of potentially liquefiable YBM Sand seams. The Site Class F classification requires that a site response analysis in accordance with ASCE 7-16 Section 21.1 be performed.

G.2.2 Young Bay Mud Undrained Shear Strength

The undrained shear strength (s_u) of YBM was evaluated based on CPT and laboratory test data. YBM undrained shear strengths from (1) unconsolidated undrained (UU) triaxial compression tests, (2) unconfined compression (UC) tests, and (3) CPT measurements (i.e., $s_u = q_{t,net}/N_{kt}$ where $q_{t,net}$ is the net total cone resistance and the cone factor N_{kt} = 20) are shown on **Figure G.2-4**. The CPT data are shown as a hexagonally binned two-dimensional histogram (hexbin). The laboratory test data are biased low (i.e., they fall near the lower bound of the CPT data) likely because of sample disturbance effects. The idealized YBM undrained shear strength used for the site response analyses (i.e., for calibration of the modulus reduction and damping factor [MRDF] constitutive model as described in **Section G.6.1**) is also shown on **Figure G.2-4**.

Note that these are static strengths which were empirically adjusted for rate effects for the site response analyses as described in **Section G.6.1**.

G.2.3 Penetration Resistance for Sand-Like Soils (Fill and YBM Sand)

Penetration resistances in the fill and YBM Sand are summarized on **Figure G.2-5** which plots $(N_1)_{60cs}$ (i.e., equivalent clean sand blow counts corrected to 60% energy ratio and an effective overburden of one atmosphere) versus elevation. Hexbin profiles of correlated $(N_1)_{60cs}$ values from CPT data (per the procedures described by Boulanger and Idriss [2014]) are in good agreement with SPT measurements (shown with triangular markers on **Figure G.2-5**). Blow counts in the saturated YBM Sand are mostly between 9 and 16, whereas blow counts in the fill range from roughly 10 to greater than 30.

G.2.4 Idealized Profiles for One-Dimensional Site Response Analyses

Figure G.2-6 shows three idealized soil profiles used for the site response analyses. These profiles reasonably represent the expected stratigraphic variation beneath the building footprint (note that deeper YBM was encountered closer to the Lake Merritt Channel, outside of the building footprint, e.g., 2020-CPT-06 on **Plate 7**). The three idealized profiles are described below.

- **Profile P1** (deep YBM) consists of 10 ft of fill overlaying 31 ft of YBM.
- **Profile P2** (deep YBM with liquefiable sand) consists of 10 ft of fill overlaying 31 ft of YBM with a 5-foot-thick liquefiable YBM Sand layer within the YBM from depths of 25 to 30 ft.
- **Profile P3** (shallow YBM) consists 10 ft of fill overlaying 18 ft of YBM.

G.3 Probabilistic Seismic Hazard Analysis

A site-specific seismic hazard assessment was conducted for a Vs30 of 860 ft/s (260 m/s) corresponding to the base of the YBM, to calculate the input design ground motions for the site response analyses.

G.3.1 Project Location

A Probabilistic Seismic Hazard Analysis (PSHA) was conducted for one representative location of the project site. The geographical coordinates of the location used for the seismic hazard analyses are tabulated in **Table G.1**.

Table G.1: Representative Project Location Coordinates used in the PSHA

Latitude	Longitude
37.7948°N	122.2624°W

G.3.2 Methodology

PSHA Framework

The methodology for a PSHA includes the following components:

1. Seismic Source Model. This includes defining the location, style, and rates of earthquake occurrence in the model area. The characterization includes developing values for the following seismic source parameters:

   i. Source location and geometry. All major active faults and seismotectonic provinces are defined within the model area. This includes the geographical extent at the surface as well as the orientation and depth of the source zones.
   ii. Source type (e.g., shallow crustal area source zones, fault sources, subduction zones, etc.) and style of faulting (e.g., normal, strike-slip, reverse, etc.).
   iii. Magnitude potential (i.e., range of earthquake sizes possible on each source) and magnitude distribution (i.e., characterized using a magnitude probability density function).
   iv. Earthquake magnitude recurrence, which is a characterization of the annual rate at which earthquakes of a specified magnitude or greater occur in each source.

2. Ground Motion Model. Characterization of ground motion attenuation characteristics of each source are based on the geologic and tectonic environment. These characteristics are described by a series of ground motion models, or GMM (also known as "attenuation relationships," "attenuation models," or "ground motion prediction equations").
3. Probabilistic Seismic Hazard Analysis. A PSHA uses inputs from the seismic source model and GMMs selected for the specific environment, to estimate the ground motion hazard at the site. The hazard is expressed in terms of the annual frequency of exceeding a given spectral acceleration at the project site (i.e., annual hazard curves). This information also can be shown in the form of uniform hazard response spectra (UHRS), which correspond to spectral acceleration having the same probability of exceedance across all structural periods. The UHRS are typically used by different design codes to define the design response spectra.

PSHA Calculation

Computation of the seismic hazard involves the combination of uncertainties in earthquake size, location, frequency, and resulting ground motions. The estimated annual rate at which the ground motion, A, will exceed a particular value, a, is computed by (Cornell, 1968):

$$\lambda[A > a] = \sum_{i=1}^{Nsource} N(M_{\min}) \iint P[A > a | m, r]\ f_M(m)\ f_R(r)\, dm\, dr$$

Equation 1

where N_{source} is the total number of seismic sources; $N(M_{min})$ is the annual rate of earthquake with magnitude greater than or equal to M_{min}; $P[A > a|m, r]$ is the probability of the ground motion, A, exceeding the threshold value, a, given the earthquake magnitude and distance from the seismic source; and $f_M(m)$ and $f_R(r)$ are probability density functions describing magnitude and distance.

The computation of this integral is carried out numerically. By assuming that earthquake occurrence can be modeled as a Poisson process, the probability of exceedance in a specified exposure period (typically corresponding to the useful life of a project) may be estimated as follows:

$$P[A > a, t] = 1 - e^{-[\lambda(a)t]}$$

Equation 2

where $P[A > a, \mathrm{t}]$ is the conditional probability of the spectral acceleration (A) exceeding a specified acceleration (a) during a time interval (t) given that an earthquake will occur, and $\lambda(a)$ is the mean annual rate of exceedance of the specified acceleration level.

Seismic Source Model

The PSHA was conducted using the seismic source model adopted by the UGSG to develop the 2014 NSHMP (Petersen et al., 2014) for California which corresponds to the Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3). The details of this seismic source model can be found in Field et al. (2013).

Empirical Ground Motion Models

The attenuation of seismic waves from a seismic source were modeled using empirical ground motion models (GMM's). These empirical GMM's should model the type of rupture mechanism as well as the regional geology to properly estimate site-specific strong ground motion parameters. Four of the Next Generation Attenuation (NGA) West 2 GMM's (Bozorgnia et al., 2014) were used. These four NGA West 2 GMM are: Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), and Chiou and Youngs (2014). The four NGA West 2 GMM's were equally weighted, following the weighting scheme used in the development of the 2014 USGS NSHMP (Petersen et al., 2014).

Implementation

The PSHA was performed using the USGS computer code *nshmp-haz*, which has been used by the USGS to develop the US national seismic hazard maps.

G.3.3 Results from the PSHA

Figure G.3-1 shows the mean annual seismic hazard curves for selected spectral periods ranging from 0.01 to 10 seconds for a Vs30 of 260 m/sec. A spectral period of 0.01 seconds is used to represent the peak ground acceleration (PGA). These hazard curves represent the total mean hazard from combining all seismic sources and ground motion models. This figure also indicates the annual frequency of exceedance corresponding to a return period of 2,475 years.

Table G.2 tabulates the mean magnitude, distance, and epsilon calculated from the seismic hazard deaggregation for PGA and Sa (spectral acceleration) at 1 second for a return period of 2,475 years. Epsilon is the number of standard deviations that the estimated ground motion amplitude deviates from the estimated median ground motion amplitude. Thus, an epsilon of 1 indicates that the probabilistic value of the ground motion corresponds to a median plus one-standard-deviation value.

Table G.2: Mean Seismic Hazard Deaggregation for a Return Period of 2,475 years and Vs30 of 260 m/sec

	PGA	Sa at 1 sec.
Mean Magnitude (Mw)	7.00	7.27
Mean Distance (km)	9.2	10.0
Mean Epsilon	1.8	1.7

Figure G.3-2 presents the 5 percent-damped mean horizontal UHRS for a return period of 2,475 years and a Vs30 of 260 m/sec. **Table G.3** tabulates the mean horizontal UHRS for periods ranging from 0.01 (i.e., PGA) to 10 seconds for a return period of 2,475 years.

Table G.3: Mean Horizontal UHRS for Return Period of 2,475 Years and a Vs30 of 260 m/sec, 5% Damping

Period (sec)	Horizontal Spectral Acceleration (g)
0.01 (PGA)	0.933
0.03	0.957
0.05	1.07
0.075	1.32
0.1	1.55
0.15	1.83
0.2	2.05
0.25	2.23
0.3	2.36
0.4	2.42
0.5	2.35
0.75	1.96
1	1.65
1.5	1.19
2	0.924
3	0.606
4	0.429
5	0.320
7.5	0.177
10	0.110

G.4 Design Response Spectra at Base of YBM

According to ASCE 7-16, for Site Class D sites with S1 (mapped 5% damped spectral response acceleration parameter at a period of 1 second) greater than or equal to 0.2 g, the design response spectrum and design acceleration parameters should be developed following the site-specific ground motion procedures defined in Section 21.2 of ASCE 7-16. The S1 for the project site was calculated as 0.660 g using the USGS web service (https://earthquake.usgs.gov/ws/designmaps/asce7-16.html). Therefore, the design ground motions for the site should be calculated using the site-specific procedures from ASCE 7-16.

ASCE 7-16 defines a site-specific Risk-Targeted Maximum Considered Earthquake (MCE_R) as the lesser of probabilistic (MCE_R) and deterministic (MCE_R) ground motions. The probabilistic MCE_R ground motion is calculated as the ground motion in the direction of maximum horizontal response that is expected to achieve 1 percent probability of collapse within a 50-year period. The deterministic MCE_R ground motion is defined as the 84th percentile ground motion in the direction of maximum horizontal response of the largest acceleration from deterministic seismic hazard analysis (DSHA) of the characteristic earthquakes on all known active faults within the project region. Additionally, ASCE 7-16 specifies a lower limit to the deterministic MCE_R ground motion. The site-specific MCE_R should not be less than 150 percent of the site-specific design response spectrum. The site-specific design response spectrum is calculated as 2/3 of the site-specific MCE_R. The site-specific design response spectrum should be greater than or equal to 80 percent of the spectral acceleration as determined by using the general response spectrum of Section 11.4.6 of ASCE 7-16, using modified Fa and Fv values provided in Section 21.3 of ASCE 7-16.

The PSHA results described in the previous section were used to calculate the probabilistic MCE_R spectrum. As specified in ASCE 7-16, to obtain ground motions with a uniform 1 percent probability of collapse within a 50-year period, the UHRS for a return period of 2,475 was scaled by a risk coefficient, C_R. The C_R values were calculated using Method 1 described in Chapter 21 of ASCE 7-16. The mapped risk coefficients at spectral periods of 0.2 and 1.0 sec, C_{RS} and C_{R1}, respectively, were determined using the USGS web service (https://earthquake.usgs.gov/ws/designmaps/asce7-16.html). The value of these risk coefficients C_{RS} and C_{R1} are 0.921 and 0.906, respectively. The ground motions in the direction of maximum horizontal response were calculated by applying the scaling factors recommended in ASCE 7-16. **Figure G.4-1** shows the UHRS for a return period of 2,475 years along with the probabilistic MCE_R response spectrum.

The deterministic MCE_R spectrum was calculated by performing a DSHA in EZ-FRISK™ (Fugro, 2019) using the same seismic sources and GMM's used in the PSHA. The UCERF3 source model includes magnitude frequency distributions (MFD's) which relate frequency of occurrence to earthquake magnitude; however, these MFD's include multi-fault ruptures scenarios with large magnitudes but with low probability of occurrence. Therefore, following the current USGS approach to calculate deterministic ground motions from the UCERF3 source model, to estimate the characteristic magnitude for the seismic sources, we used the empirical relationships proposed by Wells and Coppersmith (1994) that relates rupture geometry to earthquake magnitude. The ground motions in the direction of maximum horizontal response were calculated by applying the scaling factors recommended in ASCE 7-16. **Figure G.4-1** illustrates the calculation of the deterministic MCE_R response spectrum. The deterministic MCE_R response spectrum was calculated as the maximum of the 84th DSHA response spectrum and the lower limit specified by ASCE 7-16 Supplement 1 calculated for a Site Class D.

Figure G.4-2 presents the development of the site-specific MCE_R and design response spectra for the base of the YBM. In this case, the deterministic MCE_R spectrum is lower than the probabilistic MCE_R spectrum for all spectral periods. The site-specific MCE_R spectrum is the maximum of: 1) the minimum of the probabilistic and deterministic MCE_R, and 2) 150 percent of the design response spectrum. Following ASCE 7-16, the design response spectrum was calculated as the maximum of 2/3 of the site-specific MCE_R and the lower limit specified by ASCE 7-16 (80 percent of the general spectrum for Site Class D, using modified Fa and Fv values provided in Section 21.3 of ASCE 7-16). The transition period from constant velocity to constant displacement, T_L, required to calculate the lower limit, was estimated as 8 seconds using the USGS web service (https://earthquake.usgs.gov/ws/designmaps/asce7-16.html).

Table G.4 tabulates the spectral ordinates of the recommended site-specific MCE_R and design response spectra per ASCE 7-16 for the base of the YBM.

Table G.4: MCE$_R$ and Design Response Spectra per ASCE 7-16 for a Vs30 of 260 m/sec (base of YBM), 5% Damping

Period (sec)	Horizontal Spectral Acceleration (g)									
	UHRS for Return Period of 2,475 Years	Risk Coefficients	Max. Direction Scaling Factors	Probabilistic MCE$_R$	84th Deterministic Spectrum	Deterministic Lower Limit	Deterministic MCE$_R$	Site-Specific MCE$_R$	80% General Response Spectrum	Design Response Spectrum
0.01 (PGA)	0.933	0.921	1.10	0.945	0.711	0.555	0.782	0.782	0.400	0.521
0.03	0.957	0.921	1.10	0.970	0.717	0.559	0.789	0.789	0.459	0.526
0.05	1.07	0.921	1.10	1.08	0.783	0.611	0.861	0.861	0.518	0.574
0.075	1.32	0.921	1.10	1.34	0.928	0.724	1.02	1.02	0.591	0.680
0.1	1.55	0.921	1.10	1.57	1.07	0.831	1.17	1.17	0.664	0.781
0.15	1.83	0.921	1.10	1.86	1.29	1.01	1.42	1.42	0.811	0.946
0.190	2.01	0.921	1.10	2.04	1.42	1.11	1.56	1.56	0.927	1.04
0.2	2.05	0.921	1.10	2.08	1.45	1.13	1.60	1.60	0.927	1.06
0.25	2.23	0.919	1.13	2.32	1.57	1.26	1.77	1.77	0.927	1.18
0.3	2.36	0.917	1.15	2.49	1.66	1.36	1.91	1.91	0.927	1.28
0.4	2.42	0.915	1.19	2.62	1.75	1.48	2.08	2.08	0.927	1.39
0.5	2.35	0.912	1.21	2.61	1.74	1.50	2.11	2.11	0.927	1.41
0.75	1.96	0.909	1.26	2.25	1.50	1.34	1.89	1.89	0.927	1.26
0.949	1.70	0.906	1.29	2.00	1.33	1.22	1.73	1.73	0.927	1.15
1	1.65	0.906	1.30	1.95	1.30	1.20	1.69	1.69	0.880	1.13
1.5	1.19	0.906	1.35	1.46	0.983	0.942	1.33	1.33	0.587	0.885
2	0.924	0.906	1.39	1.16	0.783	0.770	1.09	1.09	0.440	0.724
3	0.606	0.906	1.44	0.789	0.538	0.548	0.773	0.773	0.293	0.515
4	0.429	0.906	1.47	0.572	0.383	0.400	0.564	0.564	0.220	0.376
5	0.320	0.906	1.50	0.435	0.283	0.301	0.425	0.425	0.176	0.283
7.5	0.177	0.906	1.50	0.240	0.140	0.149	0.210	0.210	0.117	0.140
8	0.159	0.906	1.50	0.216	0.124	0.131	0.185	0.185	0.110	0.124
10	0.110	0.906	1.50	0.150	0.0801	0.0852	0.120	0.120	0.0704	0.0801

G.5 Ground Motion Acceleration Time Histories for Input to Site Response Analyses

G.5.1 Selection of Seed Ground Motions

Following Section 21.1.1 of ASCE 7-16, five pairs of orthogonal recorded horizontal seed ground motion (GM's) acceleration time histories were selected and scaled to comply with the site-specific MCE_R response spectrum at the base of the YBM developed in the previous section. During the selection of seed GM's, we considered the following criteria:

- The selected GM's were recorded from seismic events that are comparable with events that control the MCE_R scenario from the seismic deaggregation.
- The shape of the GM's acceleration response spectra.
- The lowest usable frequency of the selected GM's.
- Other criteria including strong motion duration, Arias Intensity, faulting mechanism, and shear wave velocity at the site where the GM's were recorded.

Table G.5 lists the properties of the selected seed GM's.

Table G.5: Selected Seed Ground Motions

No.	Record Sequence Number (RSN)	Earthquake Name	Recording Station	Moment Magnitude (Mw)	Faulting Mechanism	Vs30 of Recording Station (m/s)	Rupture/ Closest Distance (km)	Minimum Usable Frequency (Hz)	Average Scaling Factor
1	729	1987 Superstition Hills-02	Imperial Valley Wildlife Liquefaction Array	6.54	Strike slip	179	24	0.1	4.1
2	1545	199 Chi-Chi_ Taiwan	TCU120	7.62	Reverse Oblique	459	7.4	0.0375	4.1
3	6952	2010 Darfield_ New Zealand	Papanui High School	7	Strike slip	263	19	0.0625	4.0
4	806	1989 Loma Prieta	Sunnyvale - Colton Ave.	6.93	Reverse Oblique	268	24	0.1	4.4
5	1176	1999 Kocaeli_ Turkey	Yarimca	7.51	Strike slip	297	5	0.0875	3.3

G.5.2 Scaling of Seed Ground Motions

Figure G.5-1 shows a comparison between the response spectra of the two components (H1, H2) for each of the linearly scaled ground motions (thin colored lines), the mean response spectra of the five scaled motions (thick red line) and the target MCE_R at the base of the YBM (thick black line). On average, the mean of the scaled acceleration response spectra shows good agreement with the target response spectrum.

The scale factor for each of the seed ground motions was selected such that the average of their spectral accelerations within the period range from 0.05 seconds to 5 seconds matches, on average, the spectral accelerations of the target MCE_R response spectrum within the same period range. The average scaling factor for the response spectra of the two components of the seed ground motions is listed in **Table G.5** above.

G.6 One-Dimensional Site Response Analyses

According to ASCE 7-16, for sites classified as Site Class F, the design response spectrum and design acceleration parameters should be developed following the site-specific ground motion procedures defined in Chapter 21 of ASCE 716. Specifically, site response analyses shall be performed in accordance with ASCE 7-16 Section 21.1. The approach, analyses, and results for one-dimensional free-field site response analyses are presented herein.

G.6.1 Approach

One-Dimensional Site Response Modelling in FLAC

One-dimensional site response analyses were performed using the commercial finite difference program FLAC (Fast Analysis of Continua) (Itasca, 2016). One-dimensional site response was modeled with a single column of 2.5-foot square zones. Analyses were performed for the three idealized profiles shown on **Figure G.2-6**. The water table was modeled at the base of the fill for all profiles. Analyses were performed using the user defined constitutive models MRDF (modulus reduction and damping factor hysteretic model, Hashash et al., 2010) and PM4Sand (Boulanger and Ziotopoulou, 2017). MRDF was used to model the fill and YBM, and PM4Sand was used to model the liquefiable, saturated YBM Sand in profile P2. Analyses were performed for each of the 10 scaled ground motion time histories (5 ground motion records, 2 components) developed in the previous section.

For dynamic simulation, a quiet (absorbing) boundary was used at the base of the model and the lateral boundaries were attached (i.e., at a given elevation the left and right nodes displace together). A single elastic zone was included at the base of the model with properties representative of the competent soils underlaying the YBM (i.e., Vs30 of 860 ft/s). Outcrop ground motions were input at the base of the model (at the quiet boundary) as shear stress time histories. Shear stress time histories were computed from outcrop acceleration time histories by integrating to obtain velocity and multiplying by twice the competent soil density times the competent soil Vs per the compliant base procedure proposed by Mejia and Dawson (2006).

Constitutive Calibration and Input Parameters

The bases for constitutive model calibration and input parameters are summarized in **Table G.6**. YBM shear wave velocity was modeled using the idealization shown on **Figure G.2-3**. Shear wave velocity in the fill and YBM Sand was modeled based on correlation to SPT blow count. Representative $(N_1)_{60cs}$ values of 17 and 12 were used to model the fill and YBM Sand, respectively. These $(N_1)_{60cs}$ values correspond to V_{s1} = 586 ft/s in the fill (i.e., V_s ranges from about 300 to 500 ft/s in the fil) and V_{s1} = 544 ft/s in the YBM Sand (i.e., V_s of about 550 ft/s in the YBM Sand).

Target empirical shear modulus reduction (G/G_{max}) and material damping relationships are summarized in **Table G.6**. In general, the degree to which the target relationships are represented by the calibrated models depends on the model (i.e., MRDF vs. PM4Sand) and the calibration procedure. For MRDF, fitting parameters can be selected to produce near exact matches with target shear modulus reduction and damping curves, however, such calibrations may underpredict or overpredict shear strength depending on the small-strain stiffness (G). For site response analyses, the relative importance of matching these behaviors (i.e., empirical G/G_{max} and shear strength) depends on the strain-level of interest and is problem dependent. Soft clays at the project site are expected to develop large shear strains for the MCE_R level of shaking, hence MRDF was calibrated to honor the idealized undrained shear strength profile shown on **Figure G.2-4;** a dynamic multiplier of 1.4 was applied to these idealized strengths to account for strain-rate effects. This was done following the procedure described by Hashash et al. (2010) where G/G_{max} values for shear strains greater than 0.1% are adjusted to achieve the desired shear strength. For PM4Sand primary input parameters were correlated to $(N_1)_{60cs}$ as described by Boulanger and Ziotopoulou (2017); all secondary input parameters used default values. Boulanger and Ziotopoulou (2017) demonstrate reasonable consistency with the EPRI (1993) modulus reduction and damping curves for a range of $(N_1)_{60cs}$ and effective overburden pressures.

Lastly, the PM4Sand contraction rate parameter was calibrated based on $(N_1)_{60cs}$ and the Idriss and Boulanger (2008) SPT-based liquefaction triggering correlation.

Table G.6: Constitutive Model Calibration Basis

Strata	Constitutive Model	Shear wave velocity, Vs	Basis for MRDF Strength	G/G_{max} and Damping Ratio Curve Source(s)
Fill	MRDF	$V_{s1} = 85[(N_1)_{60} + 2.5]^{0.25}$ m/s (Boulanger and Ziotopoulou, 2017)	Bolton (1986) strength-dilatancy relationship for plane strain ($\varphi'_{cv} = 33°$)	EPRI (1993)
YBM	MRDF	V_s = 310 ft/s at 10 ft depth Increasing at 5 ft/s/ft (**Figure G.2-3**)	**Figure G.2-4** with 1.4 dynamic multiplier	Fugro (2007, 2020)
YBM Sand	PM4Sand	$V_{s1} = 85[(N_1)_{60} + 2.5]^{0.25}$ m/s (Boulanger and Ziotopoulou, 2017)	N/A	EPRI (1993)

Verification of Modelling Approach

To verify the FLAC modeling approach (i.e., the numerical platform, application of earthquake loading, MRDF constitutive model implementation, etc.), a subset of analyses was performed using both FLAC and DEEPSOIL (Hashash et al., 2017). Comparisons between FLAC and DEEPSOIL were made for profile P1 for two levels of shaking (the MCE_R and a smaller level of shaking with PGA ≈ 0.45 g). Comparisons of results obtained from the two analysis platforms showed near identical surface response spectra, stress-strain responses, and profiles of maximum shear strain, PGA, and maximum shear stress. The FLAC modelling approach was adopted for all other analyses (including modelling of liquefiable YBM Sand in profile P2), as described in the preceding sections.

G.6.2 Results

Baseline Analyses

Results for one-dimensional site response analyses for profile P1, P2, and P3 are shown on **Figure G.6-1** and **Figure G.6-2**. Profiles of absolute maximum shear strain and PGA are shown on **Figure G.6-1**. The thin lines are for individual ground motions and the thick lines are mean responses per profile. Overall, large shear strains develop in the YBM at the MCE_R level of shaking. Surface response spectra and amplification ratios are shown on **Figure G.6-2**. The amplification ratios were calculated as the ratio of the response spectra at the surface to the input response spectrum. The thin lines show responses for each ground motion time history and the thick lines show mean responses per idealized profile. Overall, there is little variation in the mean surface spectra for the three profiles analyzed. The shorter period (higher frequency) mean responses exhibit significant deamplification, whereas periods greater than approximately three seconds exhibit amplified responses. Yielding in the YBM deamplifies higher frequencies and effectively base isolates the soil column, hence there is little difference in the surface response spectra for the three idealized profiles. For smaller levels of shaking, clear differences in the response of the three profiles is expected.

Figure G.6-3 shows the idealized amplification ratios developed based on the average amplification ratios from the site response analyses. The idealized amplification ratios consider variability on the soil stratigraphy and variability on ground motion time histories. However, sensitivity analyses conducted showed similar amplification ratios by considering variability in soil properties (YBM shear wave velocity and undrained shear strength).

Parametric Analyses

Parametric analyses were performed for profile P1 to evaluate the effect of lower bound YBM shear wave velocities and a range of YBM undrained shear strength idealizations on the site response. Overall, these parameter variations had little effect on the surface spectrum (for the same reasons discussed above). An upper bound undrained shear strength profile caused the most significant change to the surface spectrum, slightly increasing the amplification for periods between about 1.5 to 4 seconds while decreasing the amplification for periods greater than approximately 4 seconds. Even with an upper bound undrained shear strength, large shear strains developed throughout the YBM (mean absolute maximum shear strains were on the order of 10 to 20 percent).

G.7 Design Response Spectra at the Ground Surface

Figure G.7-1 presents the development of the site-specific MCE_R and design response spectra for the ground surface. The MCE_R response spectrum from the site response analyses is calculated as the site-specific MCE_R at the base of the YBM (input to the site response analyses) multiplied by the idealized amplification ratios presented on **Figure G.6-3**. The site-specific MCE_R spectrum is the maximum of: 1) MCE_R response spectrum from the site response analyses, and 2) 150 percent of the design response spectrum. Following ASCE 7-16, the design response spectrum was calculated as the maximum of 2/3 of the site-specific MCE_R and the lower limit specified by ASCE 7-16 (80 percent of the general spectrum for Site Class E, using modified Fa and Fv values provided in Section 21.3 of ASCE 7-16). The transition period from constant velocity to constant displacement, T_L, required to calculate the lower limit, was estimated as 8 seconds using the USGS web service (https://earthquake.usgs.gov/ws/designmaps/asce7-16.html).

Table G.7 tabulates the spectral ordinates of the recommended site-specific MCE_R and design response spectra per ASCE 7-16 for the ground surface. The corresponding design acceleration parameters S_{MS}, S_{M1}, S_{DS}, and S_{D1} are tabulated in **Table G.8**.

Table G.7: MCE_R and Design Response Spectra per ASCE 7-16 at the Ground Surface, 5% Damping

Period (sec)	Horizontal Spectral Acceleration (g)		
	Site-Specific MCE_R	80% General Response Spectrum	Design Response Spectrum
0.01 (PGA)	0.584	0.389	0.389
0.03	0.639	0.426	0.426
0.05	0.694	0.463	0.463
0.075	0.763	0.508	0.508
0.1	0.831	0.554	0.554
0.15	0.969	0.646	0.646
0.2	1.11	0.738	0.738
0.25	1.24	0.829	0.829
0.3	1.38	0.921	0.921
0.304	1.39	0.927	0.927
0.4	1.39	0.927	0.927
0.5	1.39	0.927	0.927
0.75	1.39	0.927	0.927
1	1.39	0.927	0.927
1.5	1.39	0.927	0.927
1.52	1.39	0.927	0.927
2	1.06	0.704	0.704
3	0.827	0.469	0.551
4	0.733	0.352	0.489
5	0.561	0.282	0.374
7.5	0.282	0.188	0.188
8	0.264	0.176	0.176
10	0.169	0.113	0.113

Table G.8: Design Acceleration Parameters per ASCE 7-16 at the Ground Surface, 5% Damping

Parameter	Value
S_{MS}	1.39 g
S_{M1}	2.93 g
S_{DS}	0.927 g
S_{D1}	1.96 g

G.8 References

Abrahamson N.A., Silva W., and Kamai R., 2014. Summary of the ASK14 Ground-Motion Relation for Active Crustal Regions. Earthquake Spectra, 30 (1), pp 1025–1055.

(ASCE) American Society of Civil Engineers, 2016. ASCE Standard 7-16 – Minimum Design Loads for Buildings and Other Structures. ASCE 7-16.

(ASCE) American Society of Civil Engineers, 2018. ASCE Standard 7-16 – Minimum Design Loads for Buildings and Other Structures, Supplement 1.

Andrus, R.D., Mohanan, N.P., Piratheepan, P., Ellis, B.S. and Holzer, T.L., 2007. Predicting shear-wave velocity from cone penetration resistance. In: Proceedings of the 4th international conference on earthquake geotechnical engineering, Thessaloniki, Greece (Vol. 2528).

Bolton, M.D., 1986. The strength and dilatancy of sands. Geotechnique, 36 (1), pp .65-78.

Boore D.M., Stewart J.P., Seyhan E., and Atkinson G.M., 2014. NGA-West 2 Equations for Predicting PGA, PGV, and 5%-Damped PSA for Shallow Crustal Earthquakes. Earthquake Spectra, 30 (3), pp 1057–1085.

Boulanger, R.W. and Idriss I.M., 2014. CPT and SPT based liquefaction triggering procedures. Report No. UCD/CGM-14/01.

Boulanger, R.W., and K. Ziotopoulou. 2017. PM4Sand (version 3.1): A sand plasticity model for earthquake engineering applications. Report No. UCD/CGM-17/01. Davis, CA: Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California. March. p. 114.

Bozorgnia Y., Abrahamson N.A., Al Atik L., Ancheta T.D., Atkinson G.M., Baker J.W., Baltay A., Boore D.M., Campbell K.W., Chiou B.S.J., Darragh R., Day S., Donahue J., Graves R.W., Gregor N., Hanks T., Idriss I.M., Kamai R., Kishida T., Kottke A., Mahin S.A., Rezaeian S., Rowshandel B., Seyhan E., Shahi S., Shantz T., Silva W., Spudich P., Stewart J.P., Watson-Lamprey J., Wooddell K., and Youngs R., 2014. NGA-West2 Research Project. Earthquake Spectra, 30 (3), pp. 973-987.

(CBSC) California Building Standards Commission, 2019. 2019 California Building Code.

Campbell K.W. and Bozorgnia Y., 2014. NGA-West2 Ground Motion Model for the Average Horizontal Components of PGA, PGV, and 5% Damped Linear Acceleration Response Spectra. Earthquake Spectra, 30, (3), pp. 1087–1115.

Chiou, B.S.J. and Youngs, R.R., 2014. Update of the Chiou and Youngs NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra. Earthquake Spectra, 30 (3), pp. 1117–1153.

Cornell, C.A., 1968. Engineering Seismic Risk Analysis. Seismological Society of America Bulletin, 58 (5).

Field, E. L., Biasi, G.P., Bird, P., Dawson, T.E., Felzer, K.R., Jackson, D.D., Johnson, K.M., Jordan, T.H., Madden, C., Michael, A.J., Milner, K.R., Page, M.T., Parsons, T., Powers, P.M., Shaw, B.E., Thatcher, W.R., Weldon, R.J., II, and Zeng, Y., 2013. Uniform California earthquake rupture forecast, version 3 (UCERF3)—The time-independent model. U.S. Geological Survey Open-File Report 2013–1165, California Geological Survey Special Report 228, and Southern California Earthquake Center Publication 1792, p. 97, http://pubs.usgs.gov/of/2013/1165.

