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

Oakland, California

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Peralta Community College District

Document Control

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February 28, 2020

Dear Ms. Smith,

Fugro is pleased to submit this 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 and the Amendment No. 1 (Requisition No. 2-135365, PO No. 3-122826) dated December 11, 2019, and was executed in general accordance with the scopes listed in our Proposals No. 04.72189129-P-001(Rev.02) and No. 04.72190021-P-001(Rev.00), respectively, dated February 19, 2019 and July 19, 2019.

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*, dated October 2013.

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- or 4-story high building with an estimated footprint area of about 21,000 square feet. The proposed building location is about 130 to 160 feet away from the edge of the Lake Merritt Channel west bank. The actual location and layout of the new building are yet to be finalized. No significant raising of the existing site grade is anticipated for the project. According to the preliminary information, the new building columns will most likely support loads varying from about 700 to 2,000 kips at the foundation level. Deep foundations are anticipated to be used for the proposed new building.

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).
- 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. Detailed flooding and dam inundation risk evaluation of the site was also 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.
- State of California, *Earthquake Fault Zones, Oakland West Quadrangle*, Revised Official Map, Released: January 1, 1982.
- State of California, *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 & Hoose, 1978. *Historical Ground Failures in Northern California Triggered by Earthquakes,* USGS Professional Paper 993.
- State of California, 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). 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. The passage of time could result in changes in the subsurface conditions due to either natural processes or human activities.

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 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 **Appendix 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 **Appendix 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 **Appendix 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 **Appendix B**. Some of the test results are also presented on the boring logs (**Appendix 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 **Appendix B**.

3. Geologic Setting

3.1 Regional Geology

The project site is located on the east of the San Francisco Bay in the Coast Ranges geomorphic province. Northwest-southeast-trending valleys and ridges characterize this province. These are controlled by folds and faults that resulted from the collision of the Pacific and North American Plates, Pacific Plate subduction beneath the North American Plate, and subsequent strike-slip faulting along the San Andreas fault system. The strike-slip motion along this plate boundary replaced subduction several million years ago (Atwater, 1970)[1]. The subduction-style faulting and deformation responsible for intercalating the various older rock types in this area are no longer active in the region.

The tectonic regime in the Bay Area changed from convergent to transform movement about 10 million years ago, resulting in movement of about 32 mm/year being distributed among various faults of the San Andreas fault system, including the Hayward fault (Steinbrugge, 1987)[2]. The Hayward fault provides an approximate boundary between two distinctly different geologic and physiographic provinces. Based on work by Radbruch (1969)[3], basement rocks underlying the area west of the Hayward fault are primarily those of the Jurassic Franciscan Complex (50 to 200 million years old). East of the Hayward fault, the basement rocks are of the Knoxville Formation of similar age.

The bedrock exposed in the hills east of the Hayward fault may be 10 million years old, but the surficial deposits across the flatlands to the west are probably less than a few hundred thousand to a few hundred years old. The San Francisco Bay Area has experienced several episodes of uplift and faulting during late Tertiary time (about 25 to 2 million years ago). During the last major glaciation, more than 15,000 years ago, sea level was about 330 feet lower than it is today. 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. Sediments that were formerly carried far into the Pacific Ocean were then deposited in and around the margins of the Bay. 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. Uplifted areas were eroded, and as a result, Pleistocene and recent marine sediments were deposited in San Francisco Bay, and sediments were deposited in

[1] Atwater, 1970. *Implications of Plate Tectonics for the Cenozoic Evolution of Western North America*, GSA Bulletin 81, 3513-3536.

[2] Steinbrugge and Others, 1987. *Earthquake Planning Scenario for a Magnitude 7.5 Earthquake on the Hayward Fault in the San Francisco Bay Area*, CDMG Special Publication 78.

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

low-lying stream and marshland areas adjacent to the Bay. These sediments provide the relatively level building sites for most of the development in the East Bay.

In the area of the site, fairly 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. Combined thickness of the sediments is estimated to be on the order of 500 feet thick in the area.

Structurally, the project site is in an area dominated by the active San Andreas Fault system that includes the San Andreas, Hayward-Rodgers Creek, and Calaveras faults, as well as many lesser structures. The San Andreas Fault system also is the boundary between the Pacific and North American global tectonic plates. 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, parts 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.

3.2 Site Geology

According to Witter et al. (2006)[4], and as shown on the Regional Geologic Map (**Plate 4**), the site is located within an area southeast of the historical shoreline where tidal flats adjacent to the Lake Merritt Channel have been filled. The site is roughly located in the middle of the estimated approximately 500 to 1,400 feet wide natural outlet channel of Lake Merritt, which had been dramatically reduced in width with developments of the region after 1860s. Filling of this area occurred between 1894 and 1915 based on the study by Rogers and Figuers (1991)[5].

The site is underlain by a layer of historical artificial fill over Holocene estuarine mud (afem) that is known locally as Young Bay Mud. According to Helley and Graymer (1997)[6], the fills made before 1965 in San Francisco Bay Area are nearly everywhere not compacted and consist simply of dumped materials. Based on the results of subsurface 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 marine deposit, predominantly gray, green and blue clay and silty clay underlying marshlands and tidal mud flats of San Francisco Bay and Carquinez

[4] 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 2006-1037.

[5] 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.

[6] 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.

Strait. The mud generally contains a few lenses of well-sorted, fine sand and silt, a few shelly layers, and peat. The 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 deposits at the distal edge of Holocene fans.

3.3 Regional 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 shown on Regional Fault and Seismicity Map (**Plate 5**), several major fault zones extend through the San Francisco Bay Area in a northwesterly direction, and these faults have produced several Magnitude 6.0 or greater earthquakes in the last two centuries within about 60 miles (100 km) of the site. These events were strong enough to cause structural damage. The faults causing such earthquakes are part of a system of right lateral faults along the Pacific and North American plate boundary that generally trends in a north-westerly direction and extends for at least 450 miles along the coast of California, and locally includes the San Andreas, Calaveras and Hayward faults.

As shown on **Plate 5**, the site is located about 3-1/2 miles southwest of the Hayward fault zone. According to the Alquist-Priolo (AP) Earthquake Fault Zone Map of the Oakland West Quadrangle (**Plate 6**), the site is not located within an earthquake fault zone, as designated by the State of California (CGS, 1982)[7].

The Hayward fault typically exhibits geomorphic evidence of displacement such as shutter ridges, reversed drainage, aligned topographic sags and scarps. The Hayward fault zone, while locally very wide, typically includes relatively large masses of intact rock that separate narrow, individual fault traces along which recurrent movement has occurred. Creep rates along the Hayward fault vary considerably from place to place, with an average reported slip rate of about 9 mm/year (WGCEP 2003).

The approximate distances from the site to the closest known mapped active and potentially active faults within about 60 miles (100 km) and the estimated fault seismicity by USGS are summarized in **Table 3.1.**

[7] State of California, 1982. *Earthquake Fault Zones, Oakland West Quadrangle*, Revised Official Map, Released: January 1, 1982.

Table 3.1: Regional Faults and Seismicity[8]

Fault	Approximate Closest Distance from Site to Fault (miles)	Direction from Site to Fault	Estimated Maximum Earthquake 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

Earthquakes on these or smaller, more distant, or unmapped faults could cause strong ground shaking at the site. USGS Fact Sheet (2016)[9] 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 sites from the causative fault, the type of materials underlying the sites, and other factors.

According to 2019 CBC/ASCE 7-16 and based on an average soft clay soil site condition (a Site Class E), the site geometric mean peak ground acceleration (PGA_M) from Maximum Considered Earthquake (MCE) is estimated to be about 0.805g. 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.

[8] Obtained from *USGS, 2008. National Seismic Hazard Maps – Source Parameters* website, https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query_main.cfm.

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

3.4 Historical Seismicity

Major earthquakes have been recorded along the San Andreas Fault system since the mid-1500s. **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)[10] and National Atlas of United States[11] database was accessed to obtain the historical seismicity information presented in **Table 3.2**. Historic epicenter locations are also shown on the Regional Fault and Seismicity Map (**Plate 5**).

Table 3.2: Large Magnitude (M≥6.0) Regional 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	7.7	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

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 on a splay of the San Andreas fault. Peak ground accelerations of about 0.24 to 0.13g had been reported in the region (USGS ShakeMap Record Stations DeweyOBS_68 and DeweyOBS_175).

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

[11] National Atlas of United States, *Significant United States Earthquakes*, 1568-2009, USGS dataset.

4. Site Conditions

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 located 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 **Appendix A**. Our laboratory testing results of the onsite soil samples are included in **Appendix B**. Logs of historic explorations and results of lab testing are included in **Appendix 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 (2003)[12], 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.

[12] California Geological Survey, 2003, Seismic Hazard Zone Report for the Oakland West 7.5-Minute Quadrangle, Alameda County, California.

5. Geologic Hazards Evaluation

Our site geologic hazard evaluations were performed in accordance with guidance from the CGS Note 48. 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.

5.1 Surface Fault Offset

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 considered to be very low.

5.2 Shaking Hazards

Strong ground shaking at the project site should be anticipated during a moderate to severe earthquake occurring on one of the active Bay Area faults. Strong ground shaking can cause the structures to shake; and has the potential capability of inducing other phenomena that can indirectly cause 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 these phenomena with respect to the site are presented in the subsequent paragraphs.

5.3 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)[13] and Association of Bay Area Governments (ABAG) Resilience Program Liquefaction Susceptibility Map[14], the site (as shown on **Plate 12**) is located in an area that has been characterized as having a very high liquefaction susceptibility. The CGS Seismic Hazard Zones Map of the Oakland West Quadrangle (2003)[15] 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.

5.3.1 Historical Liquefaction in Site Region

According to CGS Seismic Hazard Zone Report of Oakland West Quadrangle (2003)[16], several areas of historical liquefaction events had been documented in the past. Youd and Hoose (1978)[17] 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 are also summarized for the 1989 Loma Prieta earthquake by Tinsley et al. (1998)[18] throughout the San Francisco Bay Area. 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.

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[13] 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 2006-1037.

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

[15] State of California, *Seismic Hazard Zones, Oakland West Quadrangle*, Official Map, Released: February 14, 2003.

[16] California Geological Survey, 2003. *Seismic Hazard Zone Report for the Oakland West 7.5-Minute Quadrangle, Alameda County, California*, Seismic Hazard Zone Report 081.

[17] Youd and Hoose, 1978. *Historical Ground Failures in Northern California Triggered by Earthquakes*, USGS Professional Paper 993. Page 120

[18] Tinsley, Egan, Kayen, Bennett, Kropp, and Holzer, 1998. *Appendix: Maps and Descriptions of Liquefaction and Associated Effects, The Loma Prieta, California, Earthquake of October 17, 1989 - Liquefaction*, USGS Professional Paper 1551-B.

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)[19], which was probably resulted from soil liquefaction. A replacement swimming pool was constructed in mid-1990s.

5.3.2 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)[20] and the recommended procedures by Southern California Earthquake Center (SCEC, 1999)[21].

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 7.0 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)[22] and by Boulanger and Idriss (BI, 2014)[23] were used for CPT-based analyses. Post-liquefaction deformations were calculated for all layers by using Ishihara and Yoshimine procedures (1992)[24] for NCEER method and EERI Monograph 12 procedures (Idriss & Boulanger, 2008) for BI 2014 Method.

The SPT-based analyses generally followed the methodology described in the EERI Monograph 12 (MNO-12, Idriss and Boulanger, 2008)[25]. Per CGS Note 48 requirements, post-liquefaction

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

[20] California Geological Survey, 2008. *Guidelines for Evaluating and Mitigating Seismic Hazards in California*, CGS Special Publication 117A.

[21] Martin and Lew, March 1999. *Recommend Procedures for Implementation for DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California*, Southern California Earthquake Center.

[22] Youd and Idriss, 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.

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

[24] Ishihara and Yoshimine, 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.

[25] Idriss and Boulanger, 2008. *Soil Liquefaction During Earthquakes*, EERI Monograph MNO-12.

deformations were calculated for soil layers that have a factor of safety against liquefaction less than 1.3.

5.3.3 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 by 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 **Appendix 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 the great amount of 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 (inch)		
		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.

5.3.4 Dynamic Densification Evaluations

We performed both CPT-based and SPT-based dynamic densification evaluations based on procedures developed by Tokimatsu and Seed (1987)[26] and Robertson and Shao (2010)[27]. 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 **Appendix E**. It is our opinion that the potential for soil dynamic densification to impact the site is low.

5.4 Lateral Spreading

Lateral spreading occurs when soils liquefy during an earthquake event and the liquefied soils with the overlying soils move laterally toward unconfined spaces (Lake Merritt Channel west bank), which causes 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)[28] at each CPT and boring location. The detailed results are included in **Appendix D**.

In addition, empirical correlations developed by Youd et al. (2002)[29] 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., 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

[26] Tokimatsu and Seed, 1987. *Evaluation of Settlements in Sands Due to Earthquake Shaking*, Journal of Geotechnical Engineering, Vol. 113, Issue 8, August.

[27] Robertson and Shao, 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.

[28] Zhang, Robertson, and Brachman, August 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.

[29] Youd, Hansen, and Bartlett, December 2002. *Revised Multilinear Regression Equations for Predication of Lateral Spread*, Journal of Geotechnical and Geoenvironmental Engineering, p. 1007-1017.

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.

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

Location	Liquefiable Soil Elevation (ft)	Calculated Lateral Displacement Index - LDI (inch)	Potential Lateral Spreading Triggering Soil Elevation (ft)	Estimated Ground Lateral Displacement (inch)
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 in 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.**

5.5 Landsliding

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)[30] 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)

It should be noted that the project proposed building location is located about 130 to 160 feet away from the edge of the west bank. 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 ever 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.

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.

5.5.1 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.