Electric Power Research Institute (EPRI), 1993. Guidelines for determining design basis ground motions. Final Report No. TR-102293. Palo Alto, California.

Fugro, 2007. Geotechnical Design Report for BART San Francisco Transition Structure Seismic Retrofit. BART Earthquake Safety Program, Project 1180.018. September 24.

Fugro, 2020. Geotechnical Interpretive Report, Seawall Earthquake Safety Program, San Francisco, California. San Francisco Seawall Earthquake Safety and Disaster Prevention Program. Fugro draft Report No. 04.72170066-PR-004(V1), dated September January 31.

Fugro, 2019. EZ-FRISK, Software for Earthquake Ground Motion Estimation, Version 8.06. http://www.ez-frisk.com/.

Hashash, Y.M.A., Musgrove, M.I., Harmon, J.A., Ilhan, O., Groholski, D.R., Phillips, C.A., and Park, D., 2017. DEEPSOIL 7.0, User Manual.

Hashash, Y.M.A., Phillips, C.A, and Groholski, D.R., 2010. Recent Advances in Non-linear Site Response Analysis. In: Proceedings of the Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics. San Diego, CA, May 2010.

Idriss, I.M. and Boulanger, R.W., 2008. Soil liquefaction during earthquakes. Earthquake Engineering Research Institute.

Itasca Consulting Group Inc. 2016. FLAC, Fast Lagrangian Analysis of Continua, User's Guide, Version 8.0. Minneapolis, MN.

Mayne, P.W. and Rix, G.J., 1995. Correlations between shear wave velocity and cone tip resistance in natural clays. Soils and foundations, 35 (2), pp.107-110.

Mejia, L.H. and Dawson, E.M., 2006. Earthquake Deconvolution for FLAC, In: Proceedings of the Fourth International FLAC Symposium on Numerical Modeling in Geomechanics. Madrid, Spain.

Petersen M. D., Moschetti, M. P., Powers, P. M., Mueller, C. S., Haller, K. M., Frankel, A. D., Zeng, Y, Rezaelian, S., Harmsen, S. C., Boyd, O. S., Field, N, Chen, R., Rukstales, K. S., Luco, N, Wheeler, R. L., Williams, R. A., and Olsen, A. H., 2014. Documentation for the 2014 Update of the United States National Seismic Hazard Maps. U.S. Geological Survey Open-File Report 2014-1091.

Rogers, J.D. and Figuers, H., 1991. Engineering Geologic Site Characterization of the Greater Oakland-Alameda Area, Alameda and San Francisco Counties, California. NSF Grant No. BCS-9003785.

Wells, D.L. and Coppersmith, K.J., 1994. New Empirical Relationships among Magnitude, Rupture Length, Rupture Width, Rupture Area, and Surface Displacement. Bulletin of the Seismological Society of America, 84 (4), pp. 974-1002.

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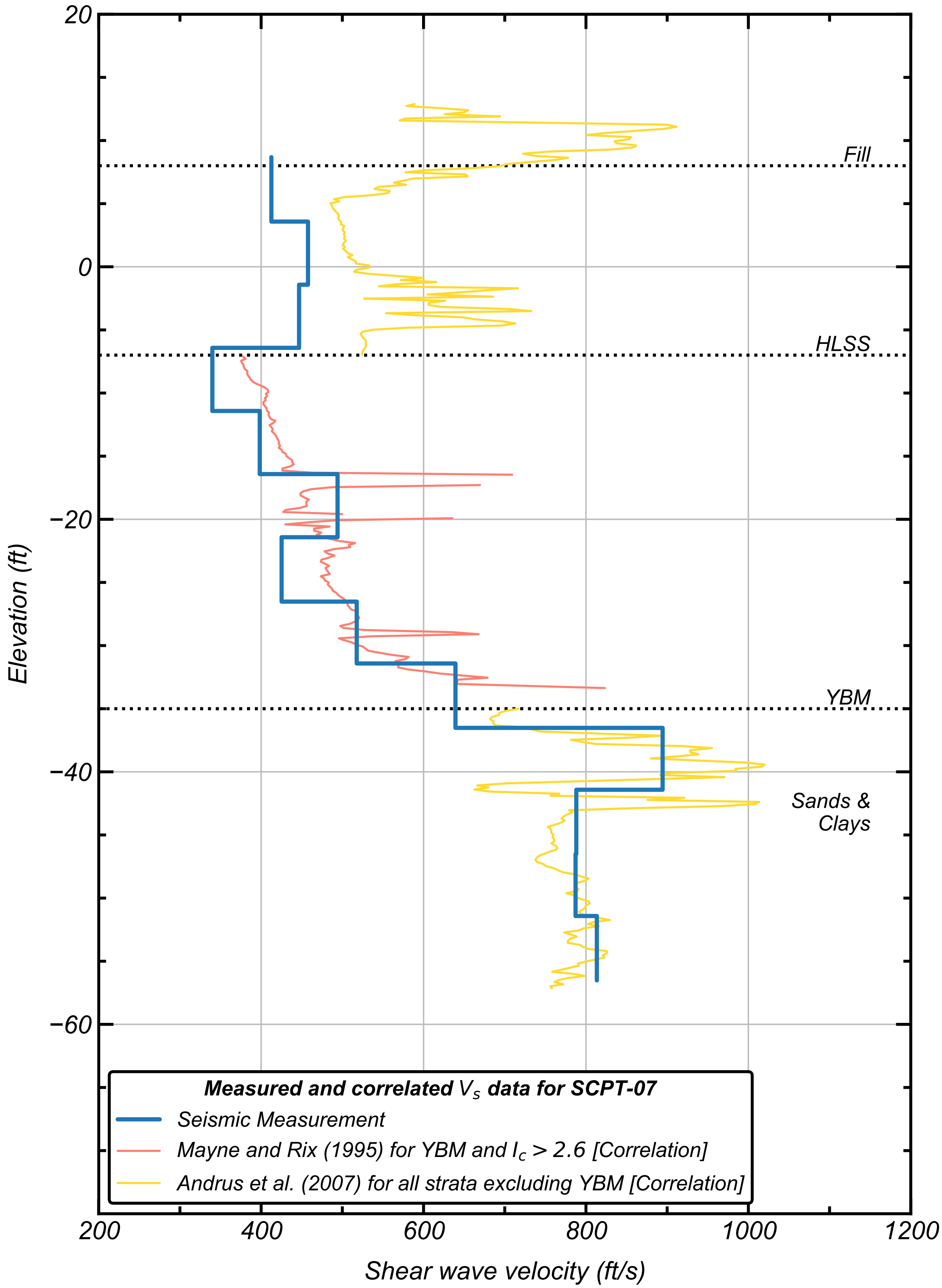

Figure G.2-1: Measured and Correlated Vs Data for SCPT-07

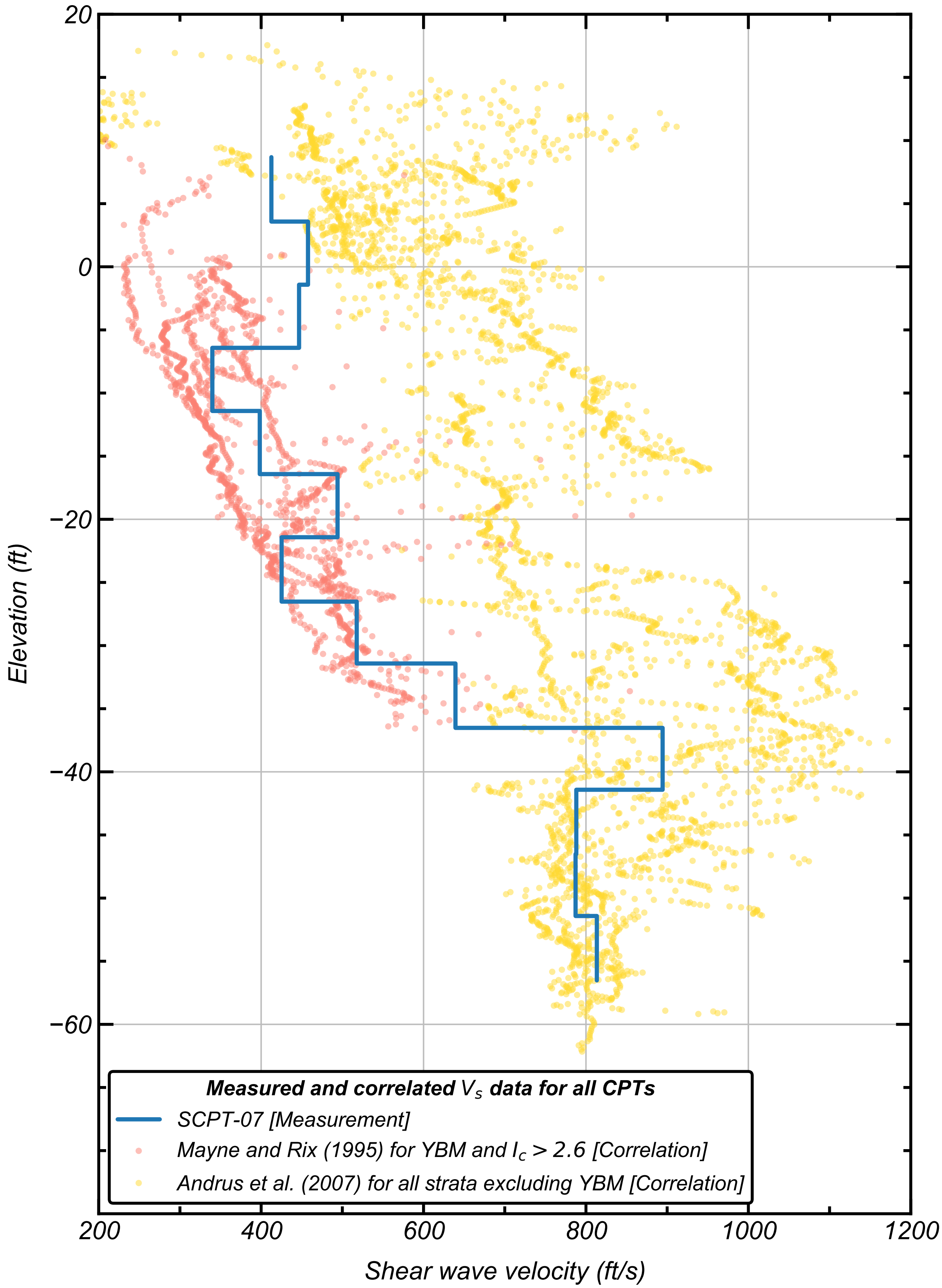

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Figure G.2-2: Measured and Correlated Vs Data for All CPTs

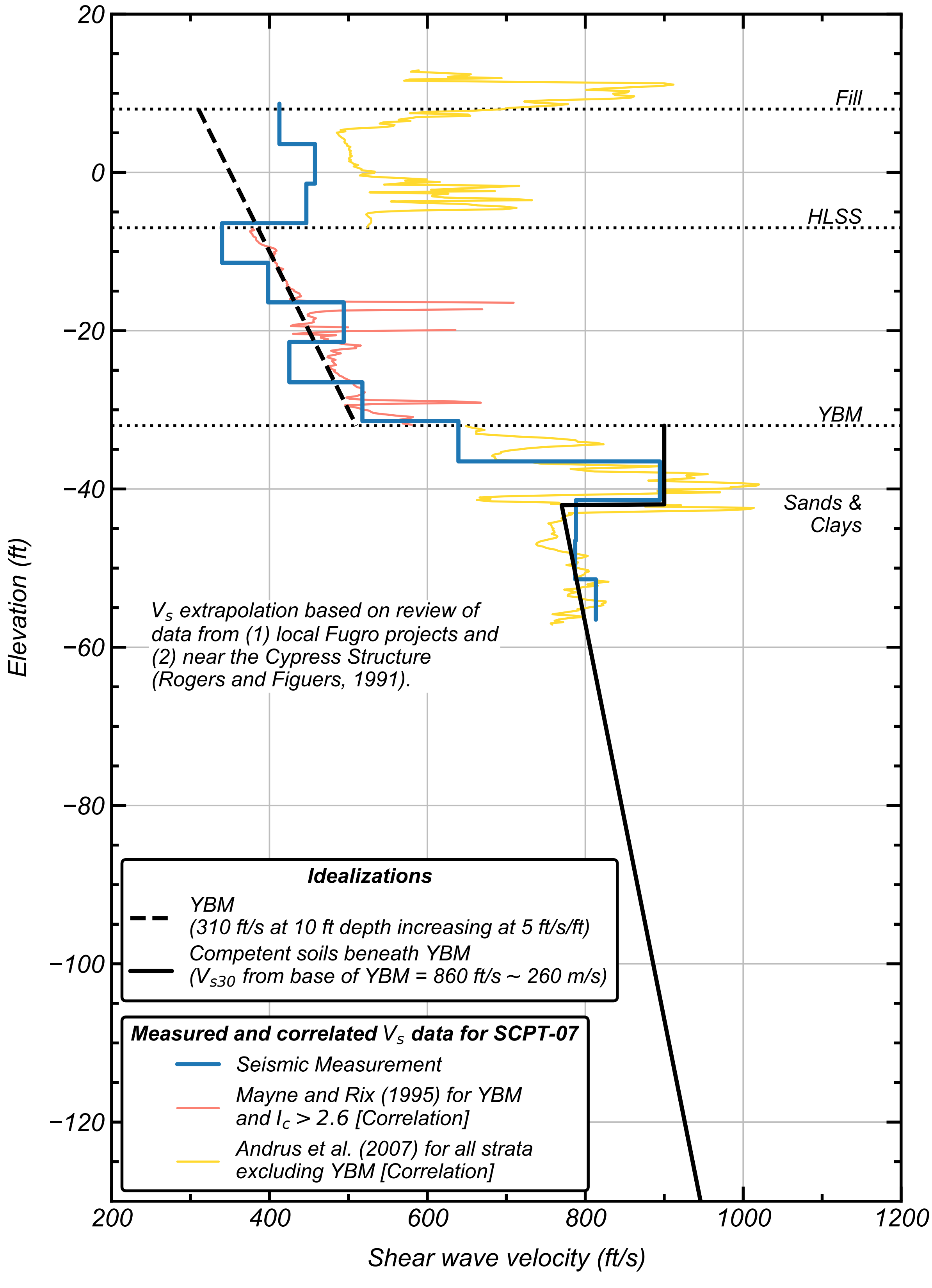
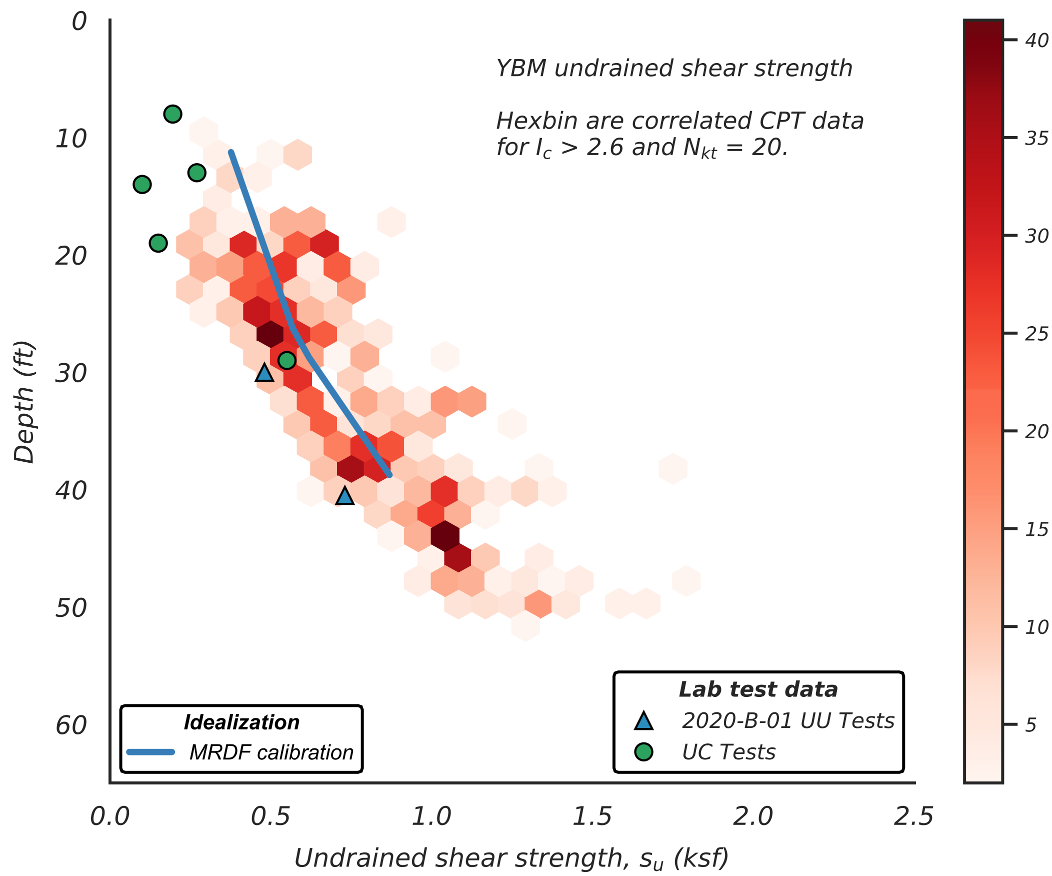


Figure G.2-3: Shear Wave Velocity Idealizations



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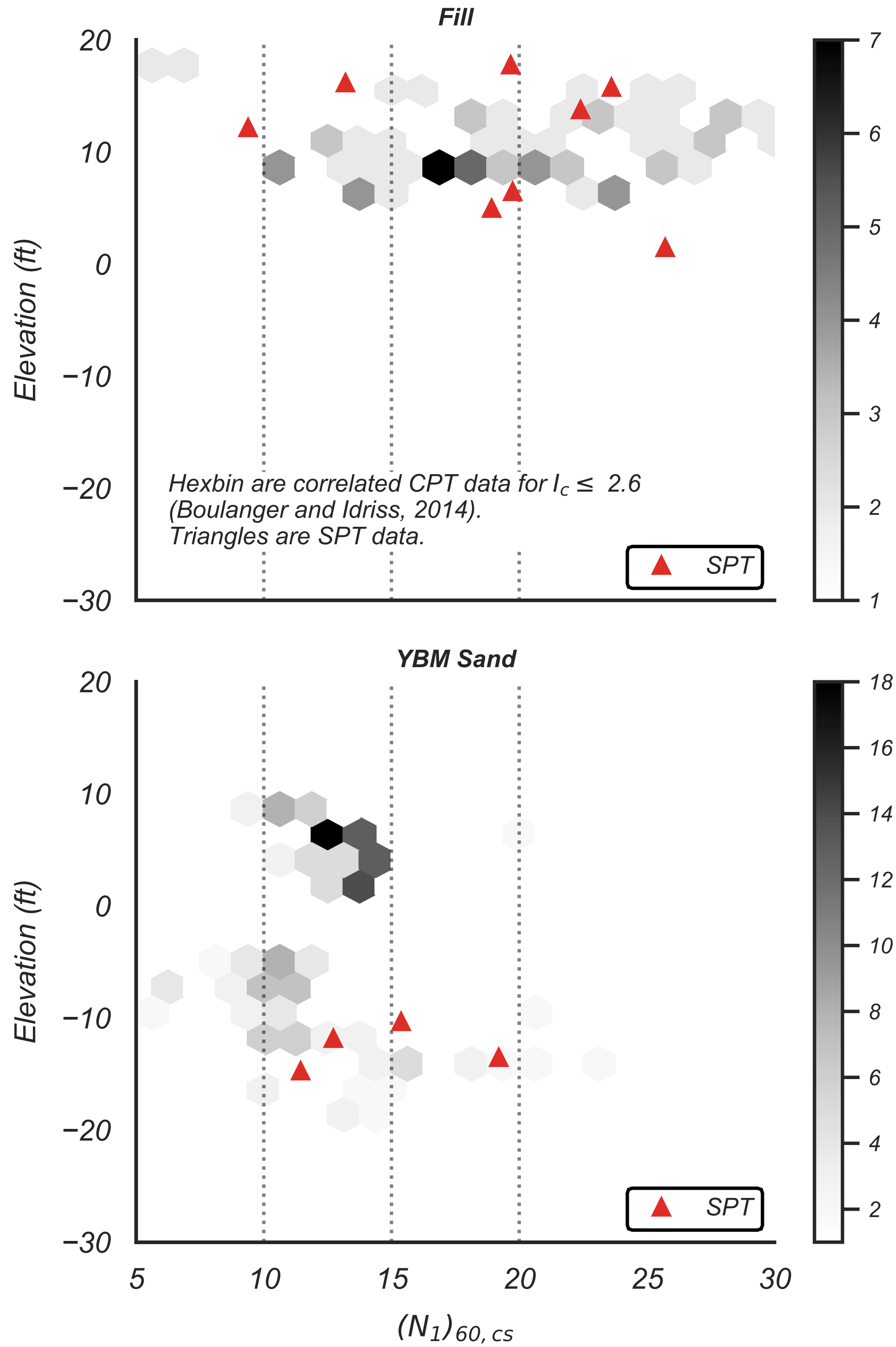
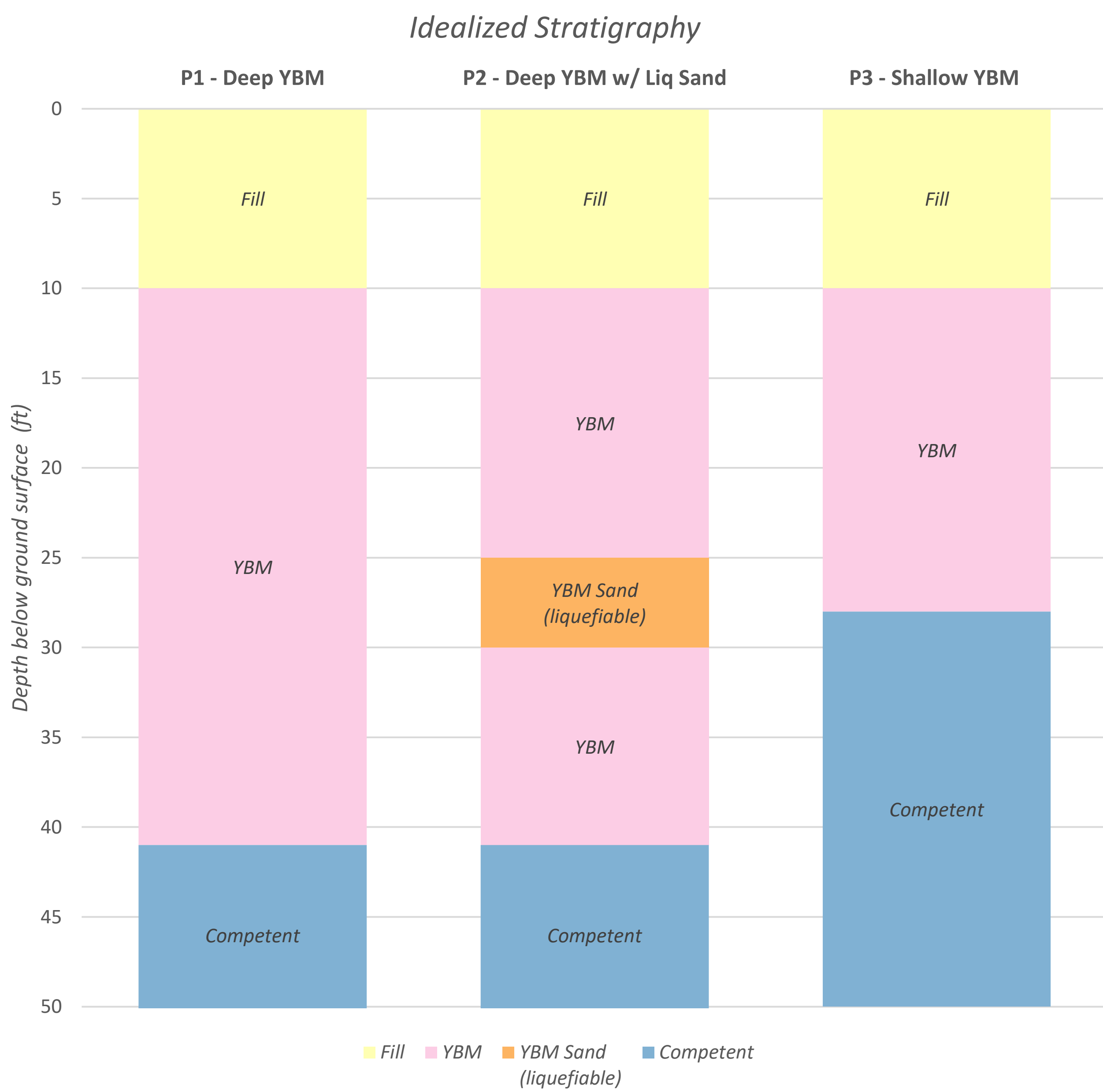


Figure G.2-5: Penetration Resistance (N160cs) vs. Elevation for Fill and YBM Sand



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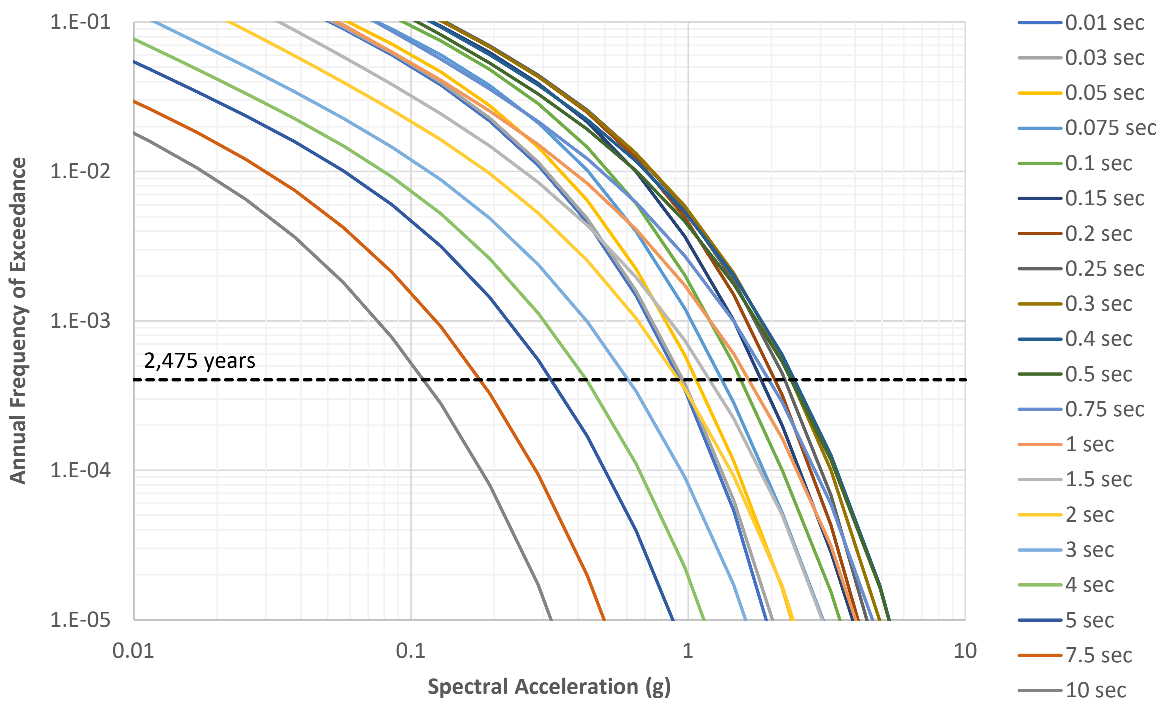

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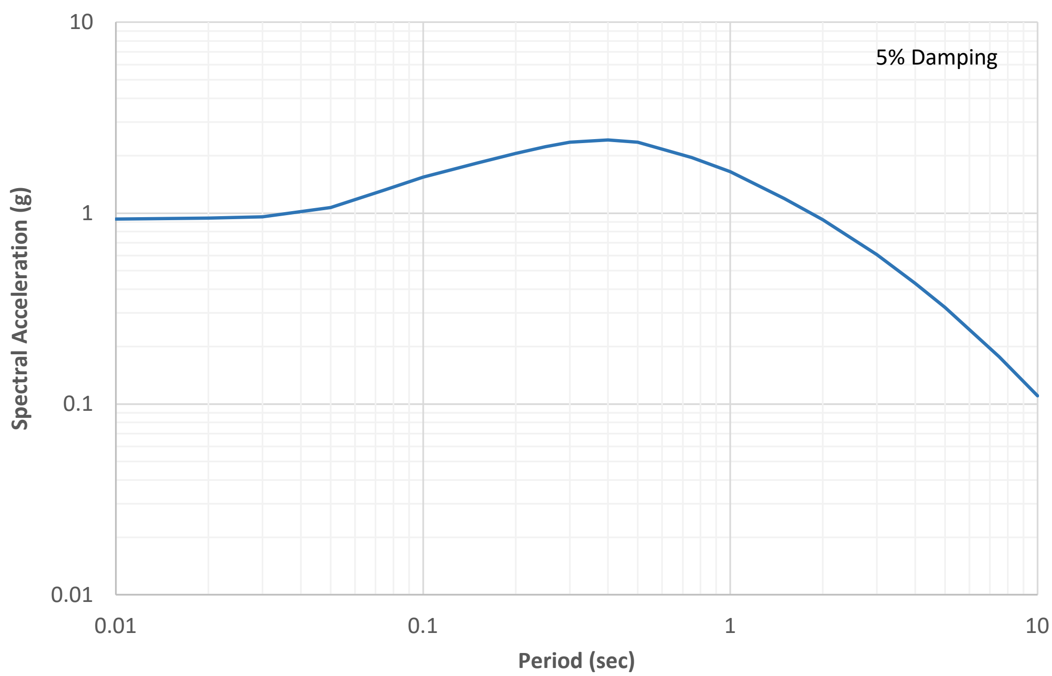

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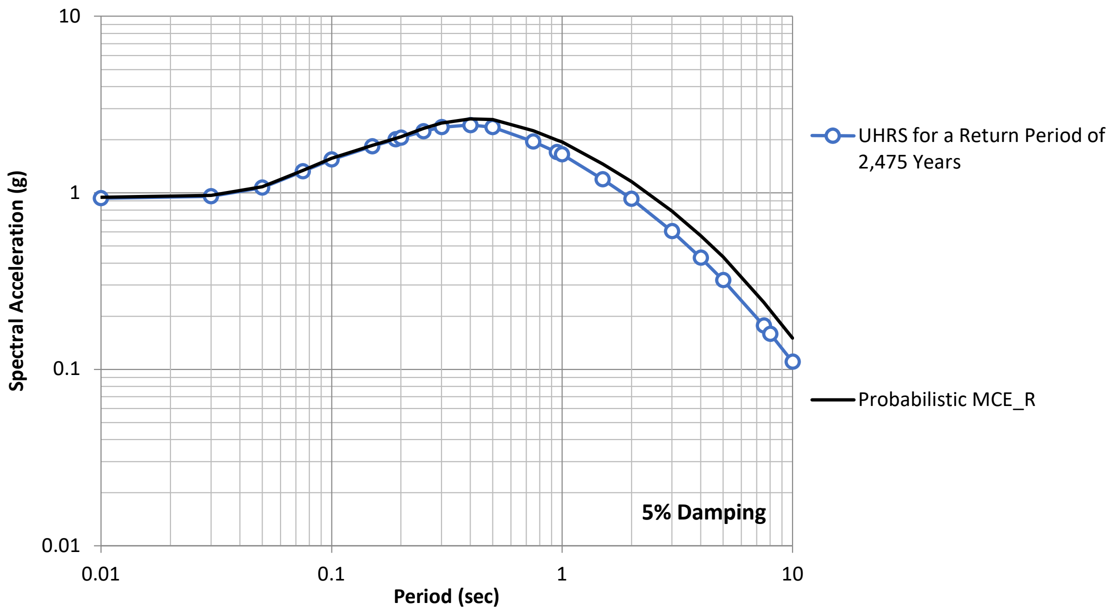

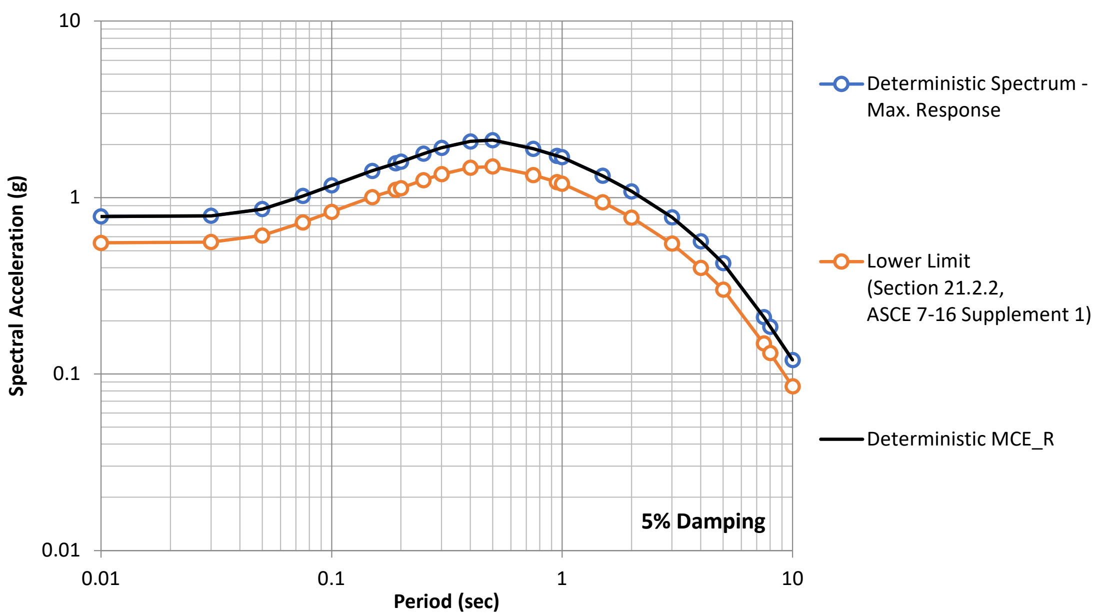

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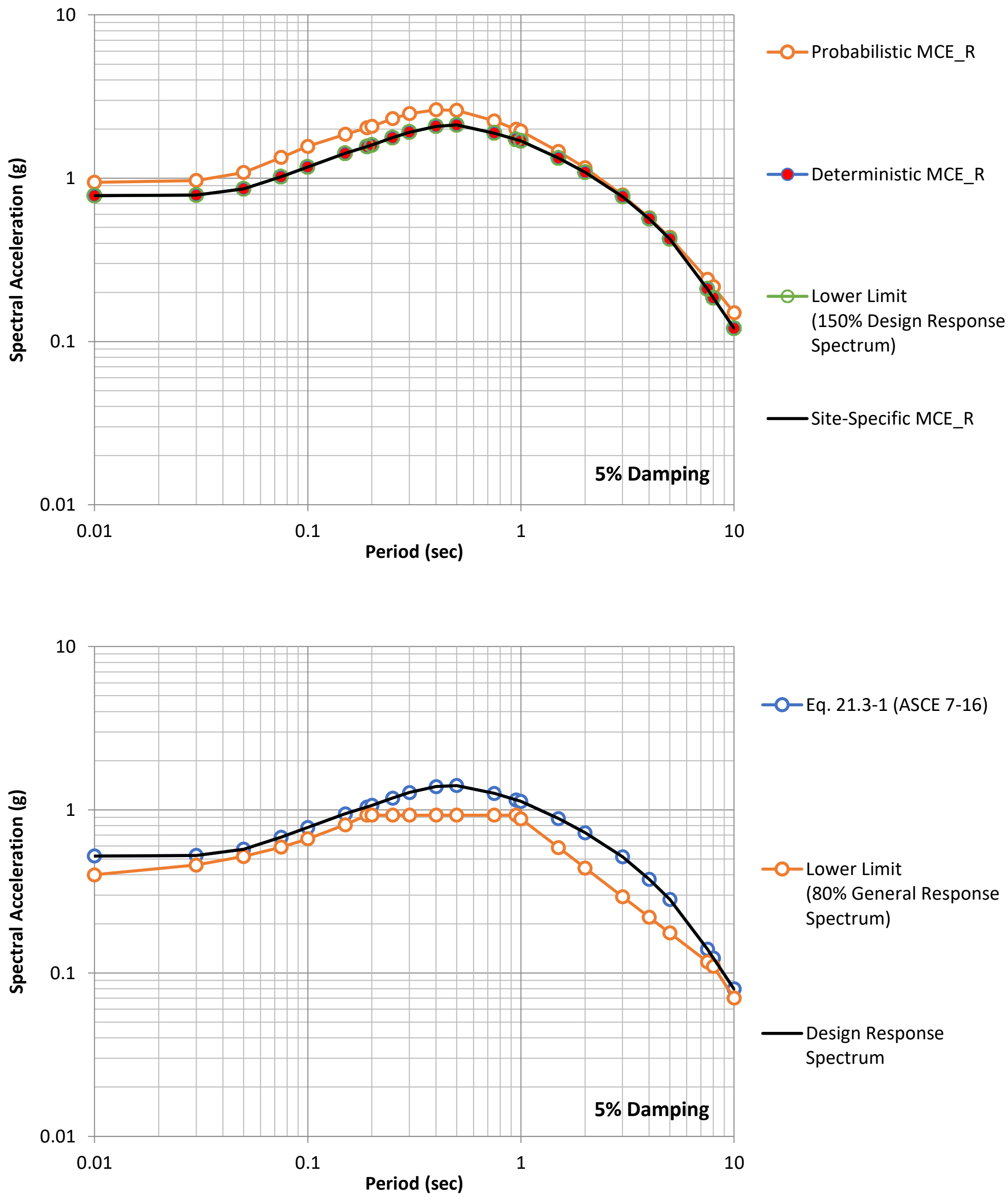

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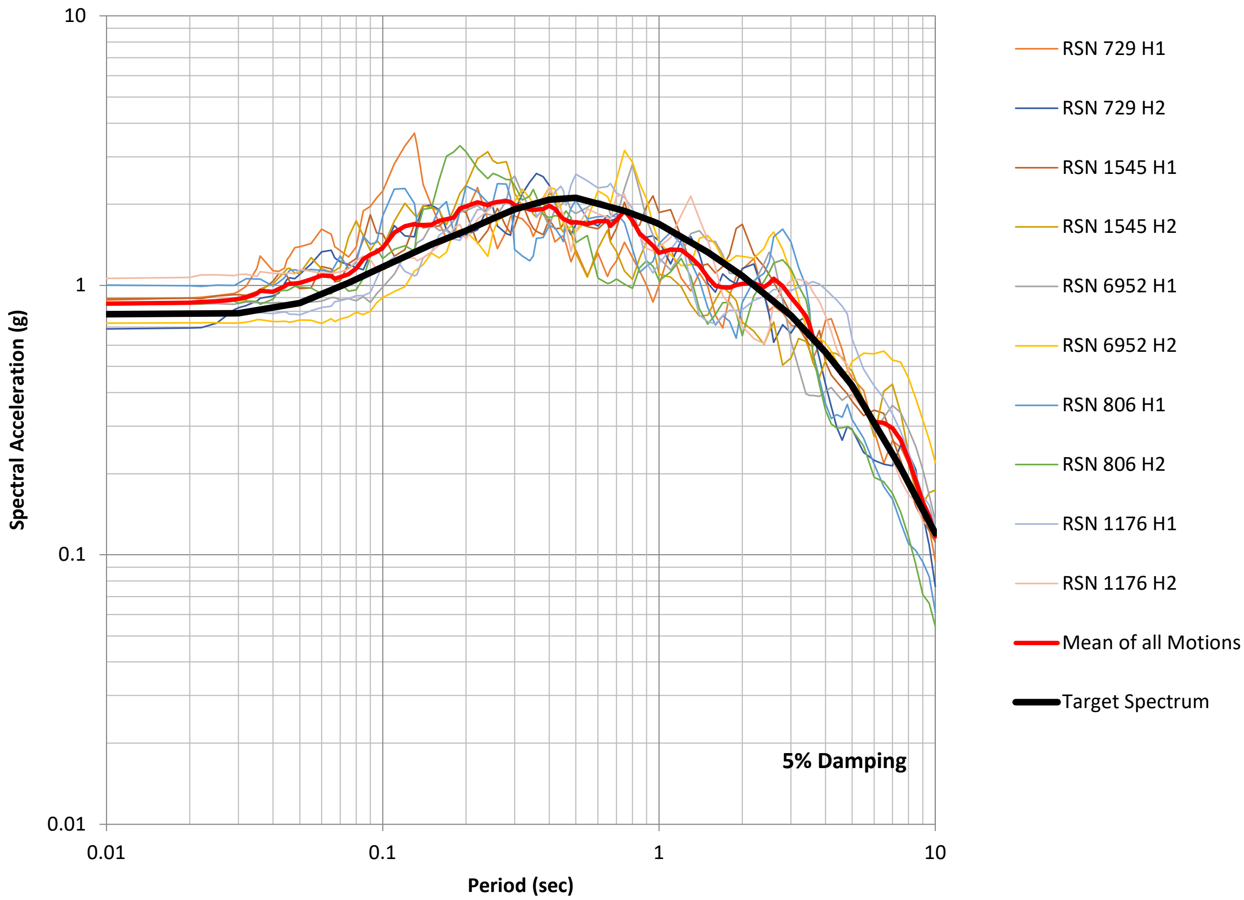

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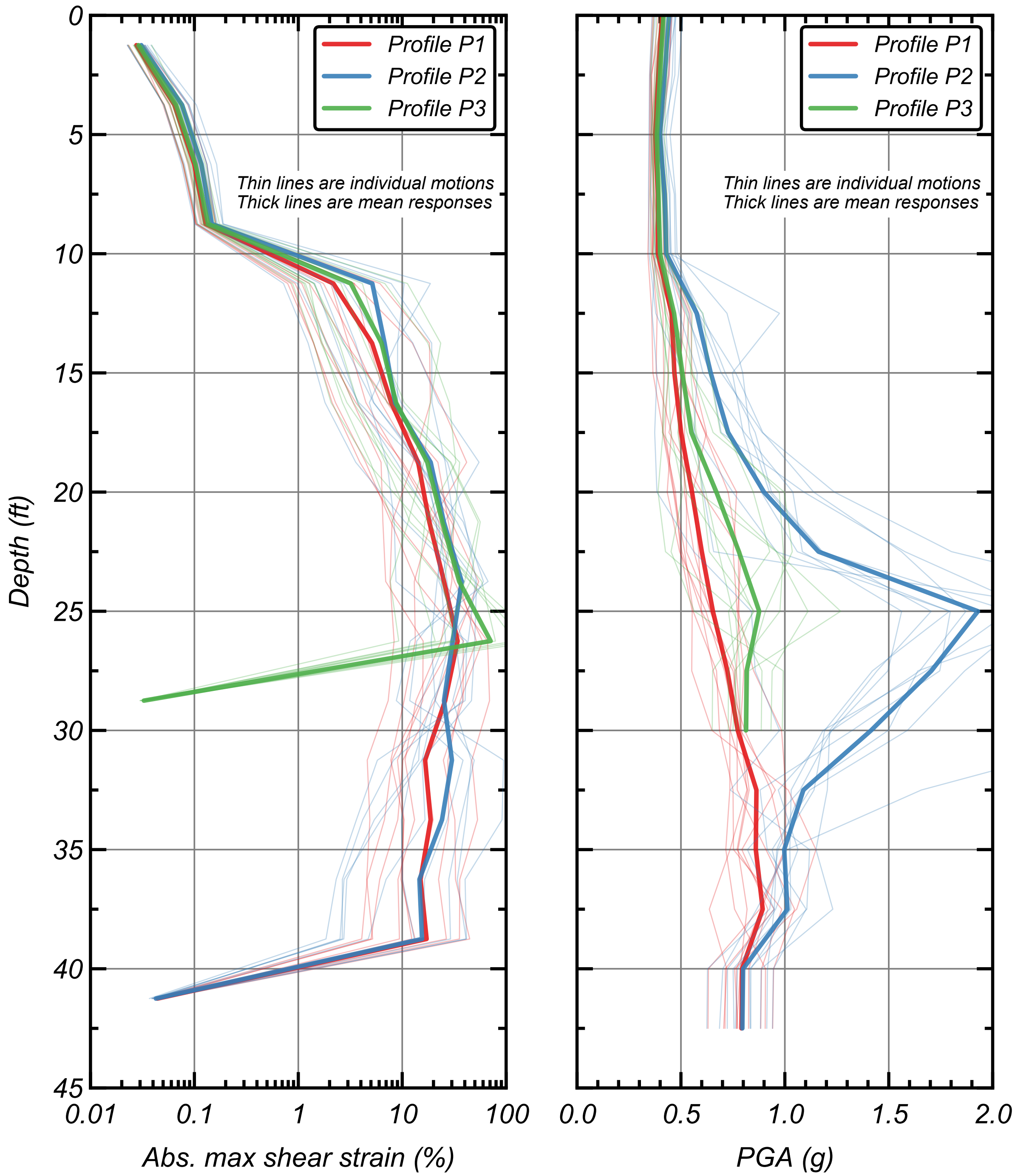

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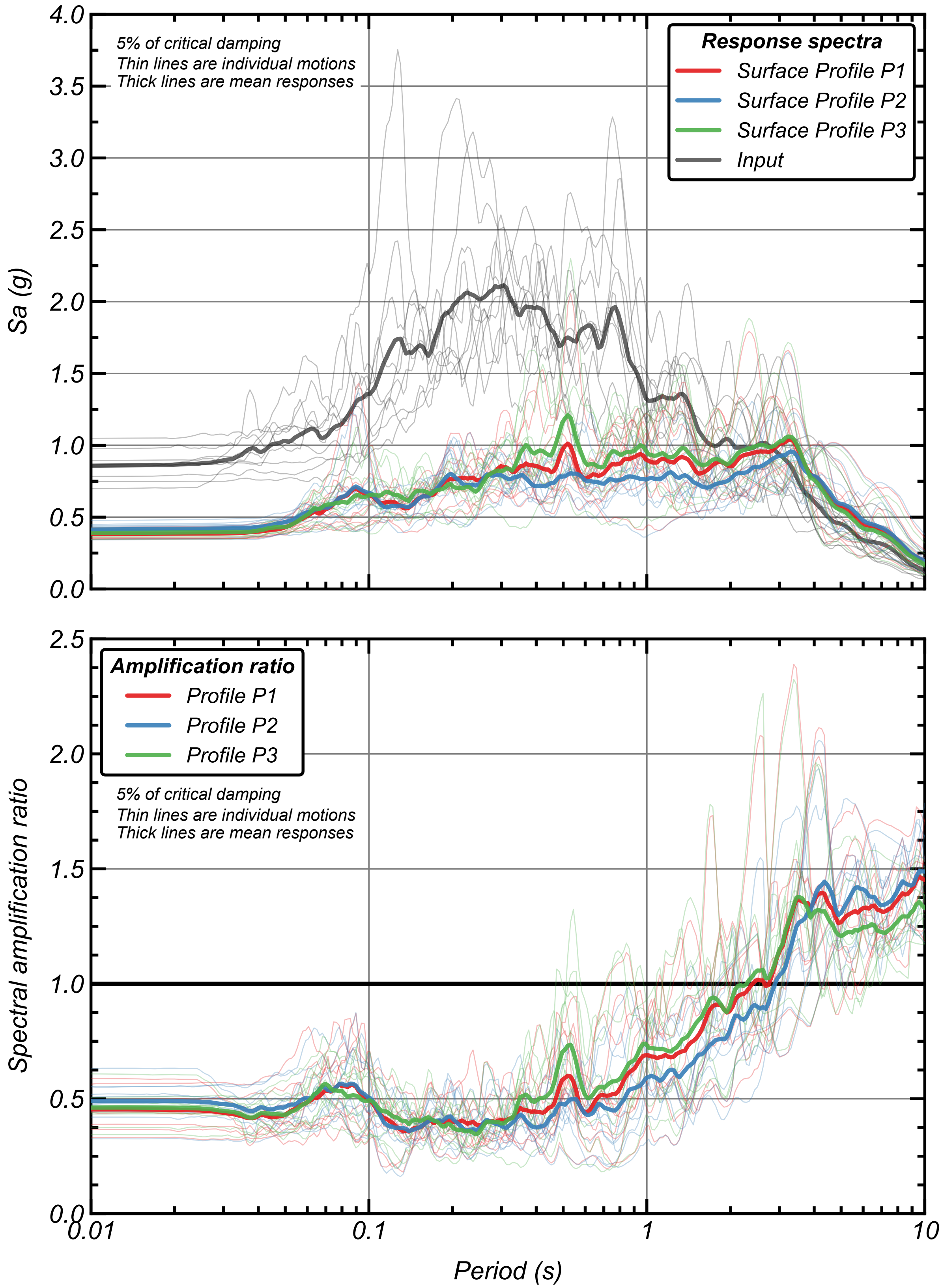


Figure G.6-2: Surface Response Spectra and Amplification Ratios from 1D Site Response Analyses

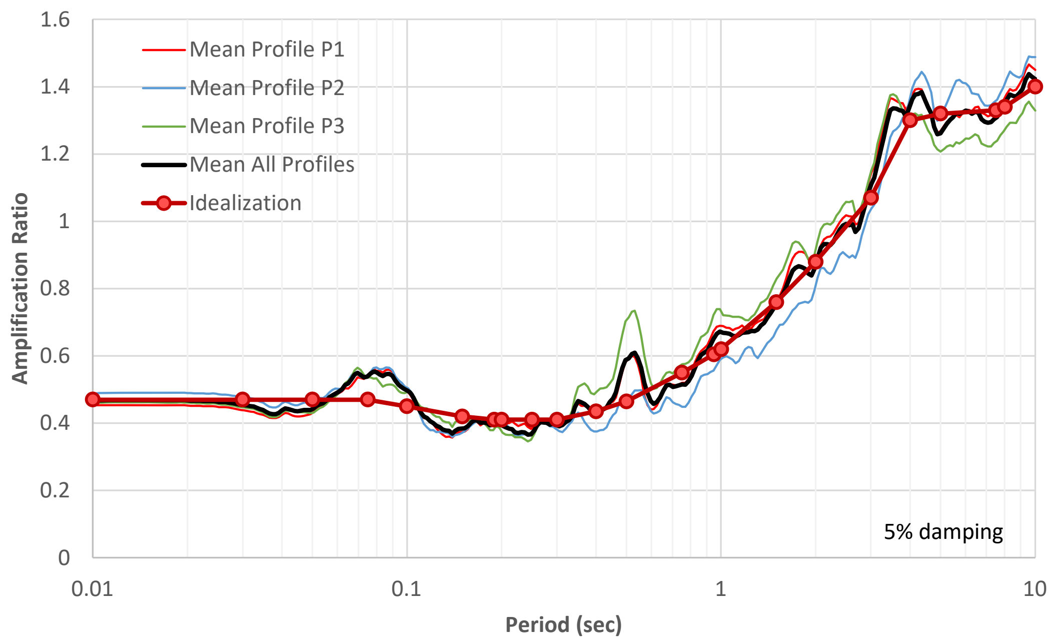
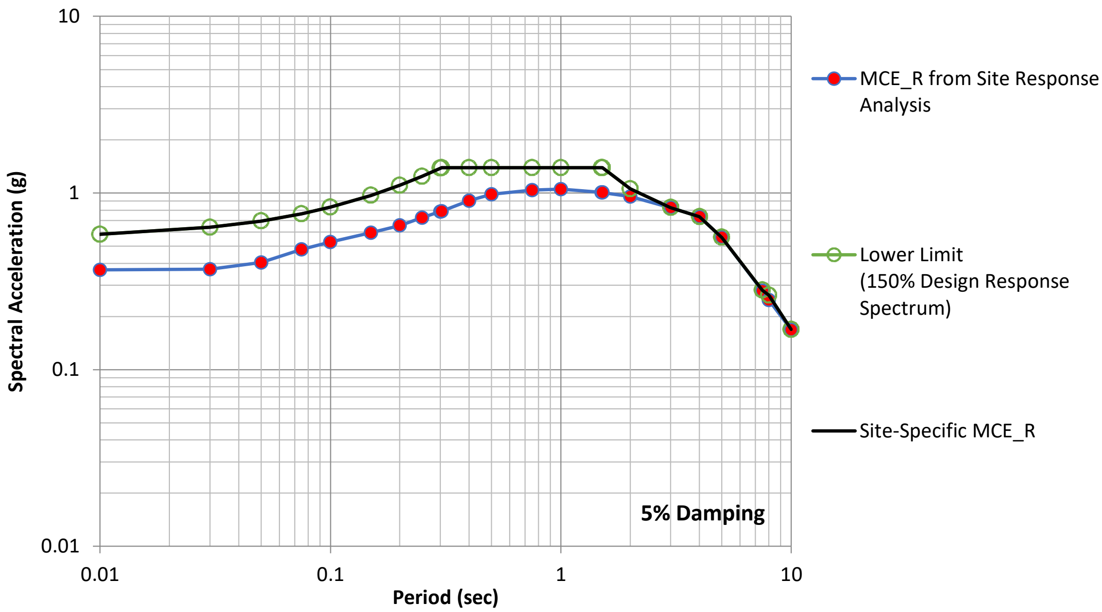


Figure G.6-3: Idealized Amplification Ratios



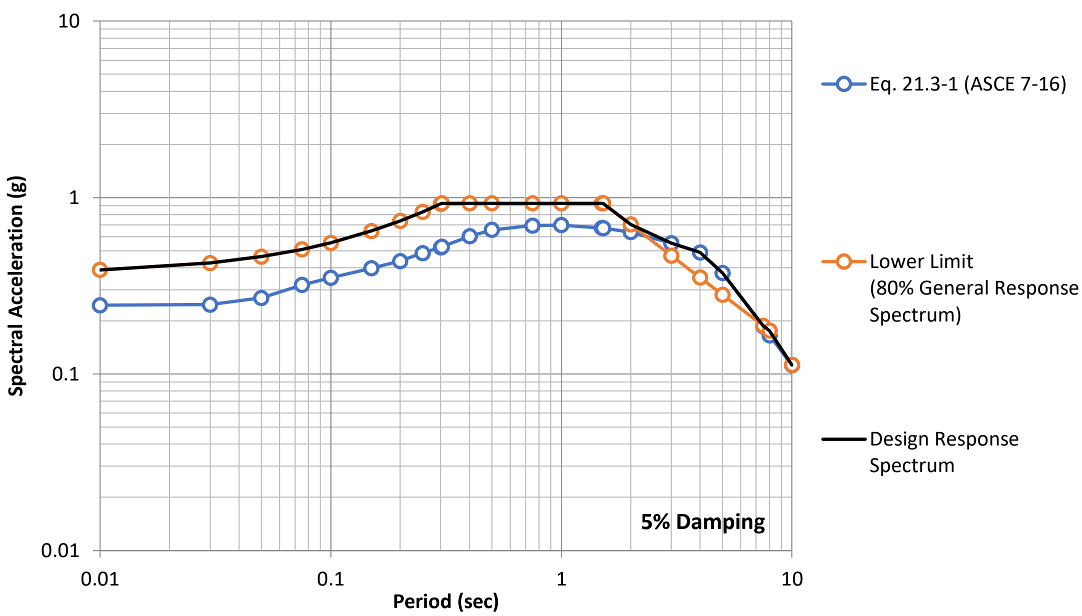


Figure G.7-1: Calculation of the Site-Specific Horizontal MCE_R and Design Response Spectra per ASCE 7-16 for the Ground Surface