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

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)	-

5.5.2 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 **Appendix 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 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 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 **Appendix 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)[31]. 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.**

5.6 Seismically Induced Waves

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 affected by them. Tsunamis are large waves in the ocean generated by earthquakes, coastal or submarine landslides, or volcanoes. Damaging tsunamis are not common on the California coast. Most California tsunamis are associated with distant earthquakes (most likely those in Alaska or Pacific Ocean), not with local

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

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 at Elevations of about +18 feet to +21 feet (NAVD 88) and the site is located about 1/4 mile from the Oakland Inner Harbor that is bounded by the Alameda island and the Oakland bay shore. According to the CGS Tsunami Inundation Map for Emergency Planning of the Oakland West Quadrangle (2009)[32], 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 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 fault displacement in the basin bottom, large landslides into the basin, or from periodic oscillation or sloshing of the water in the basin. According to City of Oakland General Plan Safety Element (2004)[33], 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.

5.7 Flooding and Dam Inundation

According to FEMA Flood Insurance Rate Map (2009)[34], the project building area is located outside a 100-year flood zone.

According to the City, 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 these above facilities. Based on this information, the potential for flooding or inundation of the project site is considered to be very low.

[32] State of California, July 31, 2009. *Tsunami Inundation Map for Emergency Planning, Oakland West Quadrangle.*

[33] City of Oakland Community and Economic Development Agency, November 2004. *City of Oakland General Plan*, *Safety Element.*

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

5.8 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.

5.9 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 **Appendix 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.

5.10 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 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.

5.11 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.

5.12 Naturally 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 rock quarrying activities where ultramafic rock is present.

The bedrock underlying the site is estimated to be on the order of 500 feet in depth. In addition, no serpentinite gravels were reportedly encountered in the previous borings at the site. Therefore, we consider the possibility of NOA at the site to be very low.

5.13 Volcanic Eruption

Potential hazards associated with volcanic activity in the site region are estimated to be very low (Miller, 1989)[35].

[35] Miller, 1989. *Potential hazards from future volcanic eruptions in California*, USGS Bulletin 1847, 17 p.

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 Seismicity 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, compressible 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 appeared to be low to negligible, in our opinion.

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 earthquake events.

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 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 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 a 18-feet high of soils. Our recommended lateral pressures for the shoring system designs are shown on **Plates 12** and **13**.

The site and any new improvements not supported on deep foundations may experience total aerial ground surface settlements on the order of about 1 to 4 inches with locally up to about 6-1/2 inches. 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 compensate for the settlement caused by the liquefaction of the underlying supporting soils. 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.

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.

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 pile 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.

In addition, in order 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 in order to prevent potential uplift water pressures from lifting the lightweight fill materials.

6.4 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 **Appendix 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 **Appendix B** are deemed insufficient by the designers of the corrosion protection.

6.5 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 **Appendix 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}, and S_{D1} are tabulated in **Table 7.2**.

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

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. In order 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. In order 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 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.

In order 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

The proposed new building foundation should be designed to provide proper bearing supports during the potential soil liquefaction events. In addition, the southeast side of the proposed new building foundation should also include a permanent shoring system 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 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 a 18-

feet high of soils. Our recommended lateral pressures for the shoring system designs are shown on **Plates 12** and **13**.

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 pile 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.

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 aerial ground surface settlements on the order of about 1 to 4 inches with locally up to about 6-1/2 inches. 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 pile 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 **Appendix 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)		Material Properties: Strain at 50% Stress		Material Properties: Friction Angle Φ′ (degrees)	Material Properties: 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)		Material Properties: Strain at 50% Stress		Material Properties: Friction Angle Φ′ (degrees)	Material Properties: 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 below tables for each soil profile. The calculated pile head deflection, bending moment, and shear force versus embedment depth are presented in **Appendix 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 pile foundations 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 in order 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 damaging the piles. The pre-drilled holes should have a diameter less than the 3/4 the diagonal width of the piles.

7.3.4 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 **Appendix 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 in order 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 retains 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 transmitting and efflorescence forming on the front wall face.

We recommend that any undrained unrestrained walls, which 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 which 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[36,37], 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

[36] Seed and Whitman, 1970, *Design of Earth Retaining Structures for Dynamic Loads*.

[37] Atik and Sitar, 2007, *Development of Improved Procedures for Seismic Design of Buried and Partially Buried Structures*, Pacific Earthquake Engineering Research Center.

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 in order 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. In the event that 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 unconsulted use of segregated portions of this report.

9. References

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List of Plates

PERALTA COMMUNITY COLLEGE DISTRICT
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER
OAKLAND, CALIFORNIA

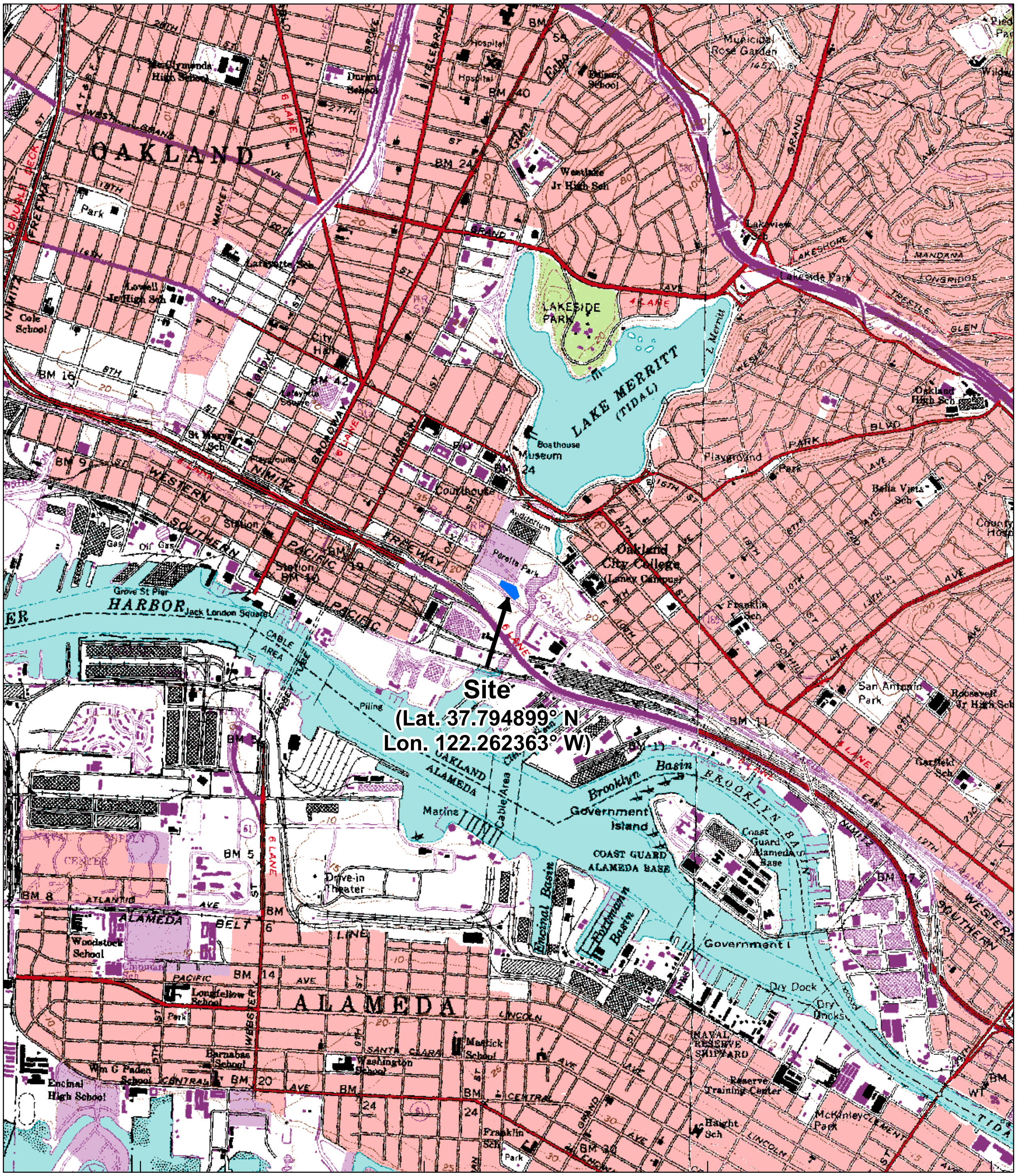
Base map from Esri, 2019.