Supplement H

LPILE Analyses

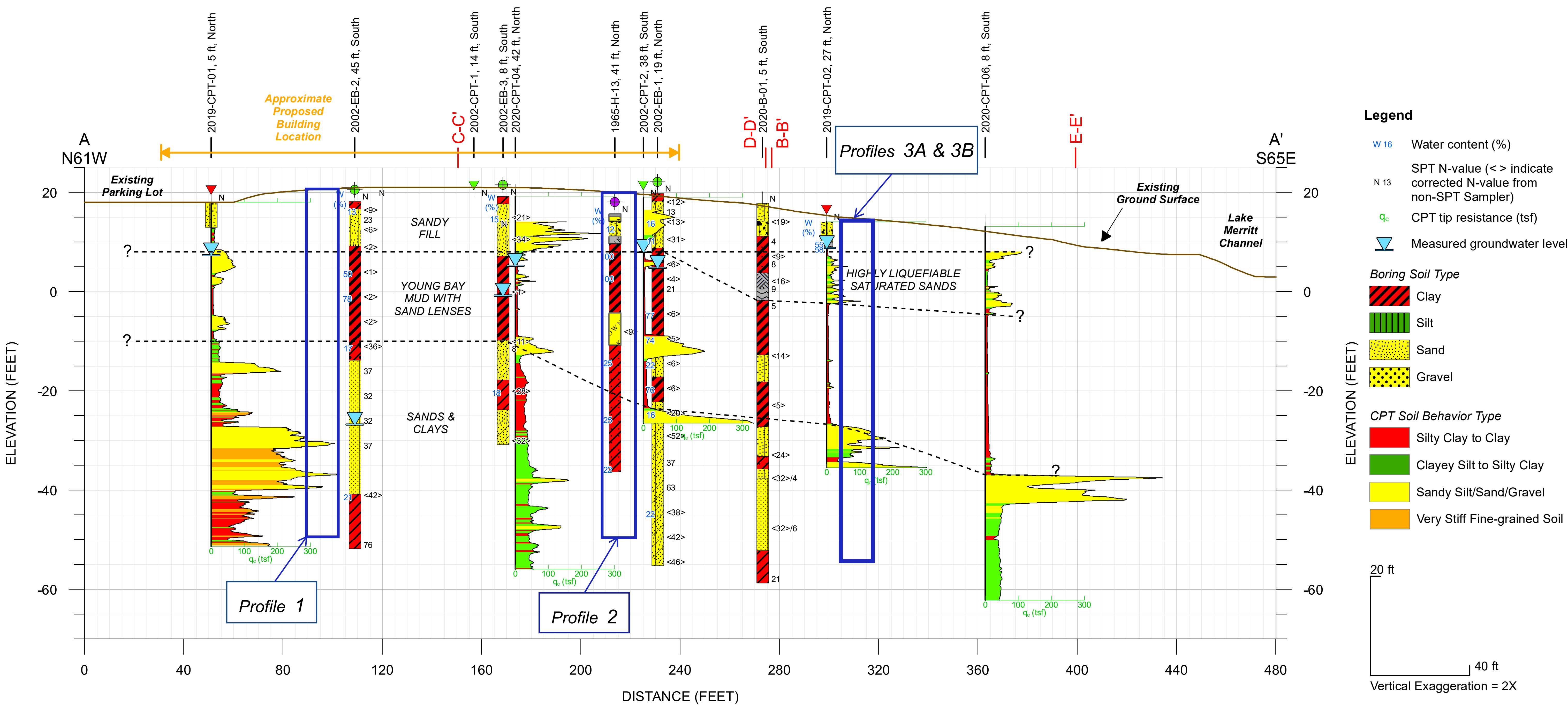


PLATE G-1: Profiles Along Cross Section A-A′

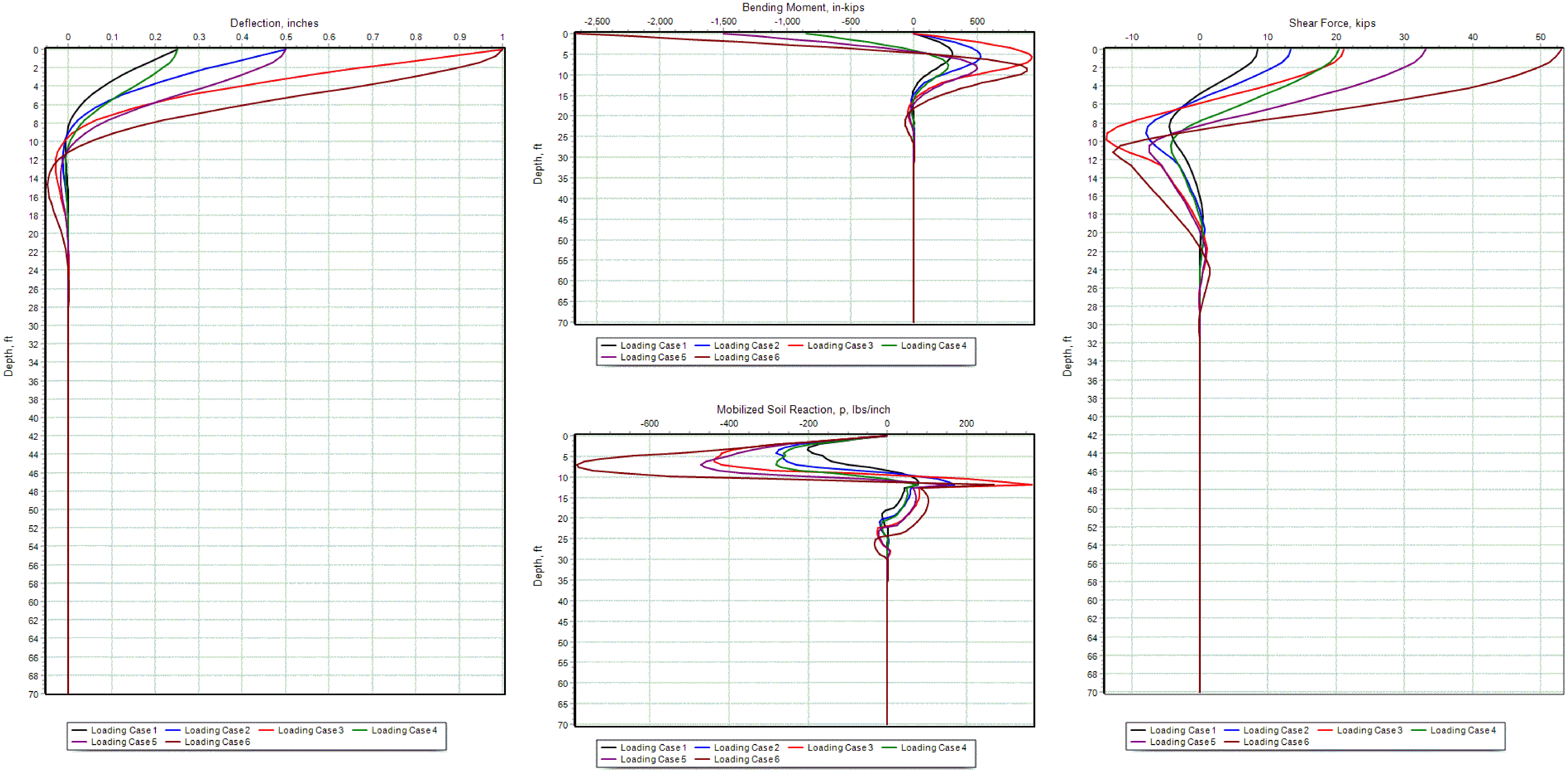


PLATE G-2: LPILE Results for Profile 1

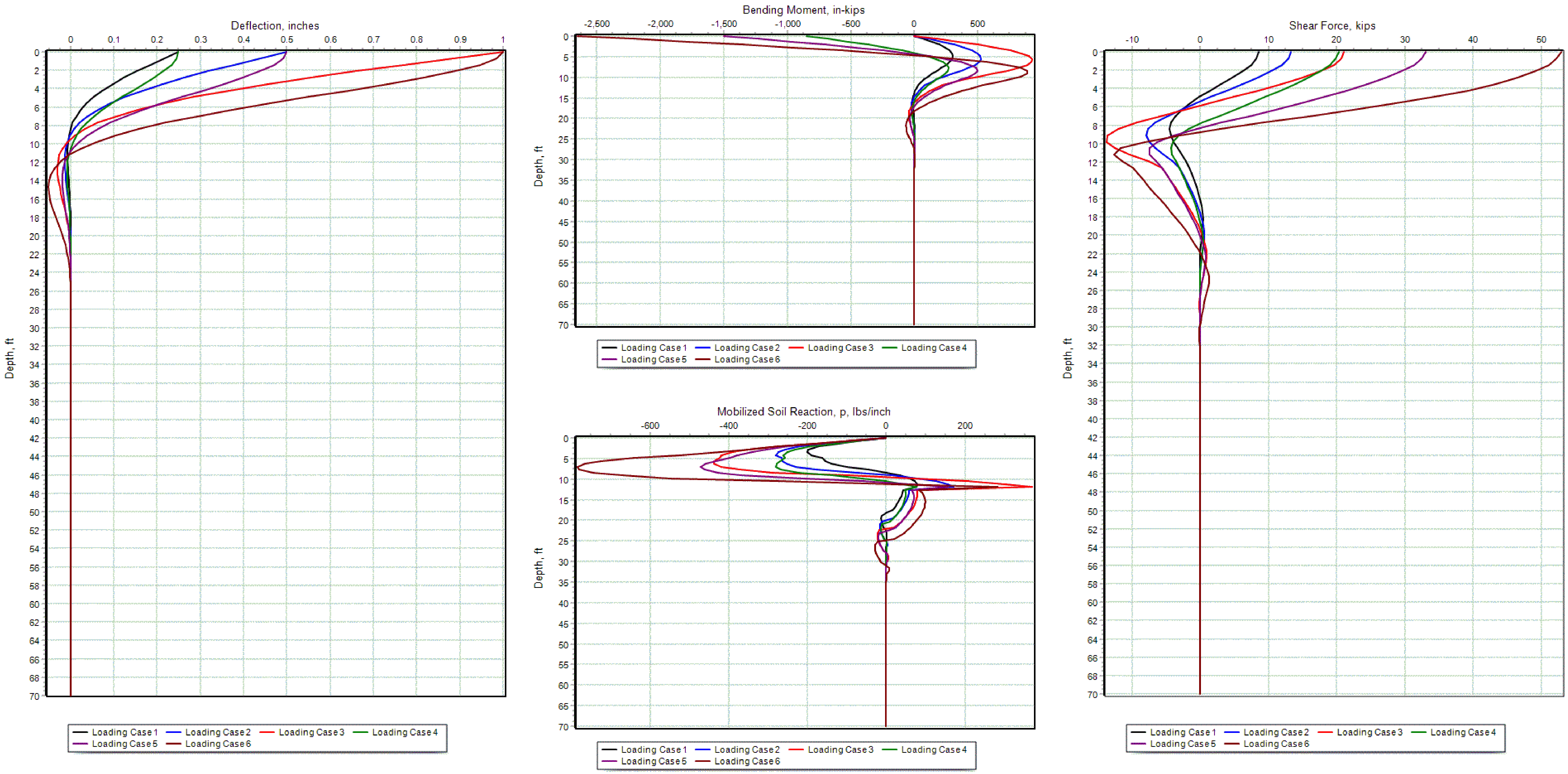


PLATE G-3: LPILE Results for Profile 2

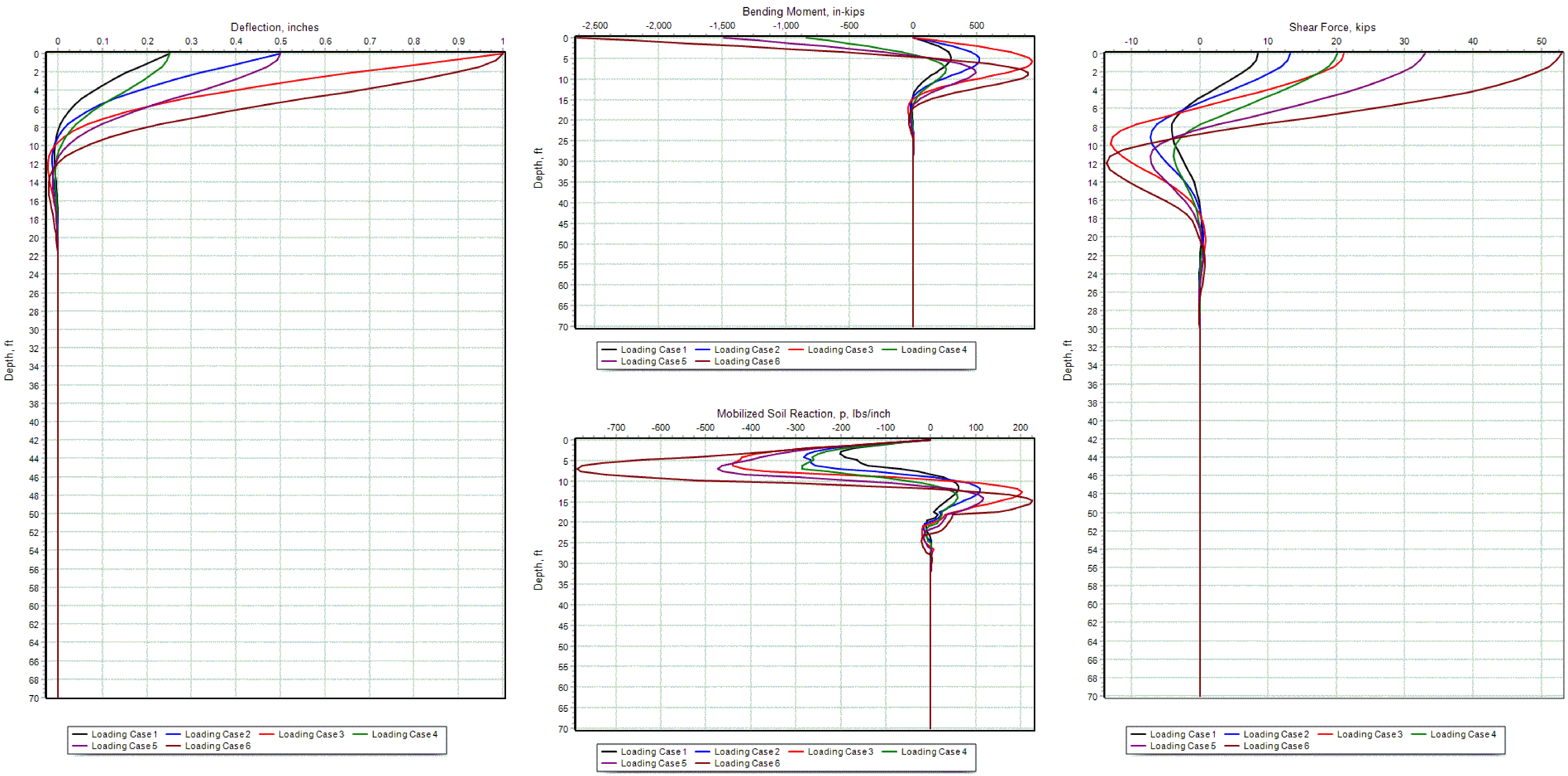


PLATE G-4: LPILE Results for Profile 3A

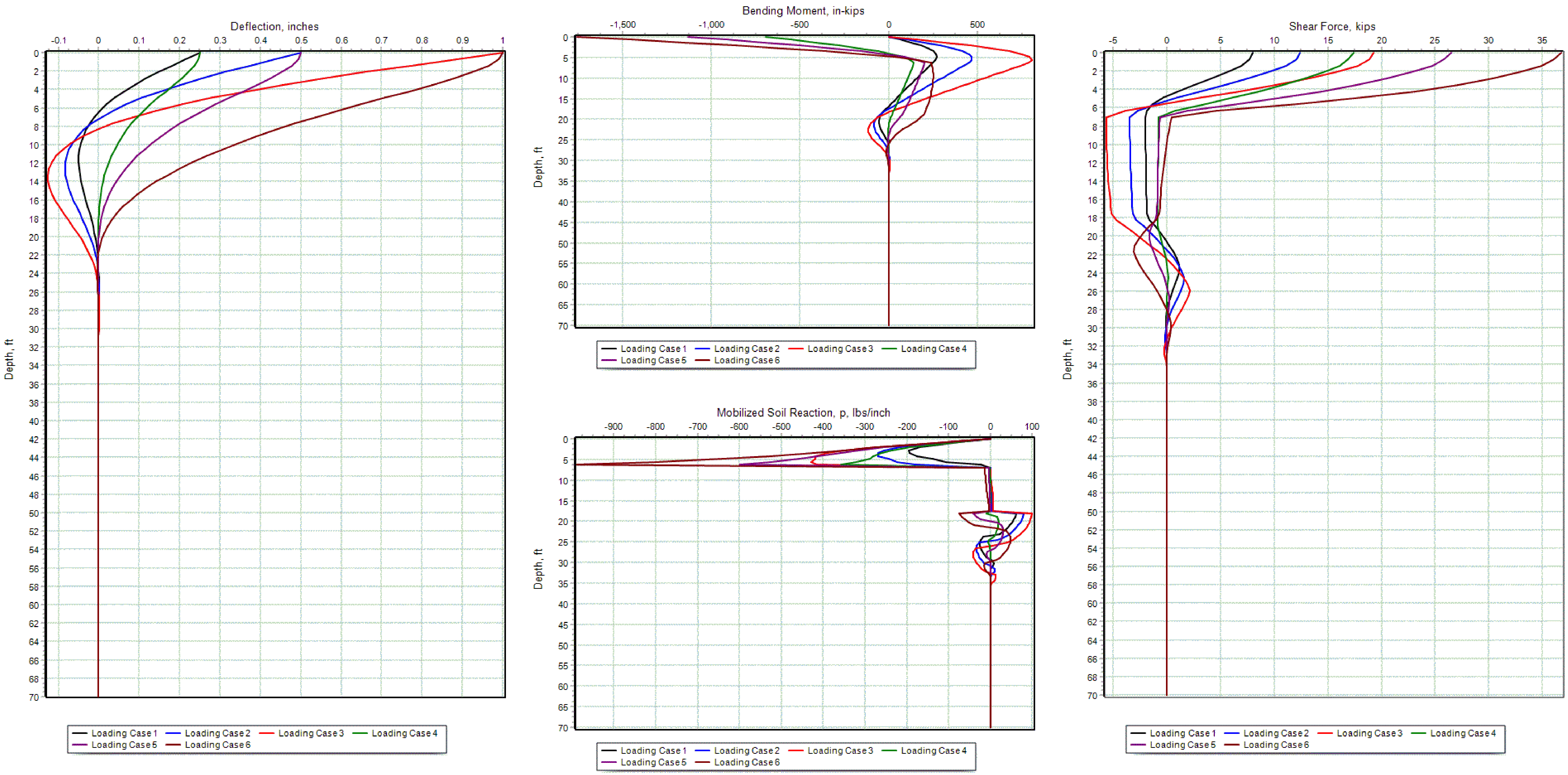


PLATE G-5: LPILE Results for Profile 3B

Supplement I

DMM Design and

Recommendations Report

I.1 Introduction

Liquefaction and seismic slope stability analyses performed during project geotechnical investigation and geologic hazards evaluation, indicated potential for significant lateral spreading (up to several feet) and liquefaction-induced settlements (generally 1 to 4 inches and up to about 6 inches closer to the Lake Merritt Channel). The loose to medium dense sand layers of various thicknesses located both above and within the Young Bay Mud layer have a high potential for liquefying when subjected to an MCE (Maximum Considered Earthquake) event. These sand layers were encountered within depths of about 30 to 40 feet (above elevation -15 feet). The seismic slope stability at the planned building location is affected by both liquefaction and the presence of relatively soft Young Bay Mud (YBM). For further description of the seismic slope stability and liquefaction hazards refer to the project geotechnical report (Fugro, 2020).

It has been decided that the foundation soil will be improved using Deep Mixing Method (DMM) columns and grids under the entire footprint of the Laney College Library & Learning Resource Center (LLRC) building, which will be supported by shallow foundations (e.g., footings, and structural slab, and grade beams). This Addendum presents our methodology for the DMM design, provide DMM specifications, and to provide geotechnical design parameters for the design of the LLRC structure and its foundation system.

I.2 Proposed Structure

The location of the LLRC building is shown on **Plate 1**. The proposed new structure will be constructed at approximately the existing grades without basements. The footprint of the LLRC building is approximately 23,750 square feet. This building will be supported by shallow spread footings with interior structural first floor slabs and grade beams.

I.3 Subsurface Conditions

The subsurface soils below the site generally consist of predominately medium dense sandy fills that extend to depths of about 8 to 25 feet (Elevations of about +8 feet to -5 feet). Clayey fills of about 2 to 4 feet thick were also encountered in some areas. These fills are heterogenous and locally contain various amounts of concrete, brick, and wood debris. Most of these fills appear to be derived from the historical filling of the natural Lake Merritt outlet channel between 1860s and 1940s, and the later development of the Laney College campus in 1960s. Most likely these fills were not compacted to current acceptable geotechnical engineering standards.

Below the surficial fill layer, very soft to soft, high moisture content, and low shear strength Young Bay Mud was encountered to a depth of about 30 feet (Elevation of about -10 feet) at the northwest side of the proposed building location and about 50 feet (Elevation of about -30 feet) at the southeast side of the proposed building location. Some thin loose to medium dense sand lenses about 2 to 6 feet thick were also encountered within the Young Bay Mud layer. About 15-

feet of loose to medium dense sands were also encountered between the surficial fill and the Young Bay Mud layers at the east edge of the building, extending towards the channel. These sands could be either historical fills placed in the natural Lake Merritt outlet channel or natural sand deposits that existed within the channel.

I.4 DMM Ground Improvement

I.4.1 Purpose

Several alternatives were considered for mitigating the lateral spread hazard at the planned building site, including installation of a retaining wall and the deep mixing method (DMM) beneath the building footprint. Considering the high seismic demand, presence of shallow liquefiable soils and soft Young Bay Mud, proximity to the Lake Merritt Channel, and constraints from the PG&E easement on the north side of the planned building, it is our experience and opinion that continuous grids of deep mixed shear walls are the most suitable, robust, and cost-effective technique to mitigate the lateral spread hazard at the planned building site. The grids of deep mixed shear walls will provide support for shallow foundation systems for seismic loading and transfer bearing loads deeper to the medium dense to very dense sands and stiff to hard clays, reducing total and differential building settlements. In addition, we recommend using structural slabs to span between DMM deep mixed shear walls, assuming that the untreated soils within the grid walls may still develop post-liquefaction reconsolidation settlements below slabs. The deep mixed shear walls will also affect the composite ground response to horizontal ground motions. This section presents a brief overview of the deep mixing method (DMM), our design approach, DMM design properties, and results of our evaluation process, including results of seismic stability analyses. Seismic design parameters incorporating the composite response of the deep mixed zone are presented in **Section I.5**, herein.

I.4.2 Description of the Deep Mixing Method

The deep mixing method (DMM) is a soil improvement technique used to treat soils in place without excavation or dewatering. A rig that is typically equipped with multi-shaft mixing augers (containing auger flights and mixing paddles) is used to inject a cementitious grout and blend it with the in-situ soils. When the design depth is reached, the augers are withdrawn while mixing on the way to the surface, leaving in-place a stabilized soil mass that is stronger, less permeable, and has improved engineering properties. A multi-shaft mixing rig creates interconnected soil mixed elements formed by partially overlapping columns. The elements can be arranged to form walls, grids, and blocks of deep mixed soil-cement. There are various diameters of multi-shaft mixing augers and they typically range from 3 to 5 feet.

While there are other methods of creating deep mixed grids, such as by Cutter Soil Mixing (CSM) or Trench Cutting and Remixing (TRD), we believe multi-shaft auger systems will be most efficient for creating the deep mixed grids, and there are several contractors locally who have such systems. We do not recommend using single shaft soil mixing equipment for creating deep mixed shear walls and grids because, in our experience, uniform mixing is more difficult to control when using single shaft soil mixing equipment.

The body of literature (case histories, numerical simulations, and physical model tests) on the effectiveness of DMM grids for mitigating liquefaction effects demonstrates that grid configurations are more effective than columns, with benefits including reduced ground settlements and lateral spread displacements, reduced earthquake-induced shear stress and strain within untreated soils bounded by the grid walls, containment of liquefied soils within the grid walls if liquefaction occurs, and reduced migration of excess pore pressure between unimproved and improved zones (Namikawa et al., 2007; Siddharthan & Porbaha, 2008; Nguyen et al., 2013; Yamashita et al., 2015; Tsukuni & Uchida, 2015 and 2017; Boulanger et al, 2018; Boulanger & Shao, 2021). The effectiveness of DMM grids to mitigate liquefaction-induced displacements depends on a variety of factors, including the treatment geometry and area replacement ratio (A_r) and deep mixed ground strength and stiffness. The area replacement ratio is defined as the ratio of the surface area of treated soil-cement to the total surface area within a given treatment zone. As area replacement ratio increases, the composite shear strength of the deep mixed zone increases, and earthquake-induced shear stresses decrease.

I.4.3 Design Approach

Design of the DMM ground improvement generally follows the approach described by the FHWA guidelines (Bruce et al., 2013) and involves the following steps:

1. Establish trial geometry (area replacement ratio, column diameter, and shear wall spacing) and deep mixed ground properties,
2. Evaluate global slope stability (static, seismic, and post-seismic),
3. Evaluate other potential external modes of failure of the deep mixed zone (overturning and bearing)
4. Evaluate internal stability of the deep mixed zone (racking failure, crushing of deep mixed shear walls at the outside toe),
5. Evaluate static and seismic settlements,
6. Repeat steps 1-5 until performance is satisfactory,
7. Evaluate deep mixed column bearing capacity for support of structural loads, and
8. Refine layout and add additional deep mixed columns to reduce floor free span distance, where appropriate, to reduce floor slab costs.

The following sections describe the evaluation process for the key steps (1 through 5) listed above.

I.4.4 DMM Design Properties

DMM design properties and geometries were initially selected based on rule of thumb and review of relevant case histories and published design guidelines and technical papers on the subject. Following the design approach presented above, DMM properties and geometries were iteratively adjusted to achieve acceptable performance. The final DMM properties used for design are summarized in the **Table I.4.1**.

Table I.4.1: DMM Design Properties

Parameter	Design Value
Unconfined Compressive Strength ($q_{dm,spec}$)	125 psi
Shear Strength (s_{dm})	74 psi (curing time = 365 days) (40% of unconfined compressive strength)
Young's Modulus (secant modulus at 50% mobilized strength; E_{50})	37,500 psi (300 times unconfined compressive strength)
Shear Modulus Ratio ($G_r = G_{dm}/ G_{soil}$)	5-20
*dm = deep mixed	

I.4.4.1 Area Replacement Ratio

An area replacement ratio (Ar) of 50 percent was selected for design to limit lateral spread displacement to an acceptable magnitude. The basis for tolerable lateral spread displacement and our evaluation of seismic slope displacement are presented in the following section. An area replacement ratio of 50 percent with conventional DMM strengths will also provide adequate support for all footings and moment frame grade beams. In addition, case histories indicate good performance against soil liquefaction hazards with A_r greater than approximately 20 percent.

For example, the 14-story International Hotel in Kobe Japan used an A_r of about 20 percent. During the Kobe earthquake, this structure performed very well despite having the ground surrounding the building liquefy, laterally spread several meters, and settle significantly. DMM grids of about 34 feet deep and with an A_r of about 30 percent were reportedly used for several two-story new school buildings at Jordan High School in Long Beach, California (completed in 2017) to reduce liquefaction-induced settlements and support the buildings on shallow foundation systems. A design 28-days DMM unconfined compressive strength of 150 psi was used at the Jordan High School project.

The West Dowling Road Overcrossing in Anchorage Alaska was built in 2014 with deep mixed shear walls and columns supporting the approach abutment footings in part to mitigate earthquake-induced lateral deformations within shallow soft peat and liquefiable silt layers (Boulanger & Shao, 2021). The deep mixed walls and columns were spaced to produce area replacement ratios of approximately 90 percent beneath the footings (shear walls with overlapping columns) and 50 percent in the area surrounding the footings (shear walls only). The bridge performed well in the 2018 Anchorage earthquake (M = 7.1 and PGA at the overcrossing estimated to be 0.35 to 0.45) with deformations kept to acceptable levels.

We have used the deep mixing ground improvement method and our latest experience was the Agnews Campus project located in San Jose, California. The project included sixteen one- to three-story building on an approximately 55-acre site. For that project, estimated liquefaction-induced ground surface settlements exceeded the project settlement design criteria for buildings and structures that were supported on shallow foundations without ground improvement. DMM grids of about 40 feet deep with an A_r of 40% and unconfined compressive strength of 250 psi was used for the ground DMM design.

Based on available literature, case histories, and the analyses presented in the following sections, we judge that a minimum A_r of 40 to 50 percent is reasonable for support of the planned Laney College LLRC building. The design calculations presented in the following sections are based on A_r = 50%.

I.4.5 Global Slope Stability Analyses

I.4.5.1 Seismic Slope Displacement

The methodology and results of seismic slope displacement evaluations for the DMM ground improvement are presented in this section. Based on results of the stability analysis and variable Young Bay Mud (YBM) thickness encountered at the site, we performed 10 additional CPTs on November 17, 18, and 22, 2022 to better define the bottom elevation of the YBM layer. The new cross sections are shown on **Plates 2a through 2f**, herein. The updated cross sections together with results of stability analysis were used to develop the DMM depths and the zonation shown on **Plate 1**.

I.4.5.1.1 Methodology

Simplified Seismic Slope Displacement Procedures

Seismic slope displacement was evaluated for an idealized section representative of cross section A-A’ [**Plate 1**; the interpretive cross section is presented in the LLRC Geotechnical Report (Fugro, 2020)] using the simplified procedures developed by Bray and Macedo (2019) and Rathje and Antonakos (2011). In general, these procedures are based on regression of Newmark sliding block type analyses performed for a wide range of slope conditions (i.e., slope height, soil stiffness, and yield acceleration) and substantial databases of ground motions. The two models used herein differ with respect to:

1. Their representation of the dynamic response of the sliding block,

2. The ground motion databases available at the time of their development, and
3. Their parameterization of the ground motion for regression (for building their predictive models).

The Bray and Macedo (2019) procedure is based on fully coupled stick-slip sliding block analyses and the NGA-West2 ground motion database (>6,000 ground motion recordings were used to develop their predictive model). Their coupled model simultaneously captures the nonlinear dynamic response of the sliding mass and its effect on sliding episodes. The Rathje and Antonakos (2011) procedure is based on decoupled analyses, where calculations for the dynamic response of the sliding block and plastic slip (i.e., sliding) are performed independently. Their predictive model is based on an earlier version of the NGA strong ground motion database (>2,000 ground motion recordings were used to develop their predictive model). Coupled analyses are more rigorous and considered superior, although any sliding block type analysis represents potential slope deformations with a very simplistic failure mechanism, and results should be interpreted as an index of slope performance. While both models use earthquake magnitude as a proxy for shaking duration, they employ different parameterization of the seismic demand. The Bray and Macedo (2019) model uses the spectral acceleration (at the base of the sliding mass) at a degraded period equal to 1.3 times the initial period of the sliding mass to represent the seismic demand. The Rathje and Antonakos (2011) model used herein uses the PGA at the base of the sliding mass to represent the seismic demand. Both models require the initial fundamental period of the potential sliding mass (Ts) and the slope's yield coefficient (ky).

The initial fundamental period of the potential sliding mass (T_s) was estimated based on the approximate height of the potential sliding mass observed in the pseudostatic limit equilibrium analyses, the range of in-situ shear wave velocities previously idealized for site response analysis (Supplement G of LLRC Geotech Report), shear modulus ratios ($Gr = G_{dm}/ G_{soil}$) ranging from 5 to 20, an area replacement ratio of 50 percent, and the model for shear wave velocity ratio for periodic grid inclusions proposed by Nguyen et al. (2013). The resulting estimates of T_s ranged from approximately 0.17 to 0.28 seconds.

Seismic slope displacements were estimated for a design-level ground motion based on the geometric mean and without risk coefficients (e.g., PGA_M / 1.5) [CGS Note 48 (CGS, 2019)]. Acceleration response spectra for the MCE_R [tabulated in Table G.4 of the LLRC Geotech Report (Fugro, 2020)], $MCE_{G,}$ and the design-level ground motion used for seismic slope displacement analyses are shown in **Plate 3** for V_{s30} = 260 m/s (i.e., at the base of the YBM). The range of spectral accelerations used for the Bray and Macedo (2019) seismic slope displacement model (corresponding to the estimated range of degraded period of the sliding mass) is annotated on this plate. An earthquake magnitude of 7.6 was used in these analyses based on the maximum considered earthquake associated with the Hayward Fault.

Displacements were estimated using the Bray and Macedo (2019) procedure for both ordinary ground motions and near-fault pulse ground motions. The results presented in the following section were weighted by the expected proportion of pulse motions, which was estimated to be approximately 0.4 based on the Hayden et al. (2014) model. A peak ground velocity (PGV) of 105 cm/s was used for the pulse ground motion predictive model based on the correlation developed by Watson-Lamprey and Abrahamson (2006), which was found to be in good agreement with the 1,000-year PGV computed using the beta web API for the 2018 USGS national seismic hazard maps.

Pseudostatic Slope Stability Analyses and Estimation of Yield Coefficient

The yield coefficient (i.e., the horizontal seismic coefficient that results in a pseudostatic factor of safety of unity) was evaluated by pseudostatic limit equilibrium analyses performed with the commercial software program SLOPE/W (GeoStudio 2019 version 10.0.0.18569; GEOSLOPE, 2019) and the idealized stratigraphy presented in **Table I.4.2**. Circular and non-circular slip surfaces were evaluated using the Morgenstern-Price limit equilibrium method (which satisfies equilibrium of both forces and moments). The design of the DMM ground improvement was iteratively adjusted until pseudostatic stability analyses produced yield coefficient values corresponding to tolerable seismic displacements.

The deep mixing treatment zone was represented with composite properties assuming no shear resistance from native soil between the deep mixed grids. The shear strength of the deep mixed ground was computed as $s_{dm} = ½(f_r \times f_c \times q_{dm,spec})$ where $q_{dm,spec}$ is the specified unconfined compressive strength of the deep mixed ground ($q_{dm,spec}$ = 125 psi), f_c is a factor accounting for curing time (f_c = 1.48 for the 365 day curing time assumed for seismic load cases), and f_r is a factor accounting for differences between unconfined peak and confined large-strain strengths taken as 0.8 (Bruce et al., 2013). The composite shear strength of the treatment zone was then estimated as $s_{dm,grid} = f_v \times A_r \times s_{dm} \approx$ 5,000 psf where f_v is a factor that accounts for the greater variability that typically exists in the strength of deep mixed ground compared to the variability that exists in the strength of clay deposits [f_v was estimated to be 0.95 per the FHWA guidelines (Bruce et al., 2013)], A_r is the area replacement ratio (A_r = 0.5), and s_{dm} is the shear strength of the deep mixed ground defined above.

The Young Bay Mud (YBM) was modelled with an undrained shear strength ratio of 0.22 to approximate cyclic softening behaviours. This softened strength ratio was based on an average peak, static strength ratio of 0.28 and an undrained shear strength reduction factor of 0.8 (i.e., 0.8 × 0.28 = 0.22) based on the cyclic strength (i.e., cyclic resistance ratio, CRR) for M=7.5 suggested by Idriss and Boulanger (2008) for plastic silts and clays, and Fugro's past experience characterizing and modelling YBM.

The liquefiable sands were modelled with residual strength of liquefied soil estimated using the Kramer and Wang (2015) model which depends on energy and effective overburden corrected SPT blow count, $(N_1)_{60}$ and in-situ vertical effective stress. Baseline analyses were performed for $(N_1)_{60}$ =15. Sensitivity analyses were also performed for $(N_1)_{60}$ =10.

Table I.4.2: Soil Properties Used in Pseudostatic Slope Stability Analyses

Material	Unit Weight (pcf)	Material Shear Strength	
		Cohesion c′ (psf)	Friction Angle Φ′ (degree)
Sandy Fill	120	0	35
Young Bay Mud with Sand Lenses	90	0.22 x Effective Overburden Stress (psf)	0
Interbedded Clays and Sands	130	0	40
Highly Liquefiable Sands	110	Kramer and Wang (2015) $(N_1)_{60}$=15	0
DMM Composite	120	5,000	0

I.4.5.1.2 Results

Results of the pseudostatic slope stability analyses and simplified seismic slope displacement estimates are presented herein. **Plates 4a through 4c** show factors of safety and corresponding slip surfaces from pseudostatic stability analyses with a horizontal seismic coefficient (k_h) of 0.35 for block, circular, and optimized circular slip surfaces. The block slip surface (passing through the DMM grid) and the optimized circular slip surface (passing beneath the grid) both have factors of safety of approximately 1.0 (i.e., k_y = 0.35). The circular slip surface (not optimized) exhibits a factor of safety of 1.1 for k_h = 0.35 (i.e., k_y < 0.35). The deep mixed zone is deeper on the east side of the building (Zones A2 and B in **Plate 1**) extending approximately 55 feet deep (Elev. -35 feet; 12 to 22 feet below YBM). This deeper section is approximately 55 feet wide in the slope stability model. Based on the results of the stability analyses the deeper portion of the DMM is extended further back along the north side of the building so that there is at least a 55-foot-wide deep buttress for slip surfaces oblique to the building orientation (e.g., Cross Section F-F′, **Plate 2f**). Additionally, based on the additional CPTs performed in November 2022 the DMM depth was extended in Zone A2 to ensure bearing into the competent sands and clays beneath the YBM. A smaller maximum center-to-center grid spacing is specified for Zone B (**Plate 4**). The deep mixed zone on the western side of the building is shorter (Zone A1 in **Plate 1**), keyed 3-5 feet into the competent sands and clay under the YBM, generally extending to a depth of about 33 feet below ground surface (Elev. -15 feet). Sensitivity analyses with residual strength of liquefied soils based on Kramer and Wang (2015) for $(N_1)_{60}$ =10 had a small effect on the results because only small fractions of the slip surfaces were affected.

The range of estimated seismic displacements for the DMM ground improvement is shown in **Plate 5**. The range is based on: (1) the range of k_y values computed for circular and non-circular slip surfaces and several parameter sensitivity analyses, (2) reasonable ranges of average shear wave velocity of the potential sliding mass, and (3) both the Bray and Macedo (2019) and Rathje and Antonakos (2011) predictive models. Estimated median seismic displacements range from negligible to approximately 13 cm, with the Bray and Macedo (2019) model producing larger displacement estimates for all cases. The T_s = 0.17 and 0.21 seconds analysis cases likely better represent the deep mixed zone in these analyses (Gr ≈ 20 and 10, respectively). The T_s = 0.28 seconds case was included as a reasonable sensitivity analysis where the degraded period of the sliding mass corresponds to the peak spectral acceleration (i.e., $Sa(1.3T_s)$ = 1.2 g). The differences between the two predictive models are partly attributed to differences in how they model the dynamic response of the sliding mass, with the Bray and Macedo (2019) coupled model considered superior. The approximate performance of the existing slope (i.e., without DMM ground improvement) is also annotated on **Plate 5** for reference. Large seismic displacements were estimated for the existing slope for $k_y \approx 0.1$, which corresponds to a range of performance where displacement estimates are very sensitive to small changes in yield acceleration. Conversely, the DMM ground improvement performance falls on a much flatter part of the displacement vs. k_y curves.

Two seismic displacement thresholds are indicated on **Plate 5**. The 15-cm (6-inch) threshold is commonly accepted for screening-level evaluations of earthquake-induced landslide hazard (e.g., SP-117A, 2008). The 15-cm threshold likely distinguishes small to moderate displacements from larger displacements (Blake et al., 2002), and sliding block displacement estimates less than approximately 15 centimetres are unlikely to correspond to serious landslide movement or damage (SP-117A, 2007). The 10-cm (4-inch) threshold corresponds to the ASCE 7-16 upper limit for lateral spreading horizontal ground displacement for shallow foundations for buildings in Risk Category IV (Table 12.13-2 of ASCE 7-16). Therefore, we designed the DMM ground improvement to limit average lateral spread displacement to approximately 10 cm. This corresponds to a yield acceleration of approximately 0.35 (**Plate 4**) which was computed for the best estimate pseudostatic stability analyses previously presented on **Plates 4a and 4c**.

I.4.5.1.3 Conclusions

Seismic slope displacement estimates based on two methods and incorporating uncertainty in the initial fundamental period of the sliding mass were on average less than 10 cm and are considered tolerable given the ASCE 7-16 upper limit for lateral spreading horizontal ground displacement for shallow foundations (10 cm for buildings in Risk Category IV). Therefore, the design of the DMM ground improvement presented herein is judged to be acceptable with respect to global seismic stability.

The deeper treatment zone modelled on the east side of the building in the two-dimensional pseudostatic stability analyses will need to also wrap around the north side of the building to limit lateral displacement for slip surfaces oblique to the building's principal orientation and to protect the north side of the building against potential soil loss. This configuration is shown in plan on **Plate 1** and provides a similar width of deeper treatment (to what was modelled for Section A-A′) for potential slip surfaces on cross sections oblique to the building orientation. The deeper ground improvement extends along the north side of the LLRC building to the eastern limit of the Building E.

The final depth of the DMM grid should be determined based on the results and interpretation of additional CPTs that we recommend be performed prior to construction. The DMM depths presented in this section (and used for the stability analyses) are minimum depths that may need to be exceeded based on interpretation of the additional CPTs.

I.4.5.2 Post-Seismic Global Stability

Post-seismic stability analyses demonstrated factors of safety greater than five. Post-seismic global stability was evaluated using the same properties shown in **Table I.4.2**, except the YBM was modelled with an undrained shear strength ratio of 0.17 (i.e., a strength reduction factor of 0.6 to represent cyclically softened strength for a static loading rate). Analyses were performed for both the original slope geometry and for a case where the channel side soils (east of the building) are assumed to have displaced towards the channel more than the building, exposing a free face of deep mixed ground approximately 20 feet tall. In both cases the DMM ground improvement is acceptable from a post-seismic global stability perspective.

I.4.6 Additional Stability Checks

Additional stability checks were performed following FHWA guidelines (Bruce et al., 2013) and including seismic loads. Overall, the design of the DMM ground improvement was controlled by global seismic slope displacements given the favourable aspect ratio and relatively high area replacement ratio needed. External stability was checked for combined overturning and bearing. Internal stability was checked for crushing of the deep mixed shear walls at the outside toe, racking failure (shearing on vertical planes in the deep mixed shear walls), and extrusion of soil between the deep mixed shear walls.

To simplify these checks, we conservatively modelled the shallower treatment zone on the west end of the building and the deeper treatment zone on the east end of the building as separate blocks of soil-cement. Seismic activate earth pressures were estimated using the same limit-equilibrium models used for the pseudostatic analyses and with k_h = 0.35 (following the GLE approach described by Anderson et al., 2008). A horizontal inertial load of the deep mixed ground was also included as the weight of the deep mixed ground (W) times the horizontal seismic coeffect (k_h). Additionally, no passive resistance from the channel side of the deep mixed ground was conservatively assumed. Factors of safety for all additional stability checks were more than 1.3 [the minimum value for static load cases recommended by Bruce et al. (2013)].

I.4.7 DMM Treatment Zone Settlements

Post-liquefaction reconsolidation settlement within the building footprint was estimated to range between approximately 1 and 4 inches based on the local borings and CPTs. Note that the largest settlement (approximately 6 inches) was estimated for CPT-03 which was performed to the southeast of the building. The potential for the DMM to reduce cyclic shear stresses and limit post-liquefaction reconsolidation settlement was evaluated for shear modulus ratios ($G_r = G_{dm} / G_{soil}$) ranging from 5 to 20 and the design area replacement ratio of 50%. The shear stress reduction factor ($R_d = CSR_I / CSR_U$, where CSR_I and CSR_U and cyclic stress ratio for the improved and unimproved cases, respectively) was estimated using the relationship proposed by Nguyen et al. (2013) for periodic grid arrangement of shear walls. CPT-based post-liquefaction reconsolidation settlement analyses for R_d values ranging from 0.5 (A_r = 50% and G_r = 5) to 0.17 (A_r = 50% and G_r = 20) resulted in negligible (R_d = 0.5) to substantial (R_d = 0.17) reduction in estimated settlements. A R_d value of 0.3 (A_r = 50% and G_r = 10) provides a reasonable estimate of the settlement hazard that accounts for reduced seismic shear stresses in the native soil and resulted in reconsolidation settlements within the building footprint on the order of 1 to 2 inches for the MCE.

We judge that by using DMM to mitigate liquefaction effects, post-liquefaction reconsolidation settlements of 1 to 2 inches can develop between DMM grids beneath the floor slabs. Therefore, we recommend using structural slabs to span between deep mixed shear walls.

In addition, we estimate static settlement of buildings and structures supported on the deep mixed ground will depend on footing layout and service load and will be less than about 3/4 inch.

Underground pipelines (gas lines, sanitary sewers, water services, etc.) should be properly designed considering differential settlements of about 4 inches associated with post-liquefaction reconsolidation between DMM supported structure and unimproved areas adjacent to the building. Additional consideration should be given to the impacts of differential seismic slope movements (lateral and vertical) between the improved and unimproved areas.

I.5 Seismic Design Parameters

Due to the ground improvement, the combination of the DMM grids and the existing soil will create a stiffer composite medium which has a higher shear wave velocity compared to the native soft YBM and sandy fill material. We estimate that the shear wave velocity of the top 100 feet of soil (V_{s30}) will increase to 270 m/s due to the ground improvement. The corresponding composite average shear wave velocity profile was estimated based on the idealized shear wave velocity profiles previously developed for site response analyses [**Supplement G** of LLRC Geotech Report (Fugro, 2023)] and considering shear modulus ratios, G_r, ranging from 5 to 30 (based on deep mixed soil-cement E_{50} values between 300 and 600 times the specified unconfined compressive strength, as recommended by Boulanger and Shao, 2021). Overall, for the resulting range of estimated V_{s30} values the short period spectral accelerations (that control short period spectral acceleration parameters, S_{DS} and S_{MS}) increase with increasing V_{s30}. Given that the estimated building period is 0.45 seconds, we selected a representative V_{s30} based on a reasonably conservative average G_r value of 20, which for A_r = 50% corresponds with a ratio of $V_{s,av}$ / V_s of approximately 2.5 per the relationship developed by Nguyen et al. (2013) for periodic grid inclusions (where $V_{s,av}$ is the average shear wave velocity in the treatment zone and Vs is the soil's shear wave velocity) The idealized V_s profiles previously developed for site response produce V_{s30} values of approximately 270 m/s when multiplied by 2.5 over the thickness of fill and YBM.

A site-specific Probabilistic Seismic Hazard Analysis (PSHA) was performed for the new Vs30 to estimate the severity of ground motions that may affect the project site for specific design levels of hazard. The design ground motion parameters were calculated following the site-specific ground motion procedures defined in Chapter 21 of ASCE 7-16 (ASCE, 2016) as required by the California Building Code (CBC) (CBSC, 2019).

Table I.5.**1** tabulates the spectral ordinates of the recommended site-specific MCER and design response spectra per ASCE 7-16. The corresponding design acceleration parameters SMS, SM1, SDS, and SD1 are tabulated in **Table I.5.2**.

Table I.5.1: MCE$_R$ and Design Response Spectra per ASCE 7-16 for a Vs30 of 270 m/sec, 5% Damping

	Horizontal Spectral Acceleration (g)							
Period (sec)	UHRS for Return Period of 2,475 Years	Probabilistic MCE$_R$	84th Deterministic Spectrum	Deterministic Lower Limit	Deterministic MCE$_R$	Site-Specific MCE$_R$	80% General Response Spectrum	Design Response Spectrum
0.01 (PGA)	0.947	0.959	0.731	0.555	0.804	0.804	0.400	0.536
0.03	0.974	0.987	0.739	0.561	0.812	0.812	0.459	0.542
0.05	1.09	1.11	0.810	0.615	0.891	0.891	0.518	0.594
0.075	1.35	1.37	0.96	0.73	1.06	1.06	0.591	0.704
0.1	1.58	1.60	1.1	0.837	1.21	1.21	0.664	0.807
0.15	1.87	1.90	1.33	1.01	1.47	1.47	0.811	0.978
0.19	2.06	2.09	1.47	1.11	1.61	1.61	0.927	1.08
0.2	2.1	2.13	1.5	1.14	1.65	1.65	0.927	1.1
0.25	2.27	2.36	1.62	1.26	1.83	1.83	0.927	1.22
0.3	2.39	2.52	1.72	1.36	1.97	1.97	0.927	1.32
0.4	2.44	2.64	1.81	1.48	2.14	2.14	0.927	1.43
0.5	2.36	2.62	1.79	1.50	2.17	2.17	0.927	1.45
0.75	1.95	2.24	1.53	1.33	1.93	1.93	0.927	1.29
0.949	1.68	1.97	1.36	1.21	1.75	1.75	0.927	1.17
1	1.63	1.92	1.32	1.19	1.72	1.72	0.88	1.14
1.5	1.16	1.42	0.991	0.924	1.34	1.34	0.587	0.892
2	0.891	1.12	0.785	0.752	1.09	1.09	0.44	0.726
3	0.583	0.759	0.541	0.537	0.778	0.759	0.293	0.506
4	0.413	0.551	0.389	0.396	0.573	0.551	0.22	0.367
5	0.310	0.421	0.290	0.300	0.435	0.421	0.176	0.281
7.5	0.171	0.233	0.145	0.15	0.218	0.218	0.117	0.145
8	0.154	0.21	0.128	0.133	0.192	0.192	0.110	0.128
10	0.107	0.146	0.084	0.087	0.125	0.125	0.070	0.084

Table I.5.2: Design Parameters per ASCE 7-16 at the Ground Surface, 5% Damping

Parameter	Value
S_{MS}	1.95
S_{M1}	2.28
S_{DS}	1.30
S_{D1}	1.52
T_L	8 seconds

Plate 6 shows the mean annual seismic hazard curves for selected spectral periods ranging from 0.01 to 10 seconds for Vs30 of 270 m/s. A spectral period of 0.01 seconds is used to represent the peak ground acceleration (PGA). These hazard curves represent the total mean hazard from combining all seismic sources and ground motion models. These figures also indicate the annual frequency of exceedance corresponding to a return period of 2,475 years. **Plate 7** presents the 5 percent-damped mean horizontal UHRS (Uniform Hazard Response Spectrum) for a return period of 2,475 years and the representative Vs30 value of 270 m/sec. The UHRS for a return period of 2,475 years along with probabilistic response MCE_R response spectrum are illustrated in **Plate 8**.

Plate 9 presents the development of the site-specific MCE_R and design response spectra for the site. In this case, the deterministic MCE_R spectrum is lower than the probabilistic MCE_R spectrum for all spectral periods. The site-specific MCE_R spectrum is the maximum of: 1) the minimum of the probabilistic and deterministic MCE_R, and 2) 150 percent of the design response spectrum. Following ASCE 7-16, the recommended design response spectrum for the site was calculated as the maximum of 2/3 of the site-specific MCE_R and the lower limit specified by ASCE 7-16 (80 percent of the general spectrum for Site Class D, using modified F_a and F_v values provided in Section 21.3 of ASCE 7-16). The transition period from constant velocity to constant displacement, T_L, required to calculate the lower limit, was estimated as 8 seconds using the USGS web service (https://earthquake.usgs.gov/ws/designmaps/asce7-16.html).

I.6 Foundation System

I.6.1 Spread Footing

We anticipate an area replacement ratio (A_r) of at least 50% will be used in the DMM design. The DMM columns can be constructed in various diameters and selection of the diameter and depth of these columns is project specific. Typically, the overlap of adjacent DMM columns is about 30% of the column diameter. The A_r may vary based on number of DMM columns under the foundation. To achieve the full allowable bearing pressure, DMM should extend laterally beyond footing bases such that the area replacement ratio under the footing is 100%. Otherwise, the bearing capacity should be multiplied by the actual A_r under the foundation.

The axial capacity of the DMM grids is the minimum of the structural capacity and geotechnical capacity of the grids. For the designed DMM grids, the structural capacity is expected to control. Using a design DMM unconfined compressive strength of 125 psi, and following the FHWA design guidelines (Bruce et al., 2013), the ultimate structural capacity of the DMM columns (Ar=100%) is estimated to be 21,300 pounds per square feet (psf) for long term and seismic loads and 14,400 psf for short term construction loads (after 28 days). According to CBC Section 1605A.1.1, the factor of safety for soil bearing shall not be less than the overstrength factor. Factors of safety for allowable stress design are show in **Table** I.**6.1** below. For example, if the overstrength factor is 2.5, the factors of safety for Dead Load, Dead plus Live Load, and Total Load cases are 3, 2.5, and 2.5, respectively.

Table I.6.1: Factors of Safety for Axial Loading of Foundations (Allowable Stress Design)

Load Condition	Factor of Safety
Dead Load	maximum (3, overstrength factor)
Dead plus Live Loads	maximum (2, overstrength factor)
Total Loads (including wind or seismic)	maximum (1.5, overstrength factor)

Provided bearing capacities are for shallow foundations supported on DMM with 100% area replacement ratio (A_r). It should be noted that considering the relatively high bearing capacity of the DMM, the A_r under the footing may be decreased depending on the design loads. The allowable bearing pressure is a net value; therefore, the weight of the footing can be neglected for design purposes. For footings supported on DMM with A_r less than 100%, the bearing capacity should be reduced by multiplying by A_r.

Footings should be at least 12 inches wide, and bottom of footings should be founded at least 24 inches below the lowest adjacent finished grade. Estimated static settlement of building supported on the deep mixed ground depends on footing layout and service loads, but should be less than about ¾ inch.

Resistance to lateral loads may be provided by friction along the base of foundations and by passive pressures acting on the sides of foundations. An allowable friction coefficient of 0.35 may be multiplied by the dead load to evaluate the allowable frictional resistance along the bottom of foundations. Where the footing is poured neat against subgrade soils, an ultimate passive pressure equal to an equivalent fluid pressure of 500 pounds per cubic foot (pcf) can be used for lateral load resistance against the sides of footings perpendicular to the direction of loading. If the footing is poured against forming, the ultimate passive resistance will be reduced by 30%. The upper 12 inches of soils should be ignored unless they are confined by pavement or slab.

The passive resistance value can be linearly interpolated between at-rest pressure (equivalent to a fluid pressure of 50 pcf) at zero deflection and the ultimate at a deflection of 0.025*D, where D is the depth of the footing. The passive pressures against the footings and grade beams along the eastern and northern edge of the treatment Zone B should be ignored in foundation design due to potential for seismic soil displacements adjacent to the building in these areas.

I.6.2 Structural Mat Slab

When used, the structural mat slabs foundations should be supported on properly prepared subgrade that is proof rolled to provide a smooth, unyielding surface for slab support. Where the slab will be located at surface grade, we recommend at least 12 inches of imported, predominantly granular, "non-expansive" engineered fills that meet the requirements presented in the **Section 7.2** of the LLRC Geotechnical Report (Fugro, 2023) be provided below the slab. For slabs that support vehicular loads, the "non-expansive" engineered fill layer should consist of Caltrans Class 2 aggregate base.

For the portion of the slab that are supported on compacted soil, we recommend a modulus of subgrade reaction (k_1, 1 foot by 1 foot) of 125 pounds per square inch per inch (psi/in) be used for the design of the structural mat slab foundation for the static condition. This value can be modified to k as 125/B psi/in, where B is the equivalent foundation width measured in feet. However, for the seismic condition, the slabs should be able to span between the DMM grids due to liquefaction-induced settlements (i.e., subgrade modulus = 0.0).

For the portion of the slab that are supported on 100% area replacement ratio DMM, we recommend a modulus of subgrade reaction (k_1, 1 foot by 1 foot) of 4,000 psi/in be used for the design of the structural mat slab foundation. This value can be modified to k as 4,000/B psi/in, where B is the equivalent foundation width measured in feet. Recommended values of modulus of subgrade reaction for various footing aspect ratios are provided in **Table I.6.2** below. For other aspect ratios, values of modulus of subgrade reaction can be linearly interpolated. Note that if the area replacement ratio is less than 100%, **rigid footing** bearing capacities provided above and **modulus of subgrade reaction** in table below shall be **reduced by** multiplying by **the** area replacement ratio.

Table I.6.2: Values of Modulus of Subgrade Reaction for Various Rigid Footing Aspect Ratios

Footing Aspect Ratio Length (L) : Width (B)	Modulus of Subgrade Reaction k (psi/inch)
1	4,000/B
1.5	3,300/B
2	2,900/B
3	2,500/B
4	2,300/B
5	2,100/B
10	1,800/B
15	1,700/B

Note: B, equivalent footing width measured in feet

I.7 Conclusions and Recommendations

We recommend that approximately 30- to 55-foot deep DMM ground improvement be used to support shallow foundations of the proposed LLRC building to mitigate seismic hazards. We conclude that the LLRC building supported on DMM that are designed using the recommendations provided below will meet the project settlement criteria and allowable lateral spread displacement for shallow foundations per ASCE 7-16. We estimate total settlements less than ¾ inch (differential settlements up ½ inch over 30 feet or between adjacent structural columns) for DMM supported foundations and less than approximately 4 inches of average lateral spread displacement for the design-level ground motion.

A summary of our key recommendations for DMM follows:

- Ultimate DMM compressive capacities of 21,300 psf and 14,400 psf can be used for design of footings and slabs supported on DMM with 100% A_r for long term/seismic and short-term construction loads (after 28 days), respectively. For footings supported on DMM with A_r less than 100%, the bearing capacity should be reduced by multiplying by A_r.
- The allowable axial capacity should be calculated by dividing the ultimate axial capacities by the factors of safety provided in **Table I.6.1**.
- Lateral resistance of footing bases and slabs supported on DMM can be calculated using an allowable frictional coefficient of 0.35.
- The DMM should be constructed using multi-shaft mixing equipment to create columns that are a minimum of 3 feet and a maximum of 6 feet in diameter.
- The overlapping between any two adjacent DMM columns should be at least 30 percent of the column diameter.

- The bottom of DMM columns should be extended to below elevation -15 feet for Zone A1 and -35 feet for Zones A2 and B.
- The design unconfined compressive strength ($q_{dm,spec}$) is 125 psi .
- The mixed-in-place soil cement grids and blocks should cover a minimum Area Replacement Ratio (A_r) of 50 percent for a specified unconfined compressive strength ($q_{dm,spec}$) of 125 psi.
- Center-to-center spacing of DMM grids should not exceed 4.0d Zones A1 and A2 and 3.2d for Zone B, for min A_r = 50% ($q_{dm,spec}$ = 125 psi)., where d is the diameter of the DMM columns.
- DMM within the building footprint should be arranged in an uninterrupted grid that follows the building column lines and underlies all footings and moment frame grade beams.
- The DMM should underlie the entire building footprint and extend laterally to include any attached structures which are deemed to be essential parts of the buildings.
- To achieve the full allowable bearing pressure, DMM should extend laterally beyond footing bases such that the area replacement ratio under the footing is 100%. Otherwise, the bearing capacity should be multiplied by the actual A_r under the foundation.
- Elevator shafts should also be supported by DMM grids.
- Ground floor slabs should be designed to structurally span between DMM walls. Vapor barrier recommendations for floor slabs are provided in Section 7.3.4 of the LLRC Geotechnical Report (Fugro, 2023).
- The top of the DMM elements should extend to the base of the ground floor slab section, the bottom of footings, and the bottom of moment frame grade beams. The bottom of DMM elements should extend at least to the minimum depth shown on **Plate 1**.
- The average unconfined compressive strength of the DMM core specimens should be at least 125 psi at 28 days for a minimum A_r of 50% as determined by ASTM D2166. Ninety percent (90%) of all unconfined compressive strength tests on core samples should exceed the specified unconfined compressive strength.
- Lumps of unimproved soils should not amount to more than 15 percent of the total volume of any core run from continuous full-depth core sample and all of the unrecovered core length should be assumed to be unimproved soil.
- Any individual or aggregation of lumps of unimproved soil should not be larger than 12 inches in greatest dimension.
- Detailed DMM acceptance criteria are provided in performance specifications in **Supplement J, Construction Specifications for Deep Mixing**.
- Before construction, a detailed utility locating report should be provided to the design team.
- The DMM Contractor should control and process all spoils created during the DMM construction and should coordinate with the project grading contractor for the spoils to be reused as fills at the project site. The DMM spoils can be used below all interior slabs-on-grade or structural mat slabs as non-expansive engineered fills provided, they meet the requirements provided in our Geotechnical Report (Fugro, March 31, 2023).

- Design parameters per ASCE 7-16 at the ground surface are tabulated in **Table I.7.1** below:

Table I.7.1: Design Parameters per ASCE 7-16 at the Ground Surface, 5% Damping

Parameter	Value
S_{MS}	1.95
S_{M1}	2.28
S_{DS}	1.30
S_{D1}	1.52
T_L	8 seconds

- DMM modulus of subgrade reaction (k_1, 1 foot by 1 foot) of 4,000 psi/in can be used for the structural design for portions of the slab that are supported by A_r = 100% DMM. The modulus of subgrade reaction for compacted soil between DMM grid is 125 psi/in which should be ignored when designing for seismic condition. These values can be modified to k aa k_1/B psi/in, where B is the equivalent foundation width measured in feet. DMM modulus of subgrade reaction (k) for footings with various aspect ratios are provided in Table 6.1.
- Design requirements for foundations in liquefiable sites specified in **Section 12.13.9** of **ASCE 7-16** should be used for when designing the structural members and foundation system.
- We judge that by using DMM to mitigate liquefaction effects, post-liquefaction reconsolidation settlements of up to 2 inches can develop between DMM grids beneath the floor slabs. Therefore, we recommend using structural slabs to span between deep mixed shear walls.
- We recommend DMM ground improvement be installed by a qualified specialty contractor with demonstrated experience in this type of ground improvement. Construction of uniformly mixed, high strength DMM columns requires proper equipment, trained and experienced personnel, the proper mix design for the soils encountered, careful attention to the construction procedures, continuous monitoring of the installation parameters, and sufficient quality control testing. The DMM contractor should develop construction procedures, mix design, and quality control required to achieve the desired results and meet the project design and specified acceptance criteria. Additional details regarding contractor's responsibility are included in the DMM specifications.
- Fugro should also be retained to provide geotechnical services during DMM contractor selection, construction document and drawing submittal review, and DMM implementation and testing, to observe compliance with the design concepts, specifications, and recommendations presented in this addendum and the project Geotechnical Report (Fugro, , March 31, 2023). Our presence will also allow us to modify design if unanticipated subsurface conditions are encountered.

I.8 References

Anderson, D. G., Martin, G. R., Lam, I., & Wang, J. N. (2008). *Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments* (NCHRP Report 611). Transportation Research Board.

Blake, T. F., Hollingsworth, R. A., & Stewart, J. P. (2002). Recommended procedures for implementation of DMG special publication 117 guidelines for analyzing and mitigating landslide hazards in California. *Los Angeles Section Geotechnical Group, Document published by the Southern California Earthquake Center*.

Boulanger, R. W., Khosravi, M., Khosravi, A., & Wilson, D. W. (2018). Remediation of liquefaction effects for an embankment using soil-cement walls: Centrifuge and numerical modeling. *Soil Dynamics and Earthquake Engineering*, 114(2018), 38-50, 10.1016/j.soildyn.2018.07.001

Boulanger, R. W., & Shao, L. (2021). Liquefaction mitigation with deep mixing. *Proceedings, Deep Mixing 2021*, Deep Foundations Institute, 1146-1202.

Bray, J. D., & Macedo, J. (2019). Procedure for Estimating Shear-Induced Seismic Slope Displacement for Shallow Crustal Earthquakes. *Journal of Geotechnical and Geoenvironmental Engineering*, *145*(12), 04019106. https://doi.org/10.1061/(asce)gt.1943-5606.0002143

Bruce, M. E. C., Berg, R. R., Collin, J. G., Filz, G. M., Terashi, M., & Yang, D. S. (2013). *Federal highway administration design manual: Deep mixing for embankment and foundation support* (No. FHWA-HRT-13-046). Federal Highway Administration.

Rathje, E. M., & Antonakos, G. (2011). A unified model for predicting earthquake-induced sliding displacements of rigid and flexible slopes. *Engineering Geology*, *122*(1–2), 51–60. https://doi.org/10.1016/j.enggeo.2010.12.004

California Geological Survey (CGS). (2019). California Geological Survey – Note 48, Checklist for the Review of Engineering and Seismology Reports for Public Schools, Hospitals, and Essential Services Buildings, November 2019.

Fugro, February 28, 2020, Geotechnical Investigation and Geologic Hazard Evaluation, Laney College Library Learning Resource Center, Oakland, California, Project No. 04.72190021-PR-001 02.

GEOSLOPE. (2019). SLOPE/W (GeoStudio 2019 version10.0.0.18569) [Slope stability software for soil and rock slopes]. GEOSLOPE. https://www.geoslope.com/products/slope-w

Hayden, C. P., Bray, J. D., & Abrahamson, N. A. (2014). Selection of near-fault pulse motions. *Journal of Geotechnical and Geoenvironmental Engineering*, *140*(7), 04014030.

Idriss & Boulanger (2008), *Soil Liquefaction During Earthquakes*, EERI Monograph MNO-12.

Kramer, S. L., & Wang, C. H. (2015). Empirical model for estimation of the residual strength of liquefied soil. *Journal of Geotechnical and Geoenvironmental Engineering*, *141*(9), 04015038.

Namikawa, Koseki, & Suzuki (2007), *Finite Element Analysis of Lattice-Shaped Ground Improvement by Cement-Mixing for Liquefaction Resistance, Soils and Foundations*, Volume 47, No. 3, Japanese Geotechnical Society, pp. 559-576, June 2007.

Nguyen, Rayamajhi, Boulanger, Ashford, Lu, Elgamal, & Shao (2013), *Design of DSM Grids for Liquefaction Remediation*, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, November 2013, p.1923-1933.

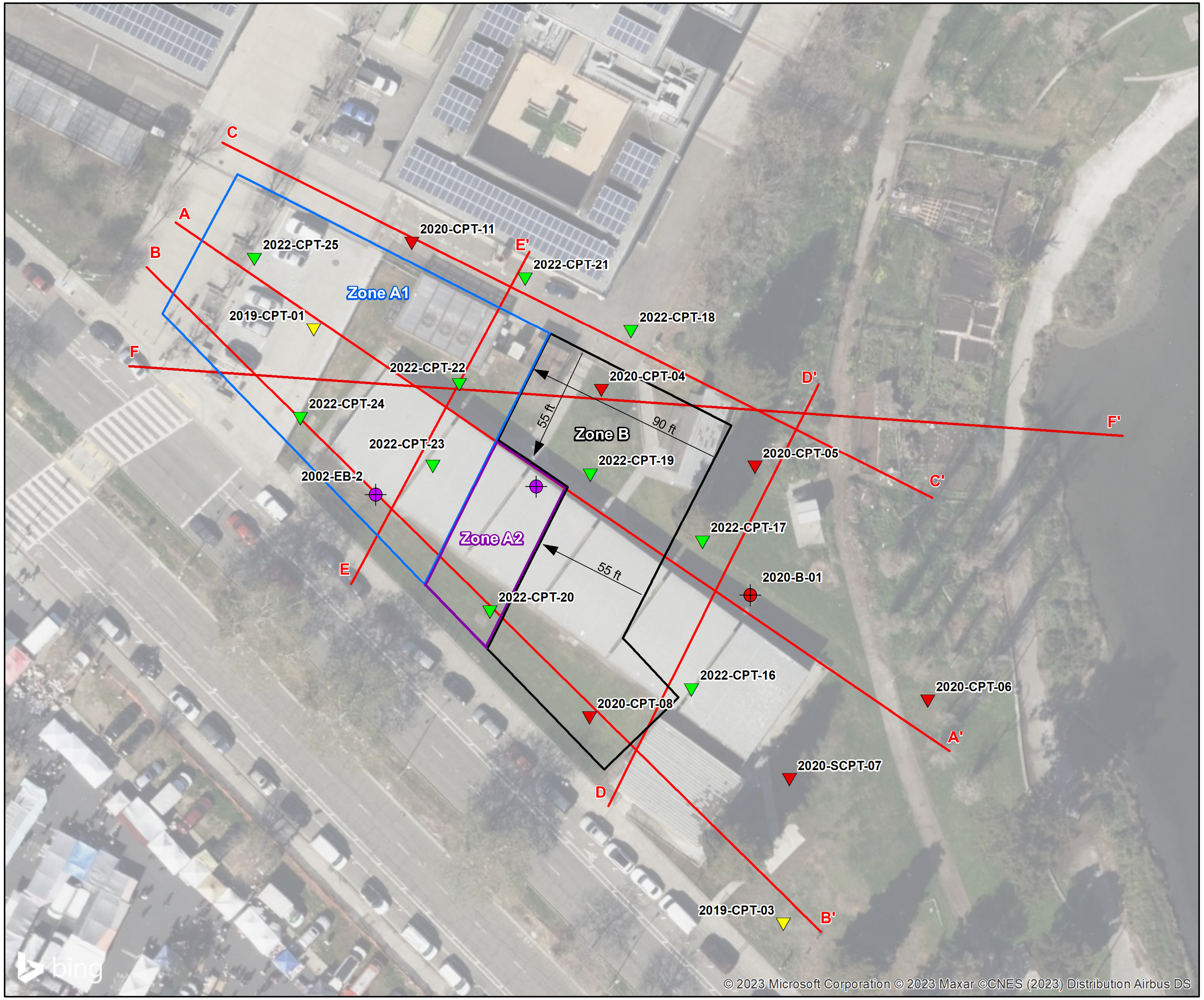
Siddharthan & Porbaha (2008), *Seismic Response Validation of DM Treated Liquefiable Soils*, 6th International Conference on Case Histories in Geotechnical Engineering, Arlington, VA.

Tsukuni & Uchida (2017), *Estimation of Liquefaction Prevention Using Settlement in Grid-Form Ground Improvement Design*, Proceedings of the 19th International Conference on Soil Mechanics and Geotechnical Engineering, Seoul 2017, p.1611-1614.

Watson-Lamprey, J., & Abrahamson, N. (2006). Selection of ground motion time series and limits on scaling. *Soil Dynamics and Earthquake Engineering*, *26*(5), 477-482.

Yamashita, Tanikawa, Shigeno, & Hamada (2015), *Vertical Load Sharing of Piled Raft with Grid-Form Deep Mixing Walls*, Proceedings of the Deep Mixing 2015 Conference, Deep Foundation Institute, p.437-446.