VICINITY MAP

PLATE 1

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\04_Outputs\2019_05_01_GeotechReport\MXD\1_VicinityMap.mxd; A.Ramirez; 6/13/2019

PERALTA COMMUNITY COLLEGE DISTRICT
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER
OAKLAND, CALIFORNIA



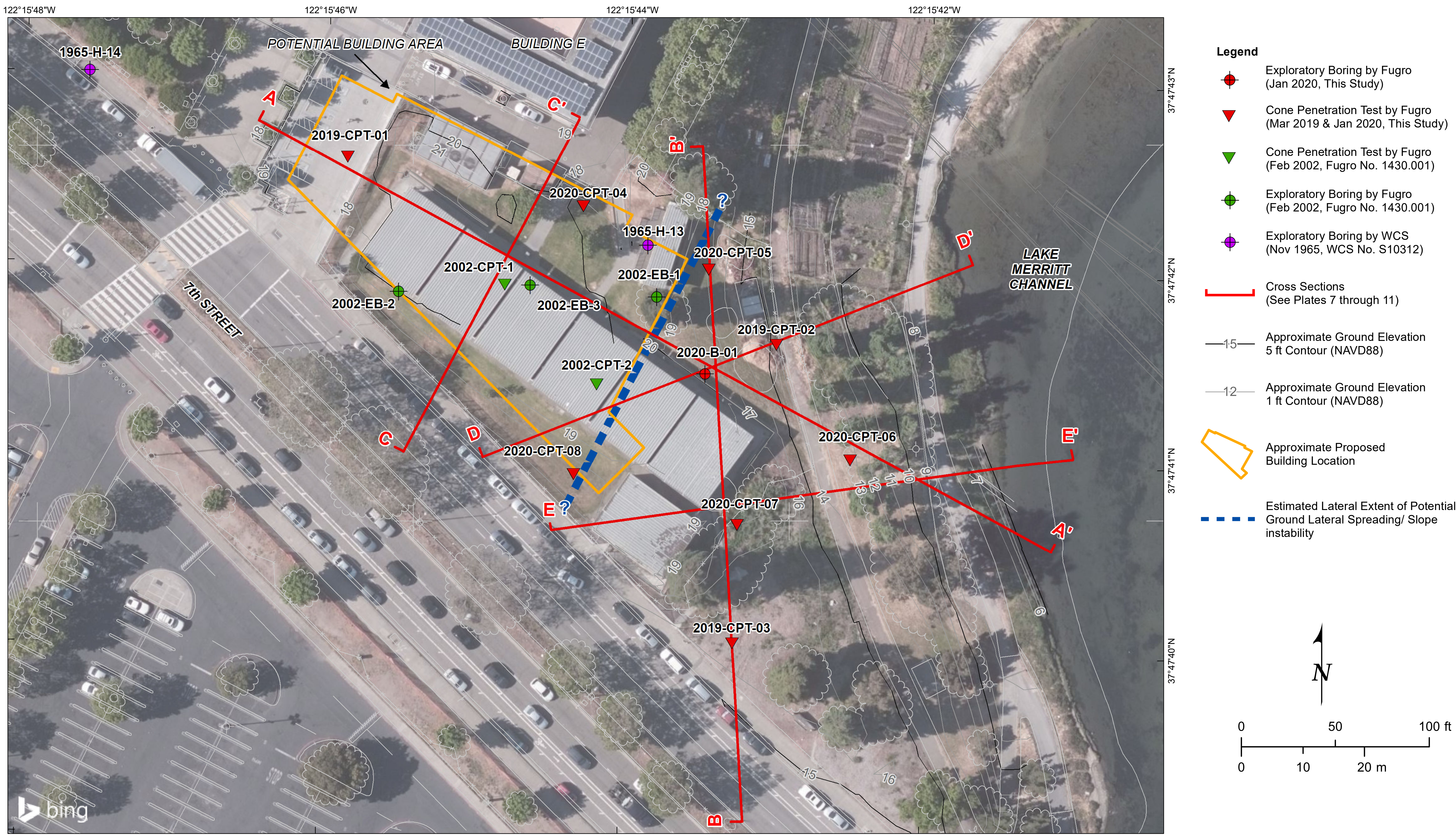
Base map USGS Oakland West and Oakland East 1:24,000-scale topographic quadrangles, 1980.

TOPOGRAPHIC SITE MAP

PLATE 2

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\04_Outputs\2019_05_01_GeotechReport\MXD\2_TopographyMap.mxd; A.Ramirez; 6/14/2019

PERALTA COMMUNITY COLLEGE DISTRICT
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER
OAKLAND, CALIFORNIA



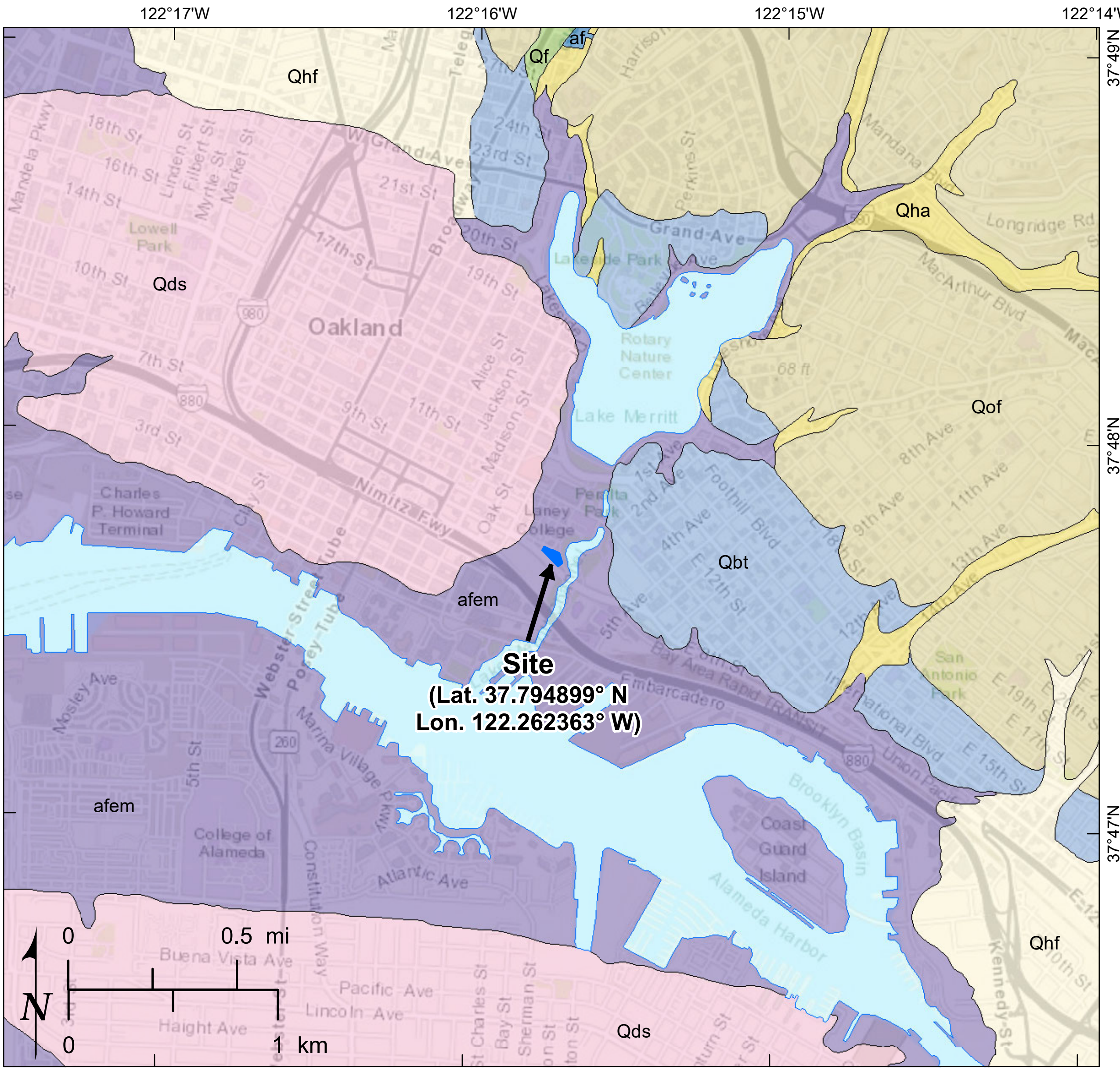
Aerial imagery from Bing Maps. Topo contours provided by CSW/Stuber-Stroeh, April 2019. Proposed building location provided by Noll and Tam Architects, January 2020.

SITE PLAN

PLATE 3

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\04_Outputs\2020_01_09_GeotechReport\MXD\3_SitePlan.mxd, 2/17/2020, m.srisabaranjan

FUGRO



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

Legend

HISTORICAL

af	Artificial fill
afem	Artificial fill over estuarine mud

HOLOCENE

Qhf	Alluvial fan deposits
Qha	Alluvial deposits, undifferentiated

HOLOCENE TO LATEST PLEISTOCENE

Qds	Dune sand
Qf	Alluvial fan deposits

PLEISTOCENE

Qbt	Bay terrace deposits

EARLY TO LATE PLEISTOCENE

Qof	Alluvial fan deposits

REGIONAL GEOLOGIC MAP

PLATE 4

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\04_Outputs\2019_05_01_GeotechReport\MXD\4_RegionalGeologicMap.mxd; A.Ramirez; 6/14/2019

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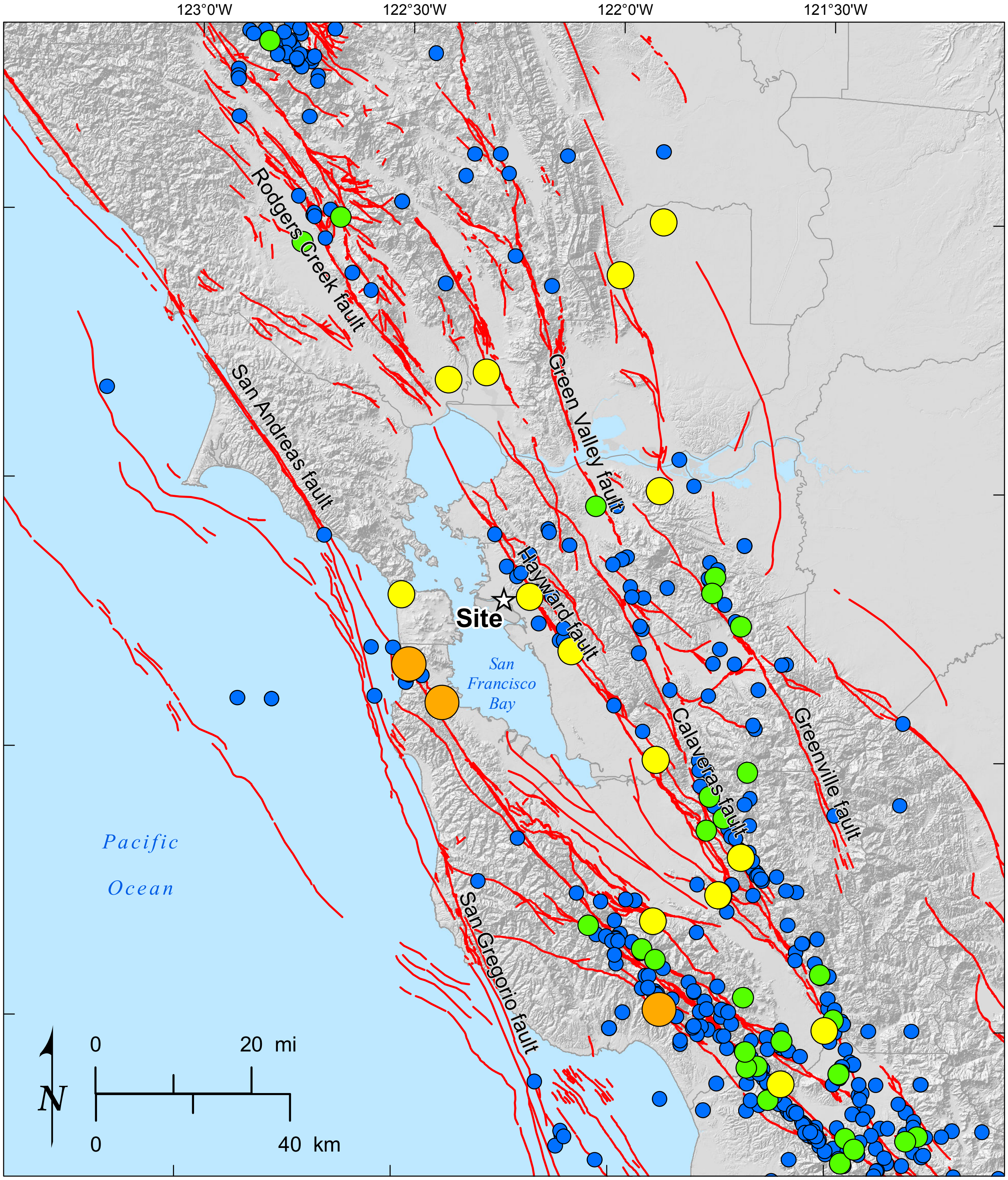