List of Plates



Legend

- Cross section
- Cone Penetration Test by Gregg Drilling LLC (October 7, 2022)
- Cone Penetration Test by Fugro (Oct 2020, Fugro No. 04.00174369)
- Cone Penetration Test by Fugro (Mar 2019 & Jan 2020, Fugro No.04.72190021)
- Exploratory Boring by WCS (Nov 1965, WCS No. S10312)
- Exploratory Boring by Fugro (Oct 2020, Fugro No. 04.00174369)
- Zone A1: DMM Columns Tip Elev. = -15 ft
- Zone A2: DMM Columns Tip Elev. = -35 ft
- Zone B: DMM Columns Tip Elev. = -35 ft

N

0 25 50 100 Feet

1 inch = 50 feet

Plate I-1: Ground Improvement Plan

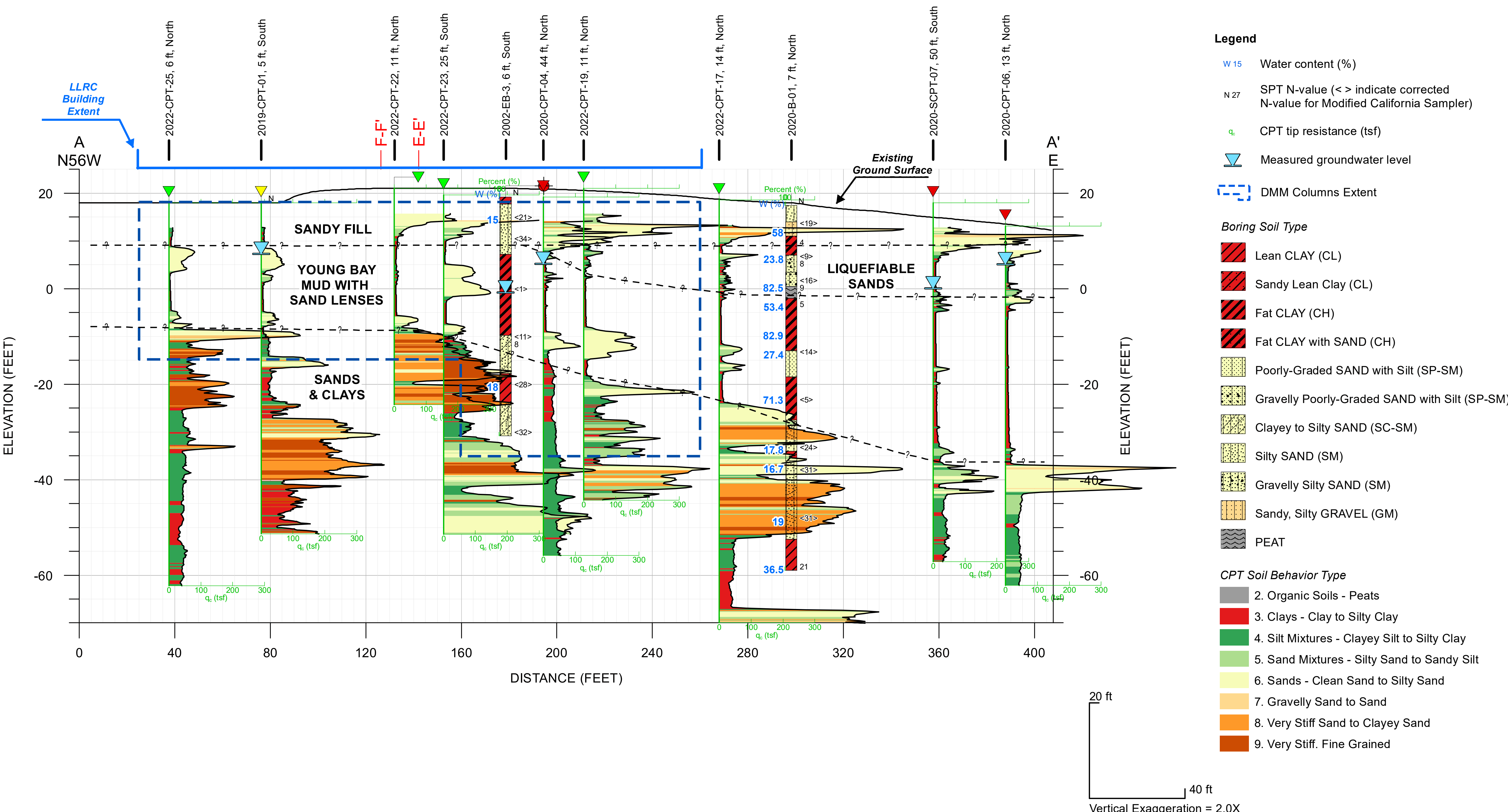


Plate I-2a: Cross Section A-A'

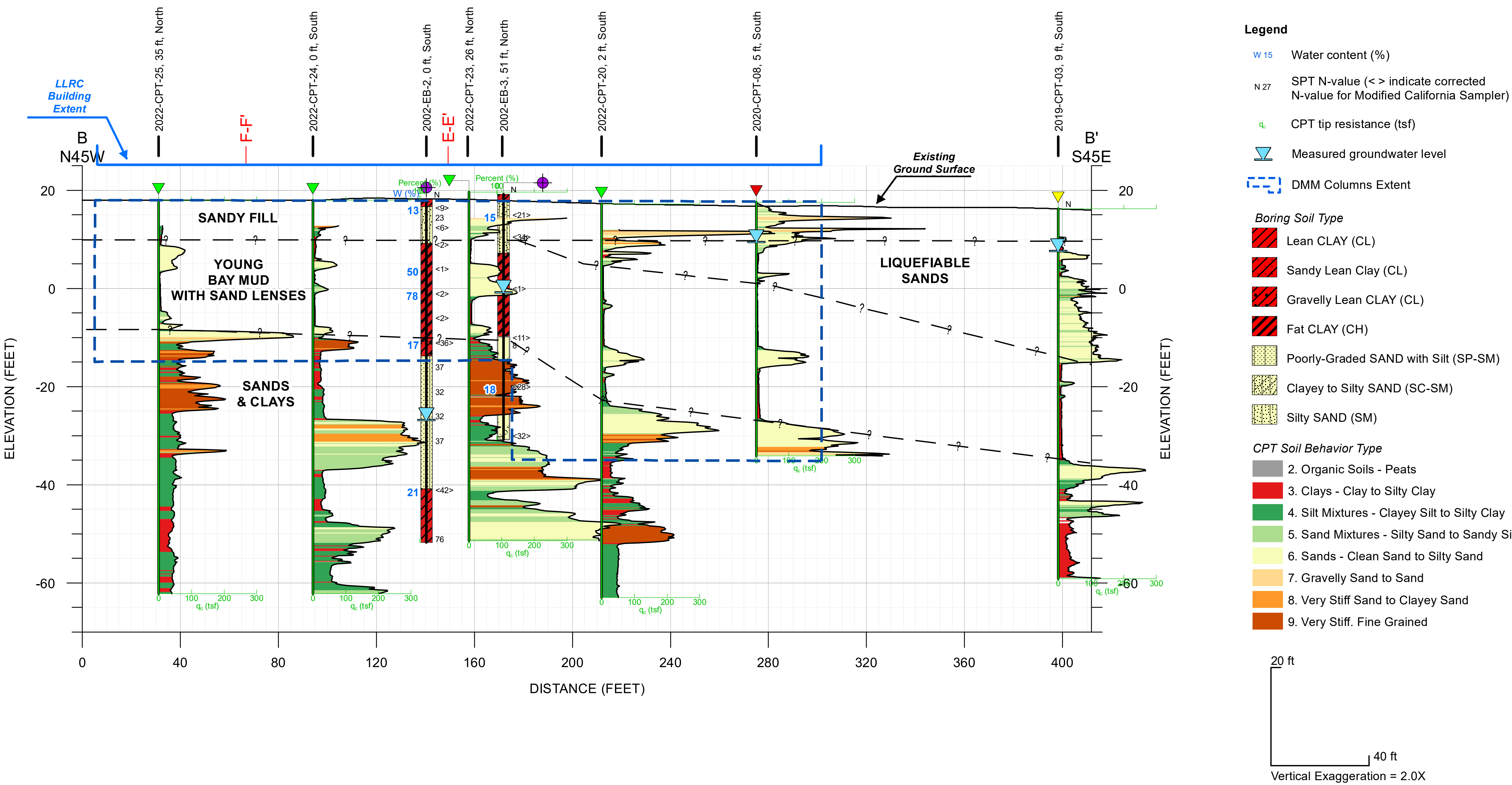


Plate I-2b: Cross Section B-B'

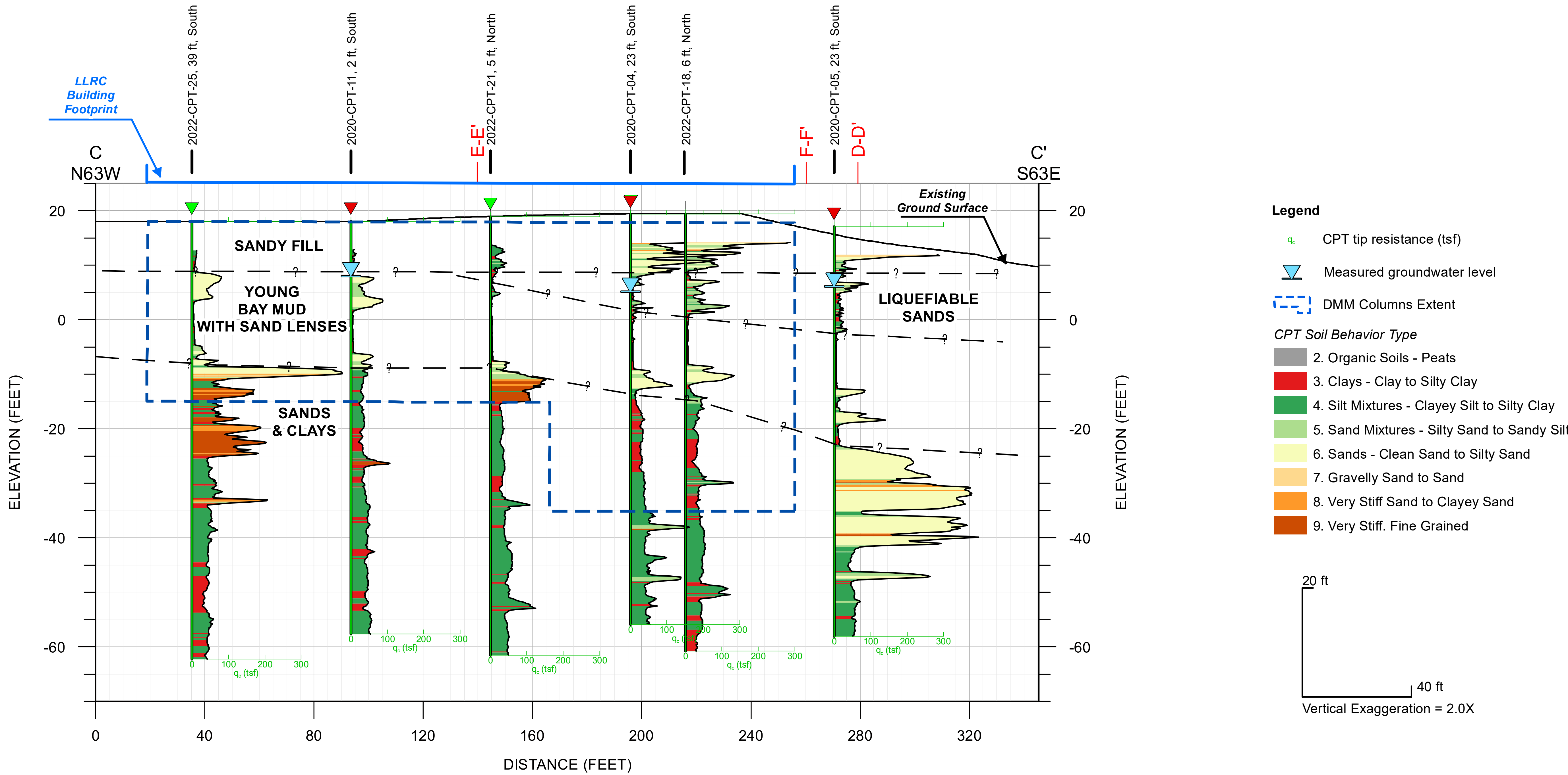


Plate I-2c: Cross Section C-C'

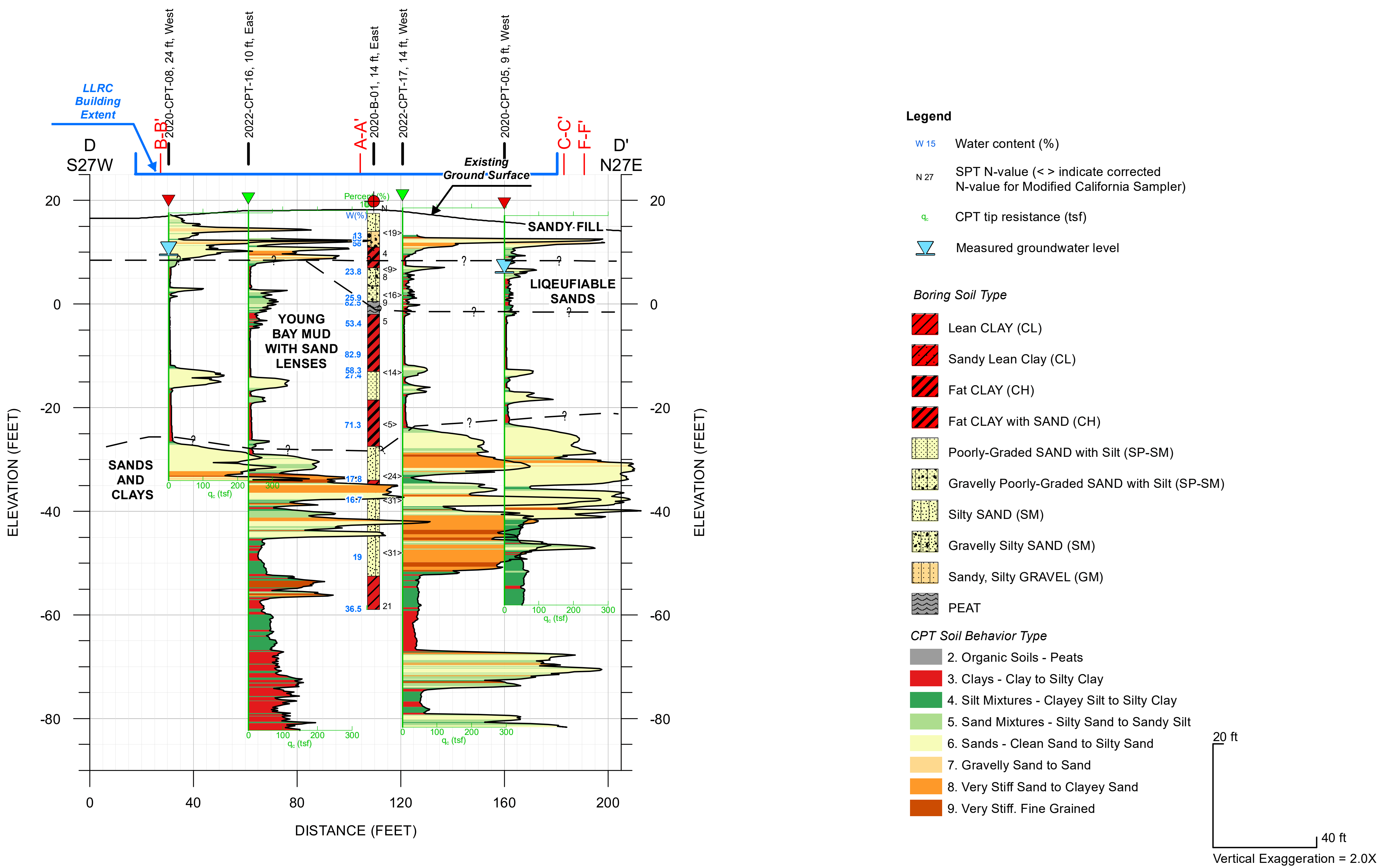


Plate I-2d: Cross Section D-D'

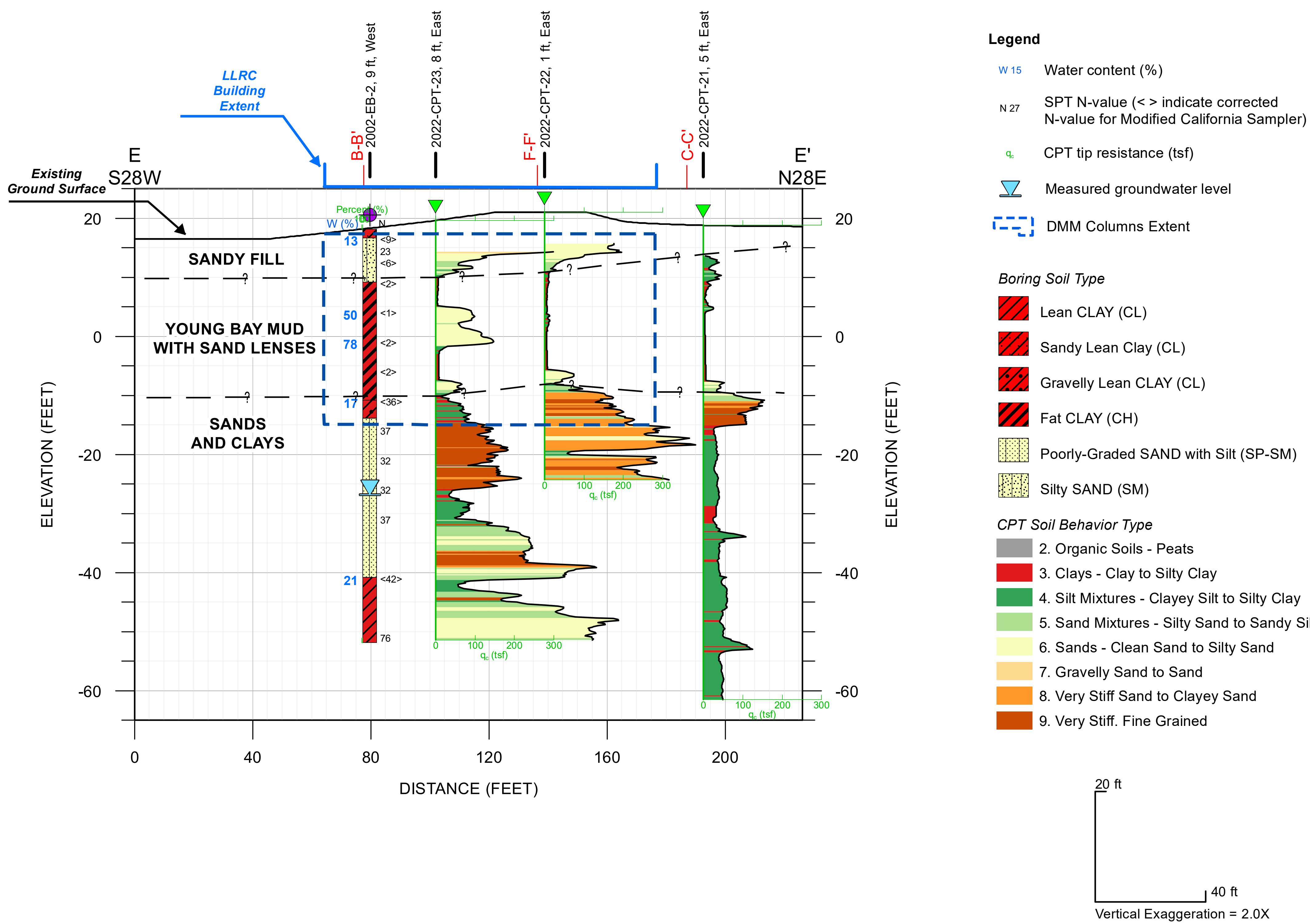


Plate I-2e: Cross Section E-E'

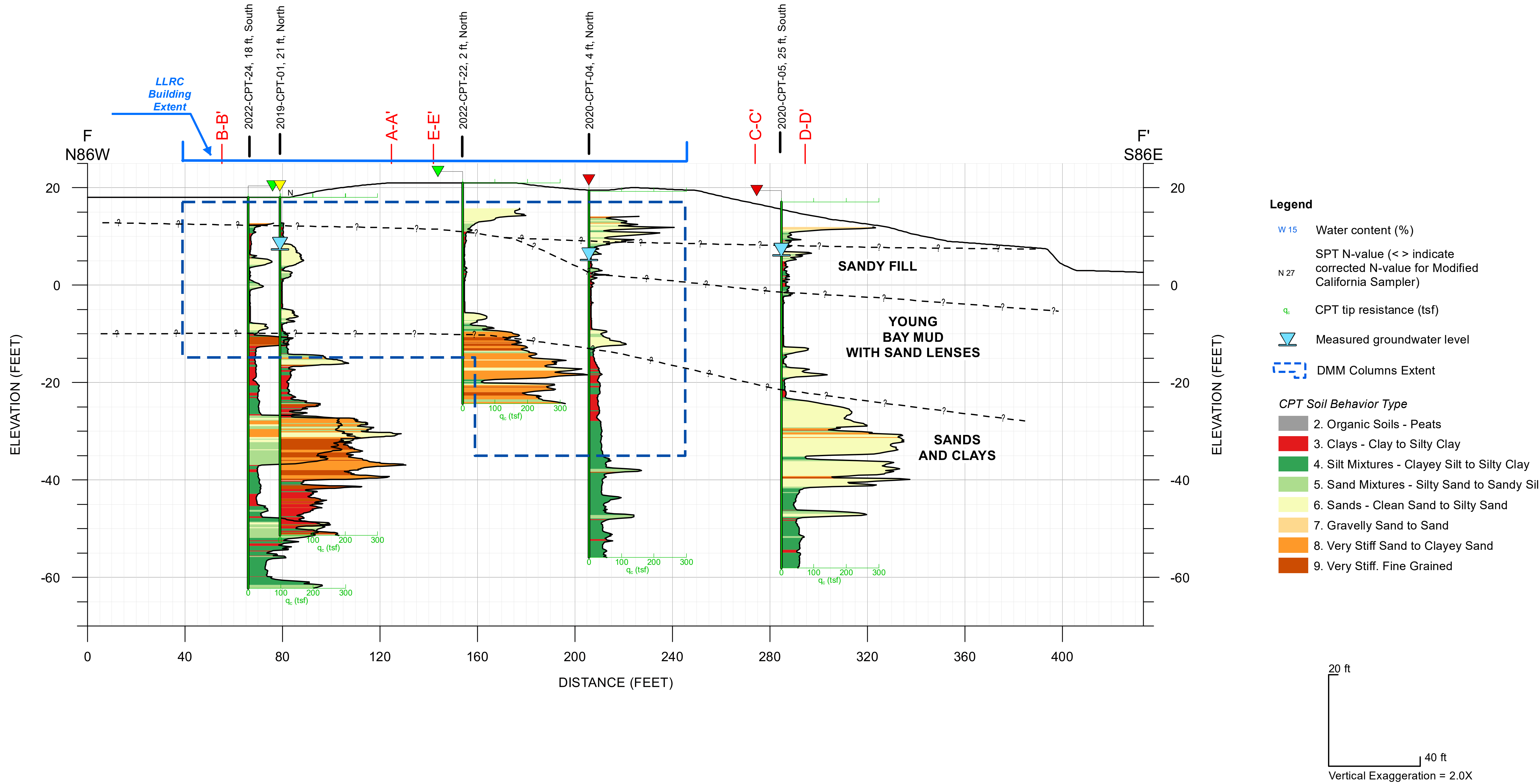


Plate I-2f: Cross Section F-F'

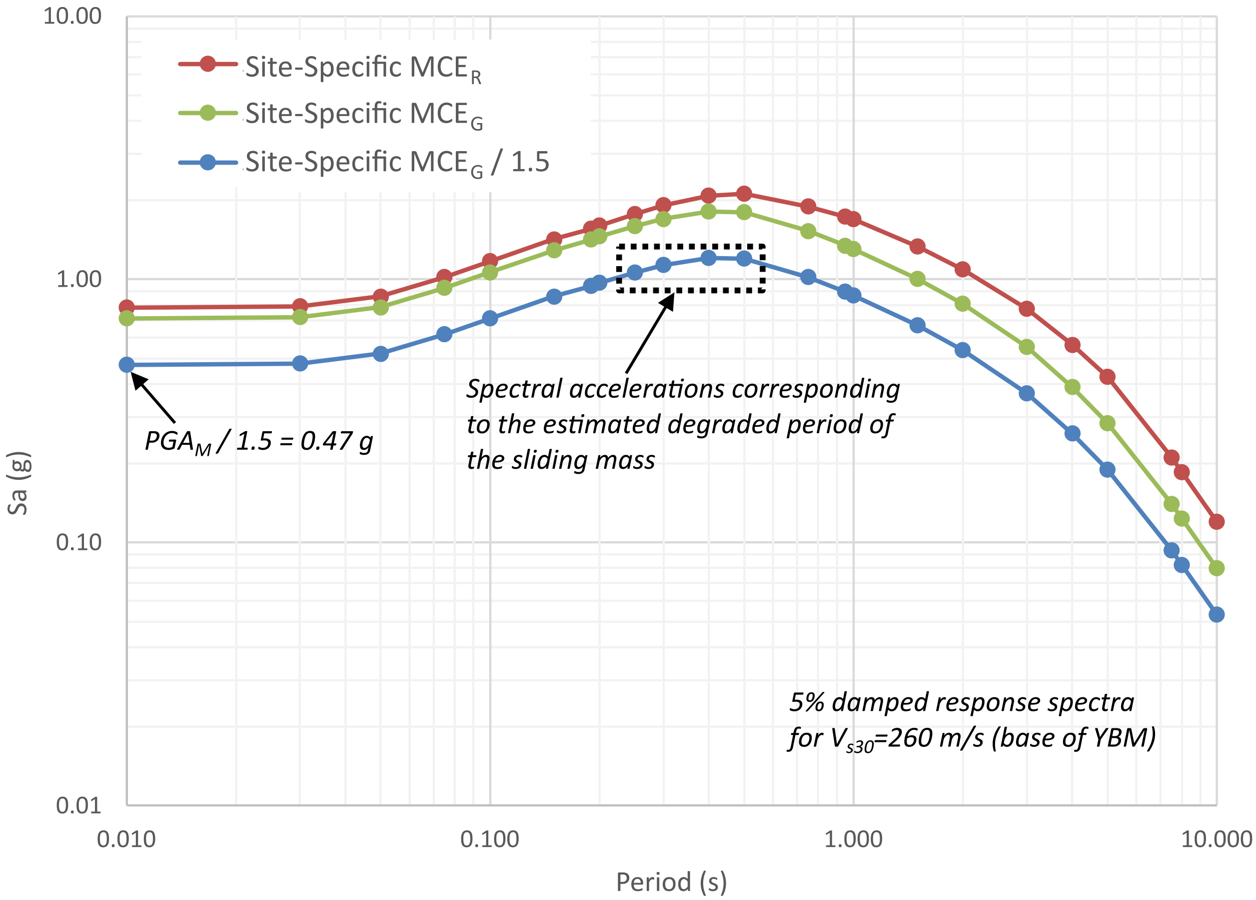


Plate I-3: Acceleration Response Spectra at Base of YBM (V_{s30} = 260 m/s)

Title: Laney College Library Learning Resource Center
File Name: Section F_rev03.gsz
Name: DSM_I_b_mp
Horz Seismic Coef.: 0.35
Method: Morgenstern-Price
Date: 05/04/2022

Color	Name	Model	Unit Weight (pcf)	Minimum Strength (psf)	Tau/Sigma Ratio	Undrained Shear Strength vs Vertical Effective Stress Function	Cohesion' (psf)	Phi' (°)
	DSM Composite	Mohr-Coulomb	120				5,000	0
	Post-Liquefaction Sand (K&W)	SHANSEP	110	0		Kramer and Wang (2015) N160=15		
	Sand and Clay	Mohr-Coulomb	130				0	40
	Sandy Fill	Mohr-Coulomb	120				0	35
	Young Bay Mud (During Earthquake)	SHANSEP	90	200	0.22			

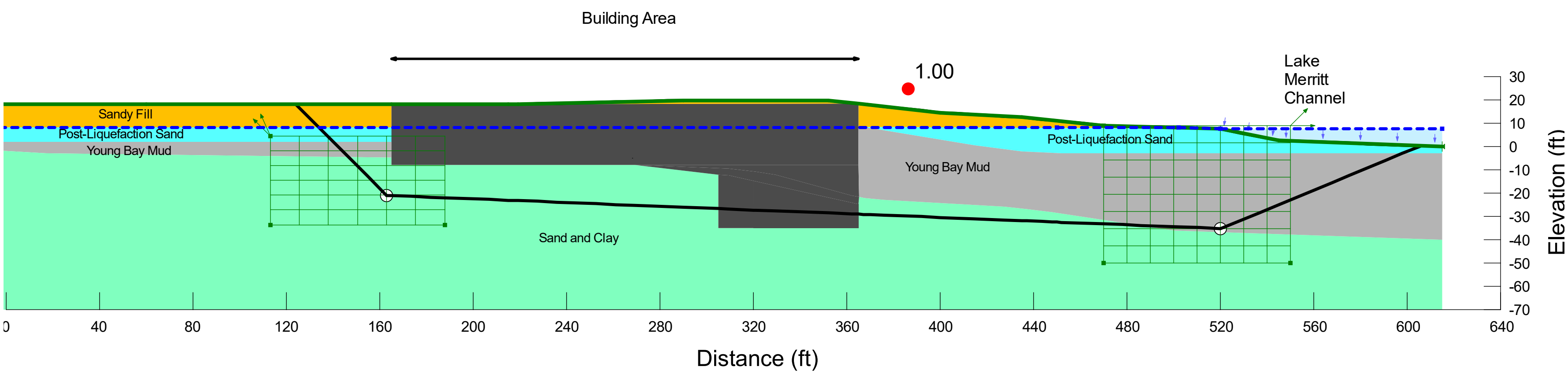


Plate I-4a: Pseudostatic Slope Stability Analysis for DMM Ground Improvement for k_h = 0.35 and Block Slip Surface

Title: Laney College Library Learning Resource Center
File Name: Section F_rev03.gsz
Name: DSM_I_c_mp
Horz Seismic Coef.: 0.35
Method: Morgenstern-Price
Date: 05/04/2022

Color	Name	Model	Unit Weight (pcf)	Minimum Strength (psf)	Tau/Sigma Ratio	Undrained Shear Strength vs Vertical Effective Stress Function	Cohesion' (psf)	Phi' (°)
	DSM Composite	Mohr-Coulomb	120				5,000	0
	Post-Liquefaction Sand (K&W)	SHANSEP	110	0		Kramer and Wang (2015) N160=15		
	Sand and Clay	Mohr-Coulomb	130				0	40
	Sandy Fill	Mohr-Coulomb	120				0	35
	Young Bay Mud (During Earthquake)	SHANSEP	90	200	0.22			

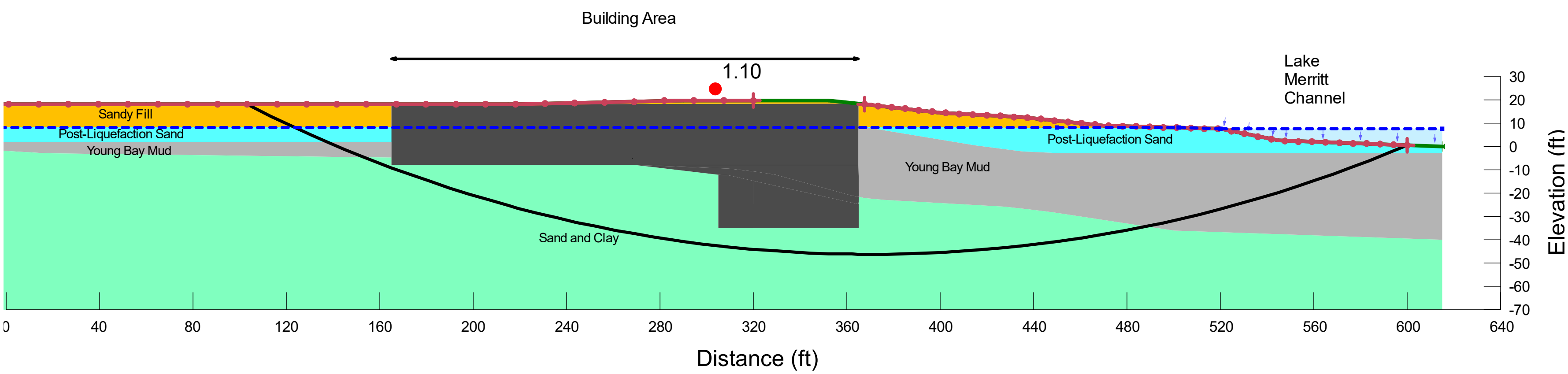


Plate I-4b: Pseudostatic Slope Stability Analysis for DMM Ground Improvement for k_h = 0.35 and Circular Slip Surface

Title: Laney College Library Learning Resource Center
File Name: Section F_rev03.gsz
Name: DSM_I_c_mp
Horz Seismic Coef.: 0.35
Method: Morgenstern-Price
Date: 05/04/2022

Color	Name	Model	Unit Weight (pcf)	Minimum Strength (psf)	Tau/Sigma Ratio	Undrained Shear Strength vs Vertical Effective Stress Function	Cohesion' (psf)	Phi' (°)
	DSM Composite	Mohr-Coulomb	120				5,000	0
	Post-Liquefaction Sand (K&W)	SHANSEP	110	0		Kramer and Wang (2015) N160=15		
	Sand and Clay	Mohr-Coulomb	130				0	40
	Sandy Fill	Mohr-Coulomb	120				0	35
	Young Bay Mud (During Earthquake)	SHANSEP	90	200	0.22			

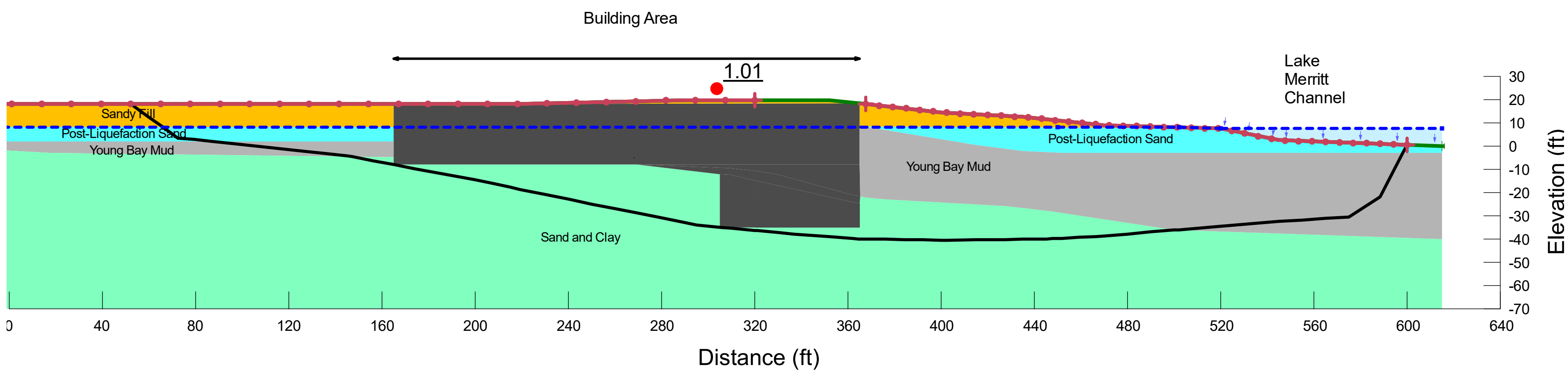


Plate I-4c: Pseudostatic Slope Stability Analysis for DMM Ground Improvement for k_h = 0.35 and Optimized Circular Slip Surface