Legend

- 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

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\04_Outputs\2019_05_01_GeotechReport\MXD\5_Regional_FaultMap.mxd; m.ticci; 6/11/2019

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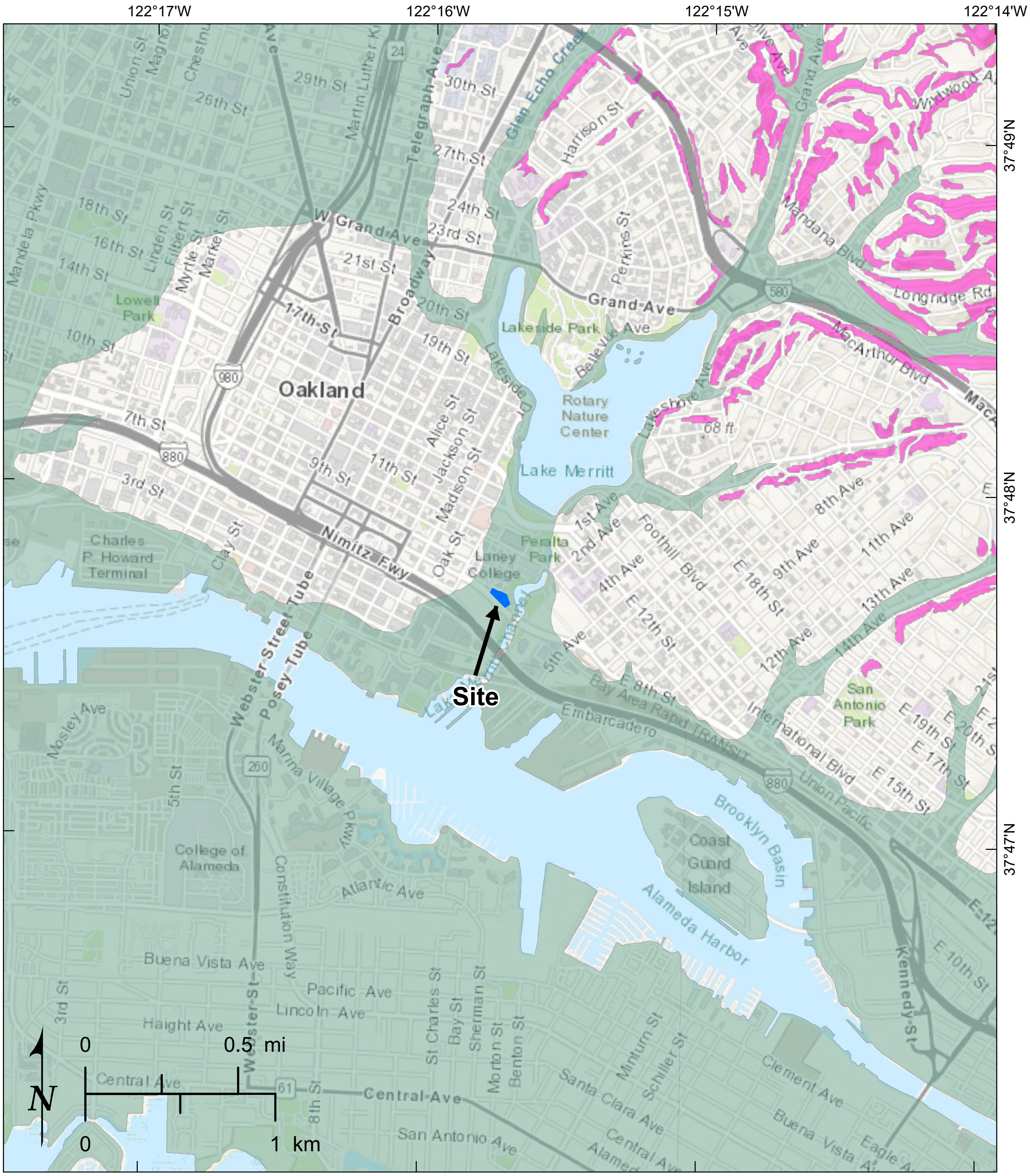

Base map from ESRI, 2019.

Legend

Earthquake-induced landslide zone (CGS, 2003)

Liquefaction zone (CGS, 2003)

CGS SEISMIC HAZARD ZONE MAP

PLATE 6

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\04_Outputs\2019_05_01_GeotechReport\MXD\6_CGS_SeismicHazardZones.mxd; A.Ramirez; 6/21/2019

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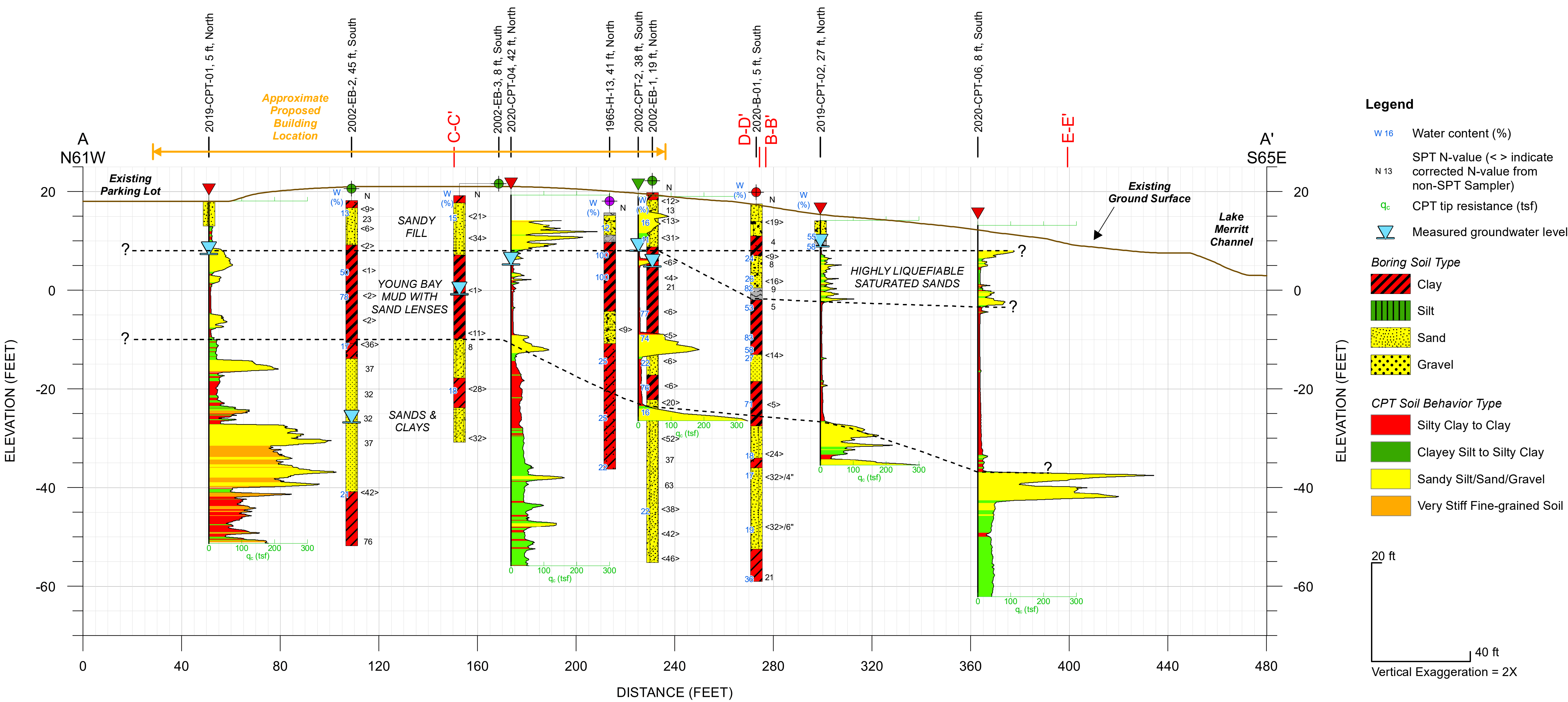

CROSS SECTION A-A'

PLATE 7

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\04_Outputs\2020_01_09_GeotechReport\MXD\7_CrossSection_A.mxd, 2/19/2020, m.srisabaranjan

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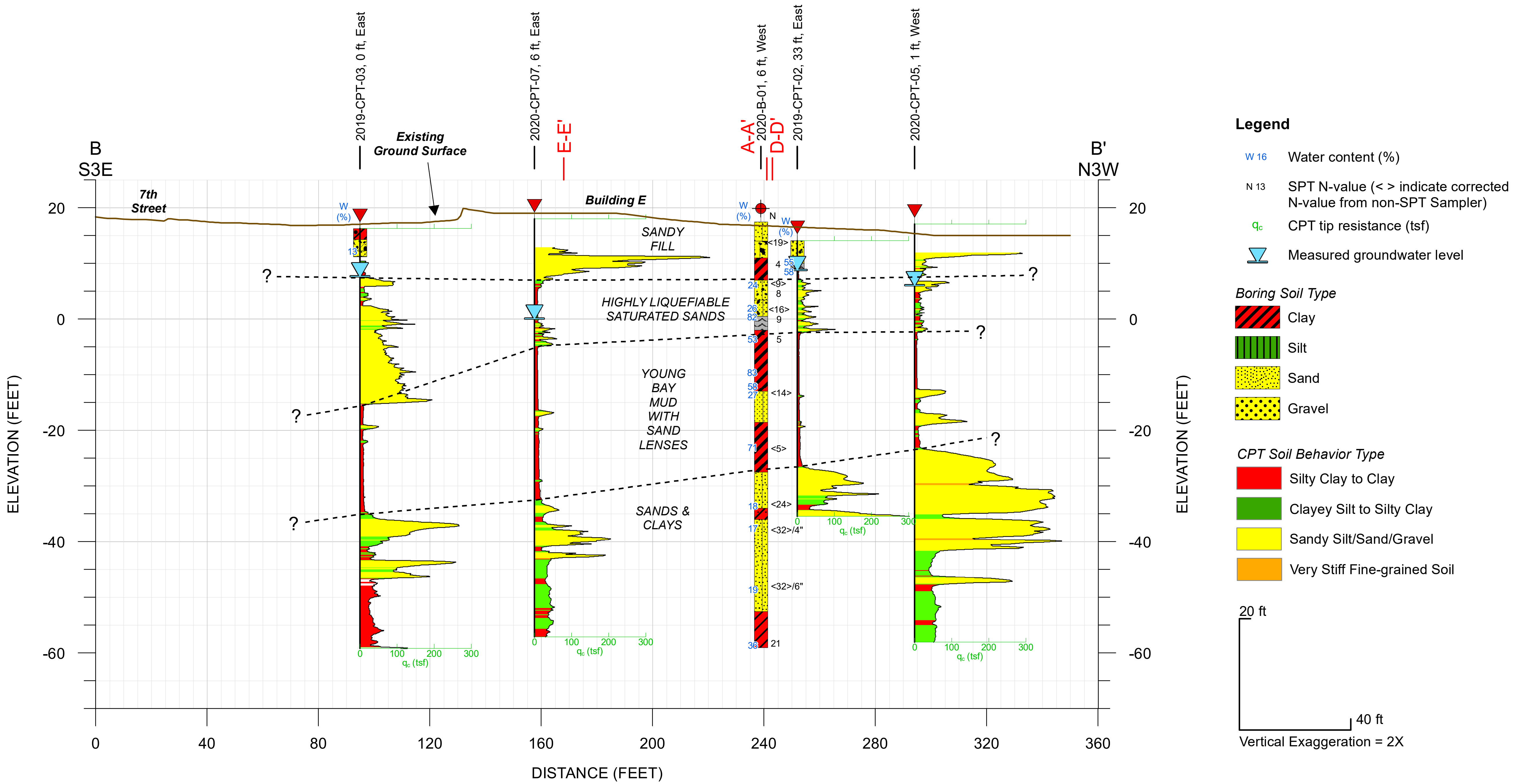

CROSS SECTION B-B'

PLATE 8

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\04_Outputs\2020_01_09_GeotechReport\MXD\8_CrossSection_B.mxd, 2/19/2020, m.srisabaranjan