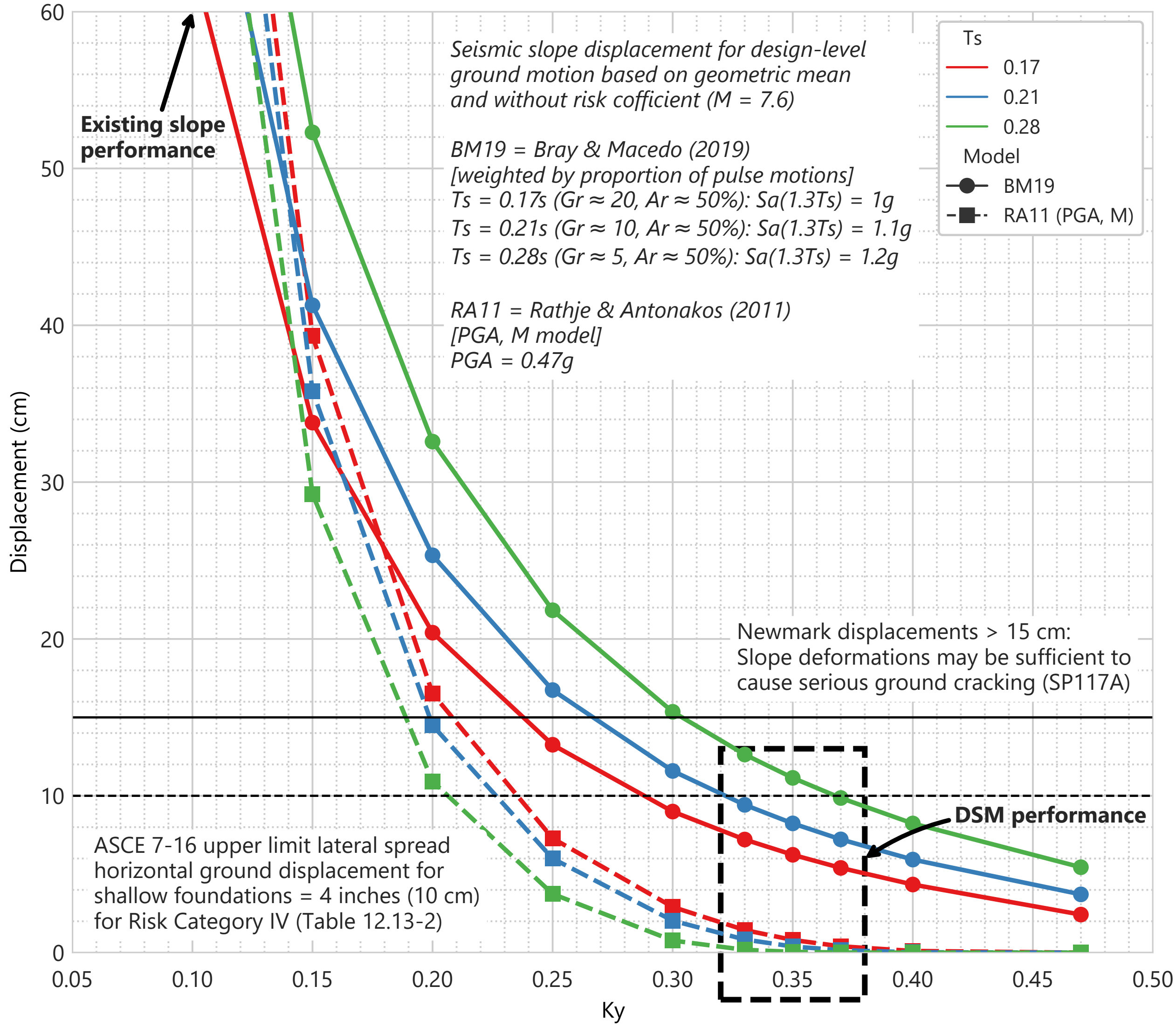


Plate I-5: Seismic Slope Displacement vs. Yield Coefficient

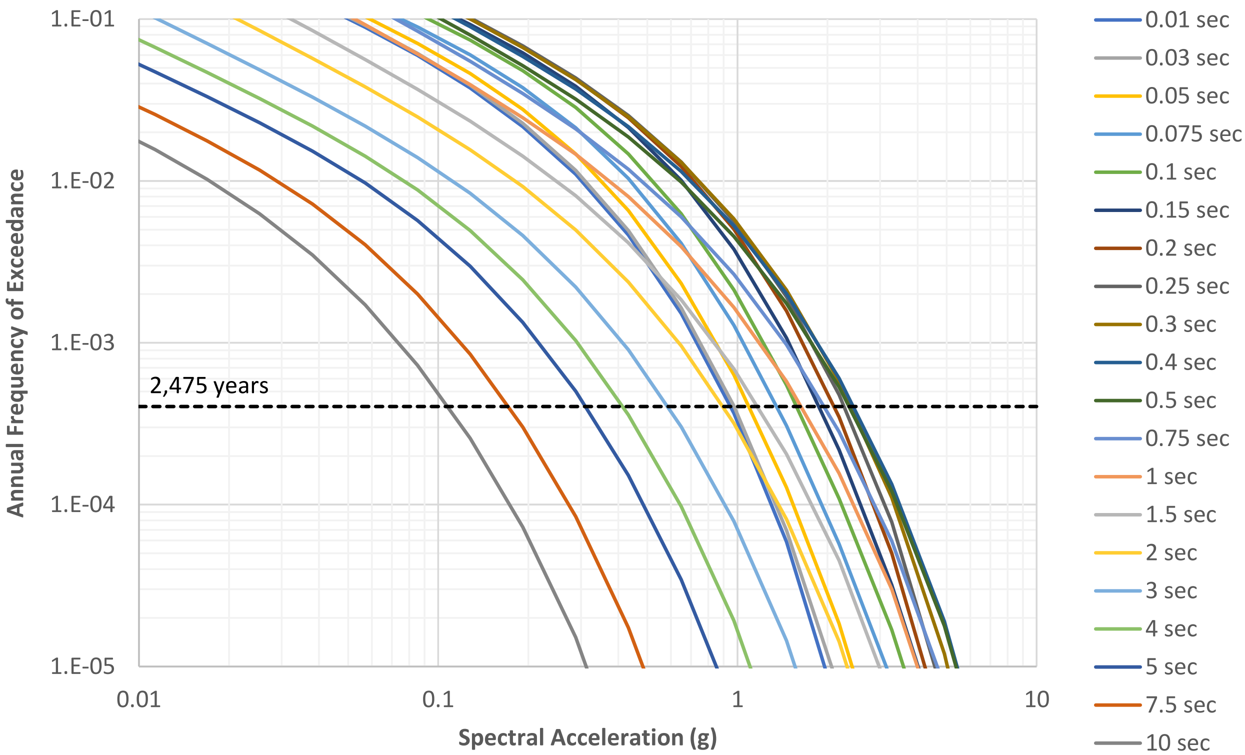


Plate I-6: Mean Annual Seismic Hazard Curves for V_{s30} of 270 m/s

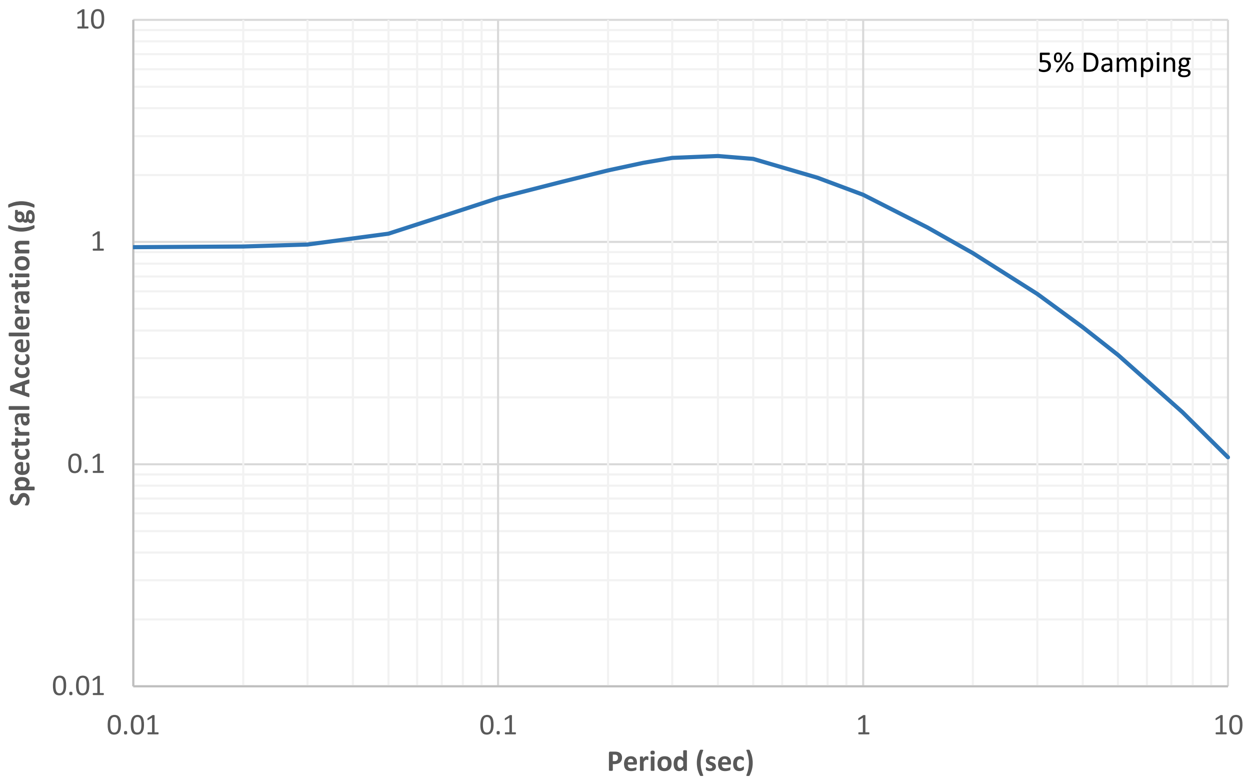
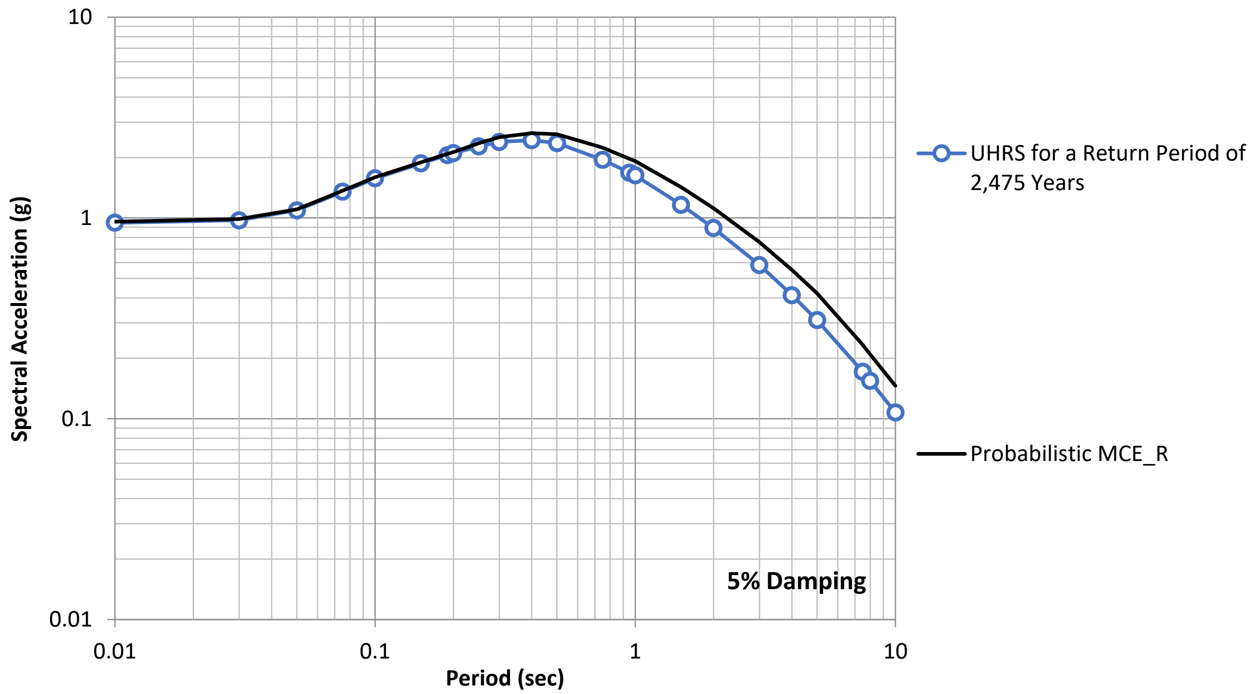


Plate I-7: Mean Horizontal Uniform Hazard Response Spectrum for a Return Period of 2,475 Years and V_{s30} of 270 m/s



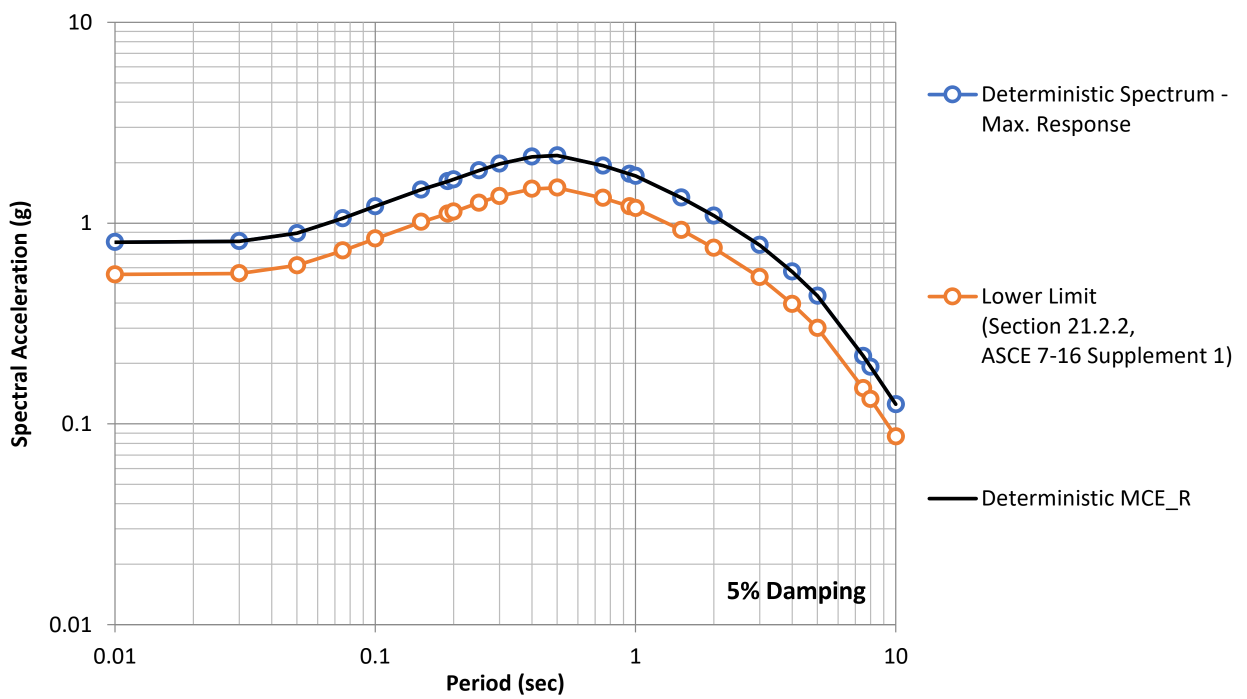
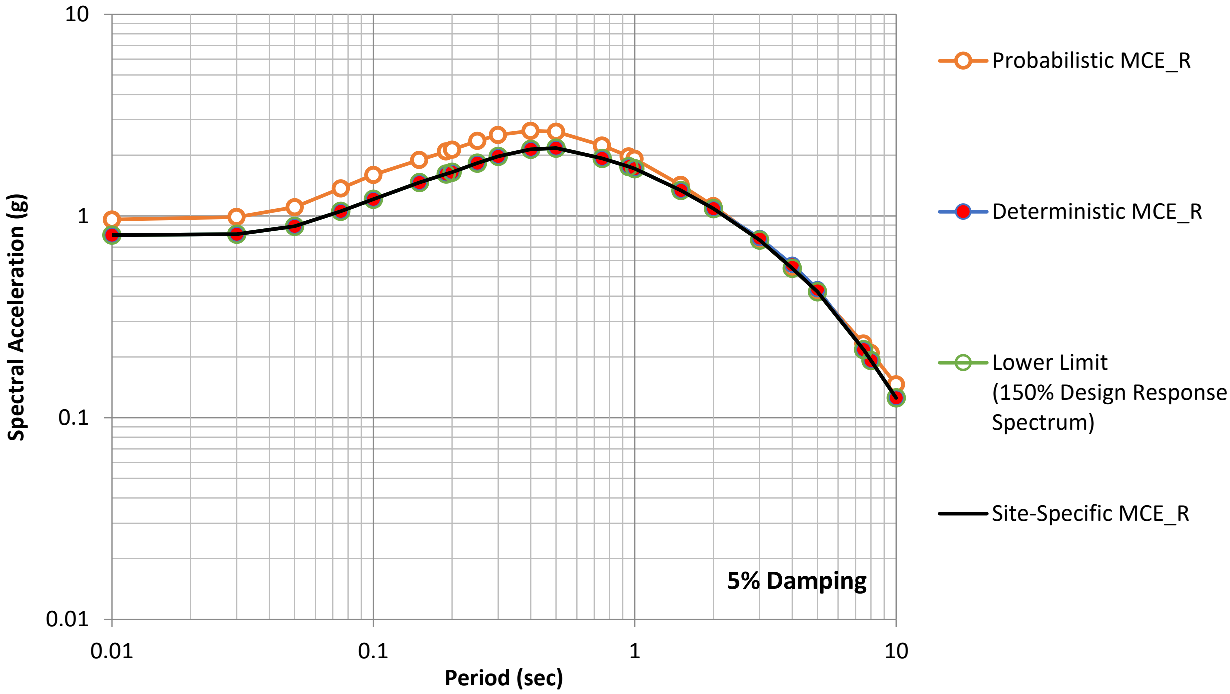


Plate I-8: Calculation of the Probabilistic and Deterministic Horizontal MCER Response Spectra per ASCE 7-16 for V_{s30} of 270 m/s



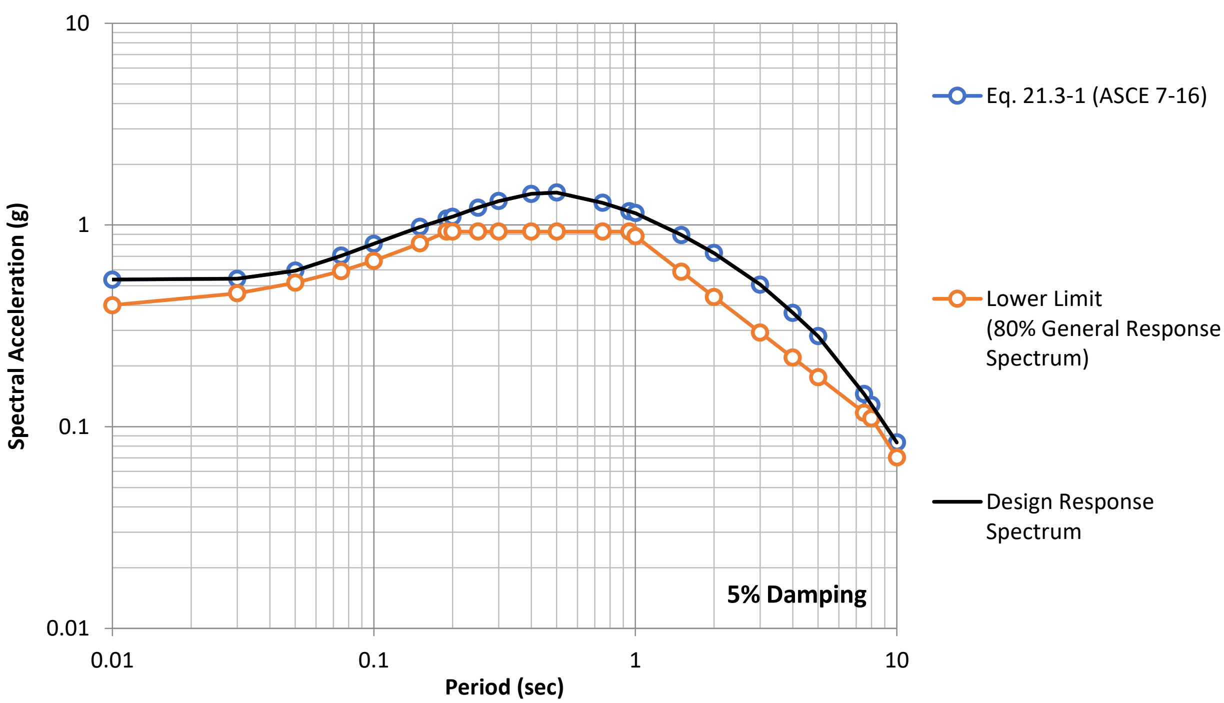


Plate I-9: Calculation of the Site-Specific Horizontal MCER and Design Response Spectra per ASCE 7-16 for V_{s30} of 270 m/s

Supplement J

Construction Specifications for

Deep Mixing

Deep Mixing Method (DMM)

J.1 Part 1 – General

J.1.1 Scope

1. The Deep Mixing Method (DMM) Contractor shall furnish all plant, equipment, labor, and materials required to construct and perform Quality Control of the DMM in accordance with the DMM Design Plans and Specifications.
2. The purpose of DMM ground improvement is to reduce seismically induced slope displacements and settlements to acceptable levels and to provide vertical and lateral support for shallow foundation systems for the Laney College Library & Learning Resource Center (LLRC) building for both static and seismic loadings. The DMM ground improvement consists of continuous underground overlapping deep mixed (DM) columns forming a series of DM walls that are arranged in a grid pattern to form a series of DM walls and blocks. The dimensions and layout of DM grids and blocks are shown on the DMM Design Plans and are described in **Section A.3.2** of this Specification.
3. This specification has been developed as a combination of performance and method specifications. The intent is that the DMM Contractor will select the means and methods for satisfying the acceptance criteria. The DMM Contractor will then demonstrate that the means and methods will satisfy the acceptance criteria using one or more test sections. Once the test section indicates satisfactory results, as determined by the Geotechnical Engineer, the DMM Contractor will follow the means and methods used to satisfy the acceptance criteria for all of the production DMM construction. If the DMM Contractor desires to change the means and methods during the course of production DMM, the changes need to first be approved by the Geotechnical Engineer and CGS. The Geotechnical Engineer may require additional test sections prior to approval of changes in means and methods.
4. The DMM Contractor shall be responsible for performing Quality Control (QC) during DMM construction, which includes QC documentation preparation and submittal, and sample collection, storage, and transportation. Sample testing shall be performed by a DSA approved testing laboratory hired by the Owner to verify that the acceptance criteria are satisfied. The Geotechnical Engineer shall make the determination as to whether the acceptance criteria have been met.
5. Upon completion of DMM installation, an as-built submittal package shall be prepared by the DMM Contractor and the Geotechnical Engineer to document that the installed DMM meets the project performance requirements. The as-built submittal package shall be submitted to CGS for approval. The submittal shall include test section results, daily quality control reports, DMM core and lab test results, as-built DMM record drawings, and any other information needed to document the work.

J.1.2 References

1. American Concrete Institute (ACI)
2. American Society of Testing and Materials (ASTM)
3. American Petroleum Institute (API)

J.1.3 Definitions

1. Area Replacement Ratio (Ar): A ratio of the surface area of soil-cement to the total surface area of ground to be improved within a given Treatment Zone. The total area of each Treatment Zone is measured to the outer tangent lines of the DMM columns along the entire Treatment Zone perimeter.
2. DMM: In situ ground treatment in which soil is blended with cementitious and/or other binder materials to improve strength, permeability, and/or compressibility characteristics (synonym terms include DSM, deep mixing, CDSM, and soil cement mixing).
   a. The DMM grids and blocks are formed by an arrangement of at least two soil mixing shafts with overlapping augers and blades (paddles), guided by a lead mounted on a crawler base machine.
   b. The mixing shafts shall be driven by a power source sufficient to provide torque for the wide range of expected drilling conditions, indicated by the available boring and CPT logs and other test data included in the Geotechnical Investigation Report (GIR) and planned future CPT logs prior to construction.
   c. As the mixing shafts are advanced into the soil, grout is pumped through the hollow stem of the shafts and injected into the soil at the shaft tips. Auger flights and mixing blades on the shafts blend the soil with grout in a pugmill fashion. When the design depth is reached, the mixing shafts are withdrawn while the mixing process is continued.
   d. The mixing shafts are positioned so as to overlap one another to form continuously mixed overlapping columns. After withdrawal, two (or more) overlapping soil-cement columns remain in the ground.
   e. The process is then repeated to form grids and blocks of overlapping DMM columns.
3. DMM Design Addendum (DA): DMM Design Addendum No. 1 prepared by Fugro USA Land, Inc. dated June 10, 2022, and subsequent addenda.
4. DMM Elements: DMM columns will be used to create DMM grids and DMM blocks of treated soil referred to as ground improvement. A DMM grid will consist of interconnected DMM walls formed by partially overlapping columns arranged in a grid pattern with a replacement ratio less than 100 percent. A DMM block used to support a building footing will consist of interconnected DMM walls formed by overlapping columns arranged in a parallel pattern with a replacement ratio of 100 percent or less as shown in project plans and drawings. For this project, individual DMM Element refers to the grouping of columns installed simultaneously during single penetration of the DMM rig.
5. DMM Contractor: The firm performing the DMM construction.

6. DMM Layout Plan: The alternate DMM construction layouts designed by the DMM Contractor, which satisfy the requirements of this Specification. The DMM Layout Plans shall be reviewed and approved by the Geotechnical Engineer and Structural Engineer.
7. Cement Dosage: The amount of cement (in terms of dry weight) used to treat a given initial volume of in-situ soil.
8. Cone Penetrometer Test (CPT): A geotechnical exploration tool, as defined in ASTM D 5778.
9. Core Run: The total length reported by the driller as the actual depth penetrated by coring, including both recovered and unrecovered lengths.
10. Geotechnical Investigation Report (GIR): Geotechnical Investigation Report prepared by Fugro USA Land, Inc. dated February 28, 2020, and subsequent addenda. Note the DMM Design Addendum No. 1 (DA) supersedes the GIR.
11. Geotechnical Engineer: The geotechnical engineer of record responsible for the DMM design, who is hired by the Owner.
12. Ground Improvement Area: The plan area contained within a single perimeter shown on the DMM Design Plans that surrounds:
    a. All planned soil-cement grids/blocks.
    b. Unmixed soil within the grids.
13. Grout: A stable colloidal mixture of water, Portland cement, and admixtures. The purpose of the grout is to assist in loosening the soils for penetration and optimum mixing, and upon setting, to strengthen the in-situ soil.
14. Grout-Soil Ratio: A volumetric ratio of grout to in-situ soil to be mixed.
15. Owner: Peralta Community College District and its representatives.
16. Structural Engineer: The structural engineer of record responsible for designs of structure foundations supported by DMM, who is hired by the Owner.
17. Testing Laboratory: The testing laboratory of record performing construction material testing, which is hired by the Owner and approved by the Geotechnical Engineer. The Testing Laboratory shall be selected from the DSA approved laboratory list.
    a. Treatment Zone: A spatial zone of soil targeted for ground improvement. The vertical and lateral (horizontal) extents of the Treatment Zones are defined on the DMM Design Plans.

J.1.4 Submittals

1. Evidence of conformance to the referenced standards and requirements shall be submitted by the DMM Contractor to the Geotechnical Engineer for the following, but not limited to, in accordance with the requirements in this Specification.
    a. Cement: Certificate of compliance for each truck load delivery.
    b. Admixtures: If used, certificate of compliance for each load or lot of material delivered.
    c. Preliminary Mix Design: Proposed mix designs including all materials and quantities and documentation of calibration of the grout mixing plant.

d. Proposed Test Section Program, Sampling Plan, and Laboratory Testing Program, conforming to the requirements described in this Section.
e. Construction Schedule: Submit a detailed schedule that identifies start dates and duration of each major task in the work. The schedule shall at a minimum include information regarding equipment mobilization, equipment setup, soil-cement mixing test section, production installation, and verification testing.
f. Site Work Plan: Submit a site plan showing staging area for all on-site equipment, including anticipated sections of the streets which may require blocking of parking spaces or traffic clearances.
g. DMM Layout Plans: Submit 1”=20’ scale drawings showing proposed layout of DMM Elements (including test section(s) and production DMM), including column diameters, column overlap, grid sizes, tip elevations, top elevations, coordinates of the corners, foundations, and proposed column and element numbering scheme prior to site mobilization in hard copy and electronic format using the project coordinate system at least 14 calendar days prior to beginning DMM construction. The DMM Contractor must obtain the Geotechnical Engineer’s approval of the proposed column layout prior to beginning DMM construction.
h. Equipment and Procedures: Submit a detailed description of the equipment and procedures to be used during all DMM work including, but not limited to, construction of DMM test section(s), production DMM work, and collecting samples for laboratory confirmation testing. Procedures shall include methods for locating the DMM Elements in the field and confirming that the columns are plumb. In addition, while it is recognized that the specific responses to field difficulties are dependent on several factors, the DMM Contractor shall submit their anticipated responses to the following possible situations that could occur during construction and testing of the DMM columns including poor core sample recovery or inability to retrieve core samples, and failing production test results (e.g., repair and/or treatment of failed area and modification to approved procedures or mix design).
i. The DMM Contractor shall also submit the anticipated cement dosages (proportions) to achieve the acceptance criteria outlined under acceptance criteria in **Section A.3.15** of this Specification.
j. Quality Control Program, as outlined in the Execution Section of this Specification.
k. Daily Quality Control Reports: Prior to construction, submit a proposed Daily Quality Control Report format for approval by the Geotechnical Engineer. Submit the Daily Quality Control Report at the end of the next working day. The report should be in conformance with quality control in **Section A.3.14** of this Specification.
l. DMM Test Results: Submit all QC test results as outlined in quality control in **Section A.3.14** of this Specification.

m. Calibrations: Submit all metering equipment calibration test results including mixing systems, delivery systems, alignment systems, and mixing tool rotational and vertical speed.
n. Record Drawing: Submit record drawings prepared by the DMM Contractor indicating the as-built location and elevations of the DMM Elements in terms of project coordinates and vertical datum. The record drawings shall also indicate the above structure foundation designs and locations.

2. Upon completion of DMM installation, the as-built submittal by the DMM Contractor to the Geotechnical Engineer, Structural Engineer, and CGS shall include test section results, daily quality control reports, DMM core and lab test results, DMM record drawings, and any other information needed to document the work.

J.2 Part 2 – Products, Materials, and Equipment

J.2.1 Materials

1. Grout: The material added to the blended in situ soils shall be a water-based Portland cement grout. The purposes of the grout are to assist in loosening the soils for penetration and optimum mixing, and upon setting, to strengthen the in-situ soils. The grout shall be premixed in a mixing plant which combines dry materials and water in predetermined proportions.
2. Cement used in preparing the grout shall conform to ASTM C150 "Standard Specification for Portland Cement Type II". The cement shall be adequately protected from moisture and contamination while in transit to and in storage at the job site. Reclaimed cement or cement containing lumps or deleterious matter shall not be used.
3. Water: Fresh water, free of deleterious substances that adversely affect the strength and mixing properties of the grout, shall be used to manufacture grout.
4. Admixtures: Admixtures are ingredients in the grout other than Portland cement, and water. Admixtures of softening agents, dispersions, pozzolans, retarders or plugging or bridging agents may be added to the water or the grout to permit efficient use of materials and proper workability of the grout. However, no admixtures shall be used except as approved by the Geotechnical Engineer.

J.2.2 Equipment

The DMM equipment shall meet the following requirements:

1. The mixing tools shall have mixing augers and blades (paddles) configured in such a manner so that they are capable of thoroughly blending the in-situ soils and grout.
   a. Multi-shaft mixing equipment (machines with at least two soil mixing shafts with overlapping augers and blades) shall be used.

   i. The mixing augers and blades shall be minimum 3 feet and maximum of 6 feet in diameter.
   ii. Allowable wear to mixing augers and blades will be limited such that equipment produces a column no less than the design diameter listed on the DMM Layout Plans.
   iii. The overlapping between any two adjacent DMM columns shall be at least 30 percent of the column diameter.
 b. The power source for driving the mixing shafts shall:
   i. be sufficient to provide torque for the wide range of expected drilling conditions, indicated by available boring and CPT logs and other test data included in the Geotechnical Investigation Report (GIR) and DMM Design Addendum (DA).
   ii. be sufficient to maintain the required revolutions per minute (RPM) and penetration rate from a stopped position at the maximum depth required.
2. The DMM rig shall be equipped with electronic sensors built into the leads to determine vertical alignment in two directions: fore-aft and left-right.
   a. The sensors shall be calibrated at the beginning of the project and the calibration data shall be provided to the Geotechnical Engineer. The calibration shall be repeated at intervals not to exceed three months per rig.
   b. The output from the sensors shall be routed to a console that is visible to the operator and the Geotechnical Engineer during penetration and reported. The console shall be capable of indicating the alignment angle in each plane.
3. The DMM equipment shall be adequately marked to allow the Geotechnical Engineer to confirm the penetration depth to within 6 inches during construction.
4. The grout shall be premixed in an on-site mixing plant, using a batch process, which combines dry materials and water in predetermined proportions. The mixing plant shall consist of a grout mixer, grout agitator, grout pump, batching scales, and a computer control unit.
   a. Dry materials shall be stored in silos. The dry materials shall be transported to the project site and blown into the on-site storage tanks using a pneumatic system.
   b. The air evacuated from the storage tanks during the loading process shall be filtered before being discharged to the atmosphere.
   c. Automatic batch scales shall be used to accurately determine mix proportions for water and cement during grout preparation.
   d. The dry admixtures, if used for mixing with water and cement, can be delivered to the mixing plant by calibrated auger. However, the DMM Contractor shall demonstrate that the calibrated auger can deliver the quantity of dry admixture with accuracy equivalent to that measured and delivered by weight.

   e. Calibration of mixing components shall be done at the beginning of the project and repeated at intervals not to exceed three months thereafter and after each move of the batch plant.
5. Positive displacement pumps shall be used to transfer the grout from the mixing plant to the mixing tool head. The grout shall be delivered to each slurry-injecting tool head by an individual positive displacement pump.
6. The DMM rig shall be equipped with sensors to continuously monitor and record the mixing tool penetration/withdrawal speed, mixing tool rotation speed, and injection rate.
   a. The output from these sensors shall be visible to the Operator and Geotechnical Engineer during penetration and withdrawal.
   b. The DMM Contractor may propose alternative display/monitoring systems; however, the systems shall first be reviewed and approved by the Geotechnical Engineer prior to use.
   c. Calibration of this equipment shall be performed at the beginning of the project and the calibration data shall be provided to the Geotechnical Engineer. The calibration shall be repeated at intervals not to exceed three months.

J.2.3 Products

1. DMM: The in-place grout mix together with the soils shall meet all of the acceptance criteria specified in **Section A.3.15** of this Specification, determined according to the quality control, sampling, and testing methods specified in **Section A.3.14** of this Specification.

J.3 Part 3 – Execution

J.3.1 Observation of Work

1. The work covered by these specifications shall be performed under the observation of the Geotechnical Engineer, who shall be retained and paid by the Owner. The Geotechnical Engineer will be present at the site during the conduct of work to observe the work, and to perform field and laboratory tests, as deemed necessary by the Owner. The DMM Contractor shall cooperate with the Geotechnical Engineer in performing the observations and tests. At the completion of their work, the Geotechnical Engineer shall submit a report to the Owner, including a tabulation of all tests performed. The Geotechnical Engineer's costs for observing the construction, testing, and the repair of unsatisfactory work performed by the DMM Contractor shall be billed to the Owner. The Owner shall pay them and then shall deduct the amount from monies due to the DMM Contractor.
2. This work falls under the jurisdiction of the California Division of State Architect (DSA) who will review submittals and may observe portions of the work.

J.3.2 General

1. The soil-mixing shall be constructed by the DMM Contractor to the lines, grades, and cross sections indicated on the DMM Design Plans example layouts or an alternate layout approved by the Geotechnical Engineer, Structural Engineer, and CGS. Revisions to the approved layouts shall be submitted to Geotechnical Engineer of Record (GEOR) for review and approval. DMM ground improvement within a single structure shall be arranged in an uninterrupted grid that follows the structure column lines and underlies all footings and moment frame grade beams, tie beams, and shear walls as shown on the DMM Design Plans.
2. As shown on the DMM Design Plans, the DMM shall underlie the entire structure footprints and extend laterally to include any attached structures which are deemed to be essential parts of the structures.
3. Grading after the site demolition may be required to provide suitable level ground for constructing the DMM. The DMM contractor is responsible for coordinating with the site grading operation to define the Drill-Through Zone.
4. The minimum Area Replacement Ratio (Ar) for DMM grids and blocks and the maximum spacing for DMM grids depends on the specified unconfined compressive strength ($q_{dm,spec}$) as shown in **Table A.1**. Additional DMM Elements may be added within the untreated area to meet or reduce the slab free span distance as instructed by the Geotechnical and Structural Engineer.

Table A.1: Minimum DMM A_r and Maximum DMM Grid Spacing

Specified Unconfined Compression Strength, $q_{dm,spec}$ (psi)	Minimum A_r (%)	Zone B Maximum DMM Center-to-Center Grid Spacing[1]	Zones A1 and A2 Maximum DMM Center-to-Center Grid Spacing[1]
125	50	3.2d	4.0d

[1]d = DMM column diameter

5. DMM elements shall extend to at least the elevations indicated on the DMM Design Plans based on the penetration of the shortest mixing shafts.
6. The top of the DMM shall extend to the base of the ground floor slab section, the bottom of footings, the bottom of moment frame grade beams, and the bottom of elevator pits, as indicated in the DMM Design Plans.
7. Any proposed plan and Area Replacement Ratio by the DMM contractor should be approved by the Geotechnical Engineer and CGS.
8. Elevator pits shall be supported entirely by DMM.

9. The DMM columns shall be essentially vertical columns as stated in this Specification, with a minimum diameter of 3 feet and a maximum diameter of 6 feet and shall extend from the top to the bottom of the Treatment Zone indicated on the DMM Design Plans.
10. The overlapping between any two adjacent columns at ground surface shall be a minimum of 30 percent of column diameter.
11. The completed DMM shall be a homogeneous mixture of grout and the in-situ soils. Mixing is to be controlled by shaft rotational speed, drilling speed, and grout injection rate.
12. Monitoring of construction parameters and confirmation testing will be used to verify that the acceptance criteria have been satisfied.
    a. The DMM Contractor shall establish consistent procedures to be employed during DMM construction to ensure a relatively uniform product is created.
    b. These procedures are to be defined in the equipment and procedures submittal as defined in **Section A.1.4** of this Specification and subsequently modified, if necessary, based on the results of the pre-production testing or quality control testing.
13. The DMM Contractor may request that the established grout mix/grout-soil ratio design, equipment, installation procedure, or test methods be modified. However, the Geotechnical Engineer may require additional testing, at no additional cost to the Owner, to verify that acceptable results can be achieved.
    a. The DMM Contractor shall not employ modified grout mix/grout-soil ratio design, equipment, installation procedures, or sampling or testing methods until approved by the Geotechnical Engineer in writing.
    b. The Geotechnical Engineer, at his sole discretion, may reject any modification proposed by the DMM Contractor.

J.3.3 Construction Site Survey

The location of both active and abandoned buried utilities at the site can have significant impact on the design and construction of deep mixing works. Careful consideration of the presence and location of all utilities is required.

1. Prior to bidding, the contractor should review the available subsurface information and visit the site to assess the site geometry, equipment access conditions, location of existing structures, and above-ground utilities and facilities.
2. The contractor should field locate and verify the locations of all utilities prior to starting work. The contractor should maintain uninterrupted service for those utilities designated to remain in service throughout the work. The contractor should notify the engineer of any utility locations different from those shown in the plans that may require relocation of deep mixed elements or structure design modification. Subject to owner's geotechnical engineer's approval, the contractor should be compensated for additional costs of element relocation and/or structure design modifications resulting from utility locations different from those shown in the plans.

J.3.4 Site Access for Soil Samples

1. After award of the Contract, the DMM Contractor will have the option of accessing the jobsite to collect additional soil samples for use in mix designs with the following requirements:
   a. Prior to commencing with field work, the DMM Contractor shall obtain all necessary permits for sampling activities, including drilling permits from Alameda County Public Work Agency, if applicable.
   b. The DMM Contractor shall submit to the Geotechnical Engineer a sampling plan indicating in detail the sampling activities proposed, and the proposed methods for backfilling boreholes or excavations and restoring the site.
   c. Cement grout backfill for boreholes per Alameda County Public Work Agency is required.
   d. The soil sampling and testing will be performed by DMM Contractor. The costs of additional soil sampling and testing (if performed) are to be included in the project DMM construction costs.

J.3.5 Test Sections

1. The DMM Contractor shall construct a minimum of one test section on site to demonstrate. that the proposed mix design, equipment, and procedures will meet the specified requirements. The location(s) of the test section(s) shall be determined by the DMM Contractor with the approval of the Geotechnical Engineer.
2. Additional test sections may be performed at the DMM Contractor's option to optimize the mix design and procedures.
3. Each test section must extend at least to the deepest DMM design depth as indicated by the DMM Design Plans.
4. The costs of the test section(s) are to be included in the project DMM construction costs.
5. Each test section shall consist of at least two full strokes of the DMM equipment. For example, if the DMM rig uses three augers, then the test section shall consist of 2 strokes times 3 columns equal 6 columns.
6. Test sections shall not be located directly below proposed footings, moment frame grade beams, and elevator pits. However, the test sections may be constructed in place of other production DMM columns, provided it is later demonstrated that the test sections meet all acceptance criteria. If the test sections are found to fail the acceptance criteria, the DMM Contractor shall make necessary repairs or replace the DMM Elements to the written satisfaction of the Geotechnical Engineer and CGS.
7. During the time interval between construction of the test section(s) and the completion of laboratory test results, the DMM Contractor may proceed with production DMM installation at their own risk. Any production DMM found to fail the acceptance criteria must be

repaired at the DMM Contractor's expense, to the written satisfaction of the Geotechnical Engineer and CGS.

8. A minimum of two (2) full-depth cores shall be obtained from each test section, according to the procedures detailed in this Specification.
9. Laboratory tests, as specified in this Specification, shall be performed on a minimum of ten samples per full-depth core or a minimum of one sample per core run, whichever is greater, from each test section, as selected by the Geotechnical Engineer. Additional cores may be performed to retrieve enough test samples.

J.3.6 Horizontal Alignment

1. The DMM Contractor shall accurately stake the location of DMM Elements using a surveyor before beginning installation. The main survey control for a given area shall be established by a California licensed surveyor; layout of individual DMM Elements does not require a licensed surveyor. Horizontal alignment of DMM columns shall conform to the geometric tolerances in the acceptance criteria of this Specification.
2. The DMM Contractor shall provide an adequate method to allow the Geotechnical Engineer to verify the as-built location of the DMM during construction.
3. Movement of the crawler base machine shall provide the preliminary alignment of the augers and the final alignment shall be adjusted by hydraulic manipulation of the leads.
4. One stroke of the machine shall construct a DMM Element consisting of at least two overlapping columns.
5. The DMM shall be advanced stepwise by overlapping the adjacent columns of the previous strokes.
6. Following DMM construction, the DMM Contractor shall submit as-built drawings indicating the location of the DMM elements in terms of project coordinates and elevation datum.
7. The DMM contractor should provide a construction plan at least two (2) weeks prior to the start of construction that includes the plan showing the numbering and location of the DMM columns, tip elevations or depths, and cut-off (top) elevations. The daily work plan should be provided to the Geotechnical Engineer at the beginning of workday and work progress should be checked and confirmed by the Geotechnical Engineer during and at the end of each day. The DMM contractor should provide a summary progress report to the Geotechnical Engineer at the end of each workday.
8. The location of known obstructions or utilities at or near the treatment area should be marked on the project drawings and on the ground before construction begins. Existing obstructions within the treatment zone area should be removed prior to construction. It is not anticipated that drilling obstructions will be encountered within the Treatment Zone during DMM construction unless further site investigation reveals otherwise.
   a. If an obstruction preventing drilling advancement is encountered, the DMM Contractor shall investigate the location and extent of the obstruction using methods approved by

the Geotechnical Engineer. The DMM Contractor shall propose remedial measures to clear the obstruction for approval by the Geotechnical Engineer.

b. While the investigation for an obstruction is underway, the DMM Contractor shall continue to install columns in areas away from the obstruction location. No stand-by delay will be allowed for equipment and operations during the investigation of an obstruction.

c. The DMM Contractor will be compensated for removal or clearing of obstructions as a Changed Condition, paid in accordance with the General Conditions.

d. The DMM Contractor will not be compensated for removal or clearing of obstructions without prior approval by the Geotechnical Engineer and the Owner.

9. The DMM Contractor will not be compensated for DMM Elements that are located outside of the tolerances specified in the acceptance criteria.

J.3.7 Vertical Alignment

1. The equipment operator shall control vertical alignment of the auger stroke. Verticality shall be monitored with respect to two orthogonal horizontal axes. Vertical alignment of DMM columns shall conform to the geometric tolerances in the acceptance criteria of this Specification.

J.3.8 DMM Depth

1. DMM depths shall extend to the line and grades shown on the DMM Design Plans.
2. The total depth of penetration shall be measured either by observing the length of the mixing shaft inserted below a reference point on the mast, or by subtraction of the exposed length of shaft above the reference point from the total shaft length.
    a. For each stroke, the elevation of the reference point on the mast must be established within one inch using measurements from a surveyed control point.
    b. The final depth and bottom elevation of the stroke shall be noted and recorded on the Daily Quality Control Report by the DMM Contractor. The equipment shall be adequately marked to allow the Geotechnical Engineer to confirm the penetration depth during construction.
3. If rigs with varying mixing shaft lengths are used, the shortest shafts shall extend to the minimum DMM depths indicated on the DMM Design Plans.

J.3.9 Grout Preparation

1. Dry material shall be stored in silos and fed to mixers for agitation and shearing. In order to accurately control the mixing ratio of grout, the addition of water and cement shall be determined by weight using the automatic batch scales in the mixing plant.
    a. The admixtures, if used, for mixing with water and cement, can be delivered to the mixing plant by calibrated auger. However, the DMM Contractor shall prove that the

calibrated auger can deliver the quantity of dry admixture with accuracy equivalent to that measured and delivered by weight.

2. A minimum mixing time of one minute and a maximum holding time of four hours will be enforced for the grout.
   a. The grout hold time shall be calculated from the beginning of the initial mixing.
3. The specific gravity of the grout shall be determined during the design mix program for double checking grout proportions.
   a. The specific gravity of the grout shall be checked by the DMM Contractor at least twice per shift per rig using the methods outlined in ASTM D4380.
   b. The specific gravity of the grout measured in the field should not deviate by more than 3 percent of the calculated specific gravity for the design cement ratio.
   c. If the specific gravity is lower than that required by the design mix, the DMM Contractor shall add additional cement and remix and retest the grout at no cost or schedule impact to the Owner.
   d. The specific gravity measurements shall be indicated on the Daily Quality Control Report.

J.3.10 Soil-Grout Mixing

1. Installation of each column shall be continuous without interruption.
   a. If an interruption of more than one hour occurs, the column shall be remixed (while injecting grout at the design grout ratio) for the entire height of the element at no additional cost to the Owner.
   b. If an interruption of more than ten minutes occurs, the DMM Contractor shall inject a volume of grout equal to that required for three feet of auger penetration, while maintaining constant auger elevation. Once the specified volume of grout has been injected, auger penetration may continue.
2. The completed CSDM shall be a uniform mixture of cement grout and the in-situ soils.
   a. Soil and grout shall be mixed together in place by the specially designed overlapping augers or blades on the mixing shafts.
   b. The grout shall be pumped through the mixing shafts and injected from the tip of the shafts. The shafts shall break up the soil and blend it with cement grout.
   c. The mixing action of the shafts shall blend, circulate, and knead the soil over the length of the column while mixing it in place with the grout.

J.3.11 Shaft Rotational Speed and Penetration/Withdrawal Rate

1. The mixing shaft rotational speed (measured in RPMs) and penetration/withdrawal rates shall be established before beginning work. It may be adjusted with the approval of the Geotechnical Engineer to achieve adequate mixing.

2. The contractor shall obtain the suitable shaft rotational speed during the installation of test section. The rotational speeds and penetration/ withdrawal rates shall be recorded on the Daily Quality Control Report.
3. The established rotational speeds and penetration/withdrawal rates shall be used during the work. If these parameters are varied more than ten (10) percent from those determined during the test section(s), the Geotechnical Engineer may require additional testing, at no additional cost to the Owner, to verify that the acceptance criteria are met.
4. The DMM Contractor may request that the established mixing parameters be modified during the production DMM installation. To verify acceptable results for the modified parameters, the Geotechnical Engineer may require additional testing at no additional cost to the Owner.

J.3.12 Grout Injection Rate

1. The grout injection rate per no more than three vertical feet of column shall be in accordance with the requirements of the design mix.
   a. The required mix design and grout-soil ratio shall be determined during the test section(s).
   b. The grout injection rate shall be constantly monitored and controlled.
   c. The DMM Contractor shall record the volume of grout injected continuously for each column on the Daily Quality Control Report.
2. If the volume of grout injected per three vertical feet of column is less than the amount required to meet the grout-soil ratio established during the test section, the DMM columns shall be remixed and additional grout injected (at the design grout-soil ratio) to a depth at least 3 feet below the deficient zone or until design depth is met, at no additional cost to the Owner.
3. The DMM Contractor may request that the established grout-soil ratio be modified during the production DMM installation.
   a. To verify acceptable results for the modified grout-soil ratio, the Geotechnical Engineer may require additional testing or a new test section at no additional cost to the Owner.

J.3.13 Control of Spoils

1. The DMM Contractor shall control and process all spoils created during the DMM construction.
   a. Prior to stockpiling materials greater than 10 feet in height, stockpile locations and heights shall be submitted for review and approval to Geotechnical Engineer. The DMM Contractor shall consider the locations of and avoid damage to existing utilities, structures, and other improvements, as well as recently constructed DMM Elements when stockpiling material. Lesser stockpile heights may be necessary in some areas.

b. The spoils shall be processed until they have cured to a sufficient level to allow them to be stockpiled such that they will not reform a cemented mass in the stockpile. The DMM Contractor shall dispose of spoils in accordance with all local laws, codes, and ordinances in a manner acceptable to the Owner or coordinate with the project grading contractor for the spoils be reused as fills at the project site.

J.3.14 Quality Control Program

1. The DMM Quality Control Program shall be the responsibility of the DMM Contractor and shall include, as a minimum, the following components:
   a. An approved pre-construction test program on soils obtained from the project site, to establish appropriate design parameters such as cement dosage and water content.
   b. Field monitoring by the DMM Contractor of construction parameters during DMM construction.
   c. Sample collection including full depth continuous coring, sample storage, and sample transportation to the Testing Laboratory.
   d. Reporting of the field monitoring and sampling performed by the DMM Contractor.
   e. Reporting of the core strength testing performed by the Testing Laboratory.
2. Prior to site mobilization, the DMM Contractor shall submit a detailed work plan for the Quality Control Program for review and approval by the Geotechnical Engineer. The work plan shall include, as a minimum:
   a. A description of all installation, monitoring, sampling, and testing procedures to be implemented. The proposed auger penetration and withdrawal rates shall be proposed by the DMM Contractor at this time.
   b. Descriptions of all sampling equipment.
   c. A list of parameters to be monitored.
   d. Tolerances for the parameters monitored.
   e. Names of any subcontractors.
3. The DMM Contractor shall provide all the personnel and equipment necessary to implement the Quality Control Program. Contractor to provide the number of years/projects, project descriptions, and reference list for all cases below:
   a. The DMM Contractor must have at least 7 (seven) years of previous successful experience with at least 5 (five) DMM projects for soil conditions and project scope similar to that of the project being bid.
   b. The DMM contractor must have a registered California Professional Engineer (PE) who have had at least 5 (five) years of experience with at least 3 (three) DMM projects.
   c. The DMM Contractor must have assign a project manager who have had at least 5 (five) years of experience on at least 3 (three) DMM projects.
   d. The DMM Contractor must have assign a project engineer/ supervisor who have had at least 3 (three) years of experience with at least 2 (two) DMM projects.

   e. The DMM Contractor must assign a full-time project superintendent with at-least 3 (three) DMM projects with at least 150,000 cubic yard of total treatment volume in DMM construction.
   f. The DMM equipment operator must have at least three years of experience with the equipment and DMM construction.
   g. Written requests for substitution of these key personnel must be submitted prior to personnel changes. Documentation must be submitted to the owner that demonstrates that the substitute meets the requirements listed. Substitution may not be made until written approval is provided by the owner.
4. The Geotechnical Engineer will continuously observe the DMM construction. The Geotechnical Engineer will review DMM Contractor submittals to check that the Quality Control Program is being properly implemented.
5. The established quality control procedures shall be maintained throughout the production DMM installation to ensure consistency in the installation and to verify that the work complies with all requirements indicated in the DMM Design Plans and Specifications, unless modifications to the procedures are approved in writing by the Geotechnical Engineer.
6. DMM Contractor shall perform sample collection, storage, and transportation.
   a. DMM Contractor shall collect one full-depth continuous coring should be made for every 3% of the total DMM elements or for every 900 square feet of treated ground, whichever produces the greater number of cores at locations specified by the Geotechnical Engineer.
      i. The coring rig shall be a triple-barrel rig approved by the Geotechnical Engineer and capable of achieving the required recovery. The ability to achieve the recovery criteria is solely the Contractor’s responsibility.
      ii. Full-depth samples obtained by the DMM Contractor shall have a diameter of at least 3 inches.
      iii. The continuous core sample shall extend from the top through the bottom of the Treatment Zone, and to at least 5 feet below the Treatment Zone to sample the foundation soil directly below the Treatment Zone.
      iv. Unless otherwise directed by the Geotechnical Engineer, the full-depth samples shall be obtained along an essentially vertical alignment located one-fourth of a column diameter from the column center and not within column overlaps.
      v. The DMM Contractor shall perform all full-depth sampling in the presence of the Geotechnical Engineer.
      vi. Full-depth core samples shall be retrieved using triple tube continuous coring techniques after the soil-grout mixture has hardened sufficiently.
      vii. Each core run shall be a minimum 4 feet in length.

viii. Following logging, the engineer will select at least five specimens from each full-depth continuous core for strength testing. Each test specimen should have a length-to-diameter ratio of 2 or greater.

ix. A minimum recovery of 85 percent for each 4-foot core run shall be achieved for cores from within the Treatment Zones. During coring, the elevation of the bottom of the holes shall be measured after each core run in order that the core recovery for each run can be calculated.

x. The DMM Contractor shall determine the time interval between column installation and coring except that the interval shall be no longer than required to conduct 28-day strength testing.

xi. The DMM contractor should photograph each core run and submit to the Geotechnical Engineer for test sample selection.

xii. Upon retrieval, the core runs shall be provided to the Geotechnical Engineer for logging, uniformity inspection, and test specimen selection.

xiii. Following logging and test specimen selection by the Geotechnical Engineer, the entire full-depth core, including the designated test specimens, shall be immediately sealed in plastic wrap to prevent drying and transported to the laboratory by the DMM Contractor. Alternatively, the DMM Contractor may transport only the selected test specimens to the laboratory and store the remaining core on-site in a humidity and temperature-controlled storage facility as described in this Specification.

xiv. All core holes shall be filled with cement grout that will obtain a 28-day strength equal to or greater than the strength of the DMM. However, the Contractor shall not grout the core holes until after acceptable core recover and uniformity has been confirmed by the Geotechnical Engineer.

xv. The DMM Contractor shall notify the Geotechnical Engineer at least one business day (24 hours) in advance of beginning core sampling operations.

b. In addition to coring, the DMM Contractor should obtain wet grab samples from the DMM elements at the presence of Geotechnical engineer.

i. 3 (three) wet samples from each mixed design used in each test section as directed by the geotechnical engineer.

ii. One wet sample (i.e., one selected depth at one location) should be retrieved every 2 (two) production days or for every 2,500 cubic yards of treated soil, whichever produces the higher sampling frequency.

iii. The contractor proposes locations for wet sampling as outlined in the QC program, considering input from the geotechnical engineer based on subsurface conditions, DMM layout, review of the QC results, and observation of the soil mixing operation.

iv. The contractor should report all attempts, successful and unsuccessful, to obtain wet samples. Some deep mixed material may not be able to be sampled readily because either the mixture is too stiff or the material may not flow back into the void left after the sampler is extracted, possibly leaving a damaged element.

v. The sampling tool is inserted into the DMM column to a designated depth, filled with treated soil, and lifted to the ground surface. The treated soil material is then poured into a container, screened for oversized lumps (gravel versus unmixed soil), and placed in 3-inch (76-mm)- diameter, 6-inch (152 mm)- long molds. Eight test specimens should be prepared from each wet sample.

vi. The wet treated material should be placed into the mold in three to five layers. After the placement of each layer, the specimens must be tapped or vibrated to remove trapped air bubbles. The specimens should be sealed to prevent moisture from entering or leaving the specimens, and the sealed specimens should be stored in a humid environment in accordance ASTM C192.

vii. For field validation testing, unconfined compressive strength testing may be performed on specimens at 3, 7, 28, and 56 or more days. For full production work, unconfined compressive strength testing may be performed at 3, 7 and 28 days.

viii. The DMM contractor should deliver the samples for testing to a local lab as directed by the geotechnical engineer.

ix. If wet samples produce results that are consistently acceptable, the frequency of wet sampling can be reduced as the project progresses.

x. The engineer may request additional test specimens for QA testing.

c. Untested portions of the full-depth samples shall be retained at the laboratory until completion and acceptance of all DMM, for possible inspection and confirmation testing by the Geotechnical Engineer.

7. The DMM Contractor shall be responsible for handling of test specimens, including storing of untested specimens and transporting test specimens to the Testing Laboratory.
   a. The laboratory testing shall be performed by the DSA accepted Testing Laboratory hired by the Owner and approved by the Geotechnical Engineer.
   b. The samples shall be stored in a moist room as specified in ASTM C 192 until the test date.
   c. Testing for 28-day unconfined compressive strength shall be conducted in accordance with ASTM D2166.
8. In addition to confirmation tests performed by the Testing Laboratory, additional tests may be requested by the Geotechnical Engineer on samples collected by the DMM Contractor. Both the Testing Laboratory's testing and the Geotechnical Engineer's requested additional testing (if performed) shall demonstrate that the acceptance criteria are met prior to acceptance of the work.
9. Daily Quality Control Report
   a. The DMM Contractor shall submit Daily Quality Control Reports to the Geotechnical Engineer at the end of the next working day. The Daily Quality Control Report shall

document the progress of the DMM construction, present the results of the QC parameter monitoring, present the results of the strength testing, and clearly indicate if the columns have met the acceptance criteria. The DMM Contractor shall make all Daily Quality Control Reports available to the Geotechnical Engineer

b. The Daily Quality Control Report shall include as a minimum the results of the following QC parameter monitoring for each column:
   i. Rig number,
   ii. Type of mixing tool,
   iii. Date and time (start and finish) of column construction,
   iv. Column number and reference drawing number,
   v. Column diameter,
   vi. Column top and bottom elevations,
   vii. Grout mix design designation,
   viii. Slurry specific gravity measurements (refer to Section A3.9 for number of tests and tolerance), and
   ix. Description of obstructions, interruptions, or other difficulties during installation and how they were resolved.

c. The Daily Quality Control Reports shall also include the following parameters recorded automatically for each column continuously and submitted in the form of either tables or figures (as agreed to by the Geotechnical Engineer):
   i. Elevation in feet vs. real time,
   ii. Shaft rotation speed in RPMs vs. depth,
   iii. Penetration and withdrawal rates in feet per minute vs. depth,
   iv. Grout injection rate in gpm vs. depth, and
   v. The average quantity of grout in gallons per foot injected per 3-foot (or less) vertical increment of column vs. depth.

J.3.15 Acceptance Criteria

1. The Geotechnical Engineer and CGS shall make the sole determination as to whether the acceptance criteria have been satisfied. The in-place grout-soil mixture comprising the DMM Elements shall meet the following acceptance criteria:
   a. The DMM within the Treatment Zone shall be installed within the following geometric tolerances:
      i. The horizontal alignment of the DMM blocks shall be within 4 inches of the location shown on the approved DMM Layout Plans.
      ii. The vertical inclination of the DMM columns shall be no more than 1: 100 (horizontal to vertical).

iii. Overlap between any two adjacent columns shall be a minimum of 30 percent of column diameter for the entire depth, as calculated based on depth of column embedment and measured auger lateral and longitudinal inclination.
iv. The tops of the columns shall be at or higher than the elevations indicated on the DMM Design Plans.
v. The bottoms of the columns shall extend to or lower than the levels indicated on the DMM Design Plans.

b. Two alternative specified unconfined compressive strengths are provided with corresponding minimum A_r and maximum grid spacing in **Table A.1**. The DMM Contractor shall select one of these options for the entire project.
c. The unconfined compressive strength shall be determined by ASTM D2166 at 28 days on samples taken by coring of the constructed DMM.
d. 80 percent of all unconfined compressive strength testing on core samples determined by ASTM D2166 from each tested deep mixed element shall equal or exceeds the specified strength. If a strength specimen falls below the specified strengths due to an obviously unrepresentative lump of unmixed soil in the specimen, the Geotechnical Engineer has the option to select another specimen from the same core run and allow the Testing Laboratory to test the replacement specimen and substitute the strength from the replacement specimen for the strength from the unrepresentative specimen that failed to satisfy the strength requirement. Only one such retest will be allowed per core run.
e. 90 percent of all the test results on core samples across the site should equal or exceed the specified strength.
f. To prevent a weak layer at one elevation in the DMM foundation system, strengths below the specified strength are not permitted within 10 feet of the same elevation in more than 2 nearby cored elements.
g. Uniformity of mixing within the target zone shall be evaluated by the Geotechnical Engineer based on the full-depth samples recovered by the DMM Contractor from the columns.
   i. Lumps of unimproved soils shall not amount to more than 15 percent of the total volume of any core run from a continuous full-depth core sample. For evaluating the volume of unimproved lumps of soil, all of the unrecovered core length shall be assumed to be unimproved soil.
   ii. Any individual or aggregation of lumps of unimproved soil shall not be larger than 12 inches in greatest dimension.
   iii. Continuous core recovery shall be at least 85 percent over any full-length core.

2. If the acceptance criteria specified in this Specification are not achieved for production DMM, the failed section of DMM shall be rejected.

a. Unless otherwise determined by the Geotechnical Engineer, the failed section of DMM shall be considered to include all DMM columns constructed during all rig shifts that occurred between the times of construction when passing tests were achieved.
b. The DMM Contractor may conduct additional sampling and testing to better define the limits of the failed area at no additional cost to the Owner.
c. The DMM Contractor shall submit a proposed plan for remixing or repair of failed sections for review and approval by the Geotechnical Engineer and CGS.
d. If the treated soil that failed to meet the uniformity criteria is concentrated in a narrow elevation range forming weak planes or zones, the contractor could propose redrilling and remixing to 3 feet below and above the deficient zone. IF redrilling and remixing cannot be done efficiently, the contractor must replace the elements to the full depth. If the treated zone in the narrow elevation meets the uniformity criteria but fails to meet the strength criteria, the contractor could propose to redrill and remix the deficient zone or to assign a lower strength level to the deficient zone and install additional elements to compensate for the strength deficiency.
e. If the treated soil that failed to pass cannot be isolated in a specific zone, the contractor must provide remedial measures for all elements constructed during all rig shifts that occurred between passing elements.
f. Remedial measures are subject to coring and application of the specification acceptance criteria.