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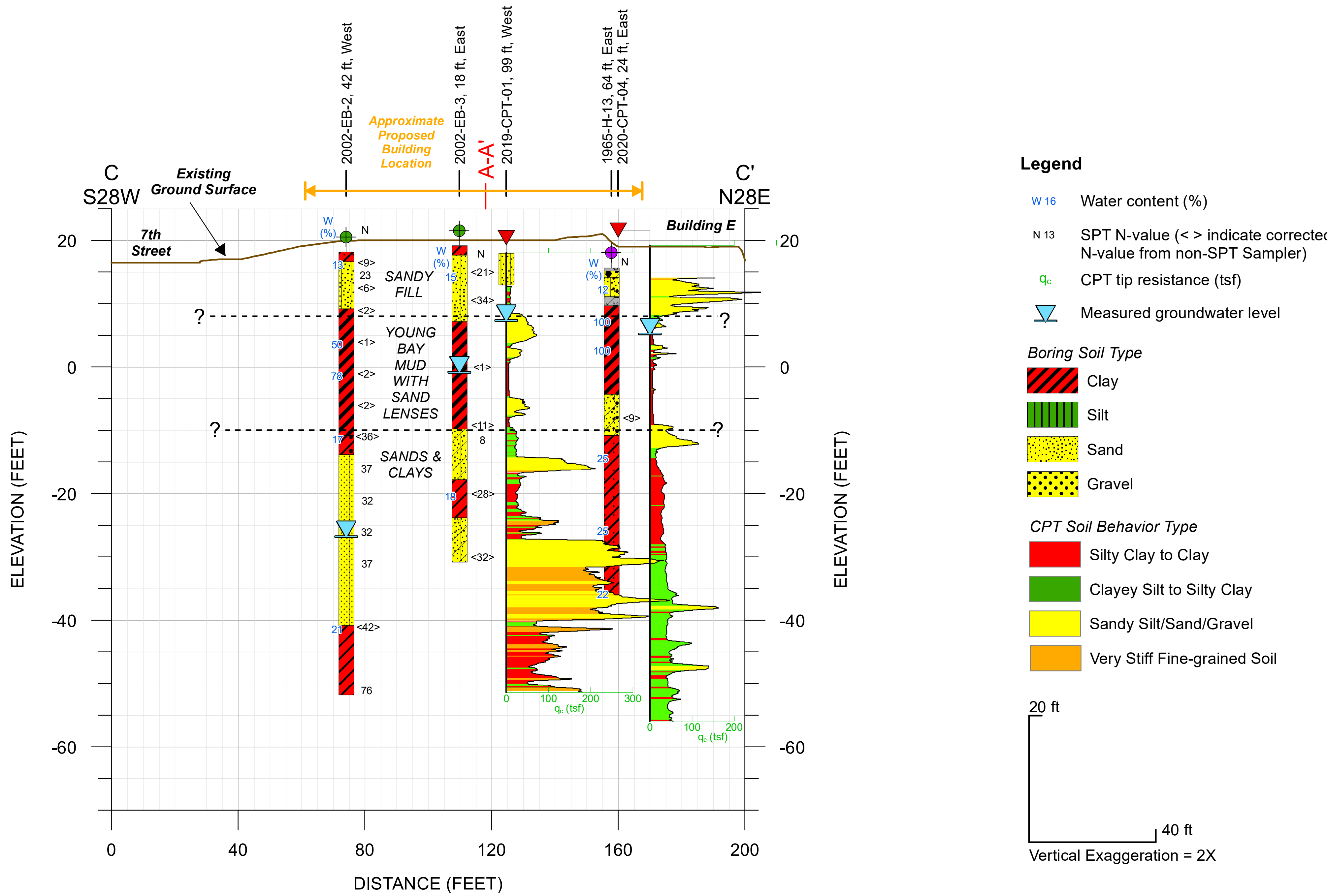

CROSS SECTION C-C'

PLATE 9

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\04_Outputs\2020_01_09_GeotechReport\MXD\9_CrossSection_C.mxd, 2/17/2020, m.srisabaranjan

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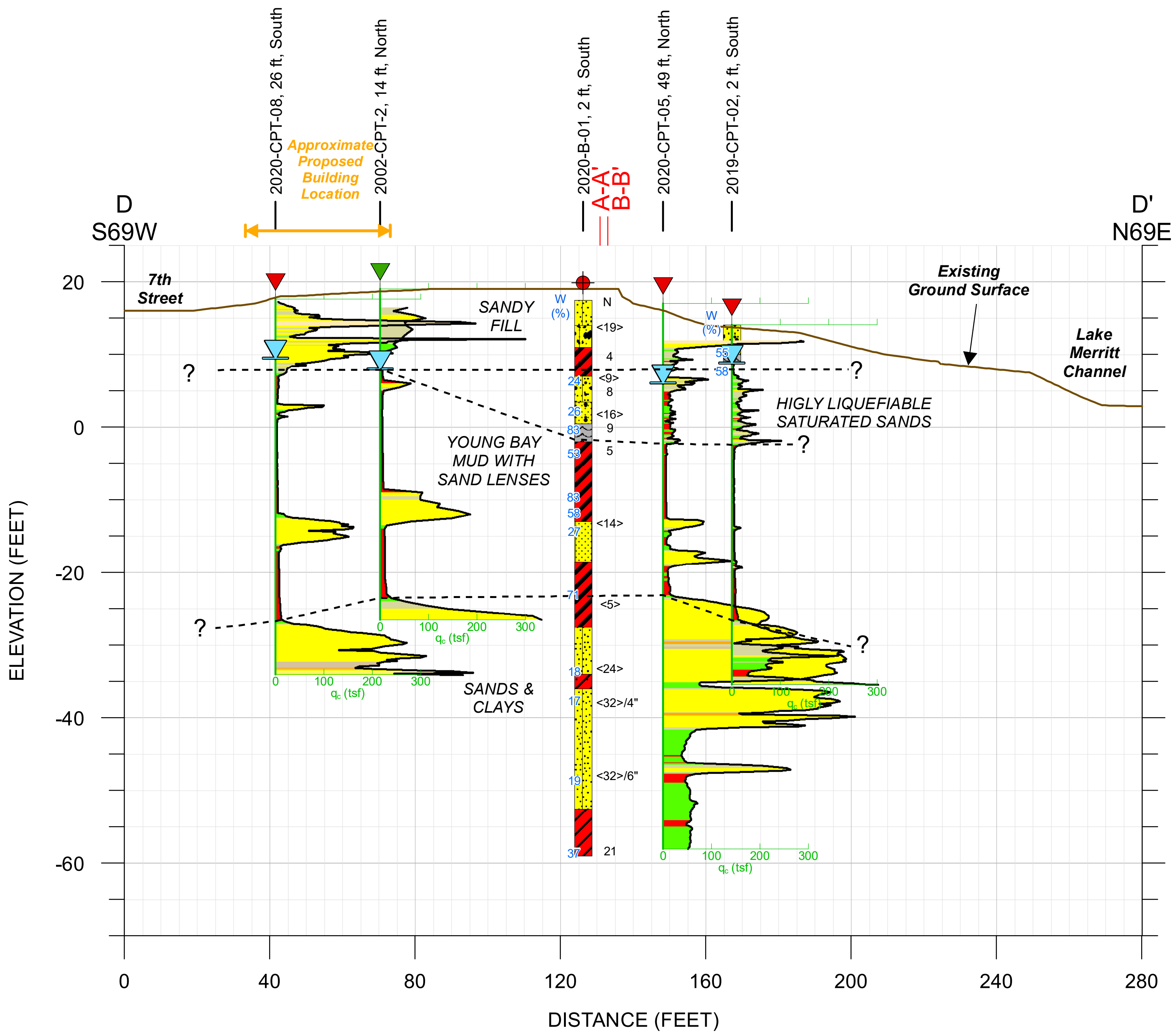


Legend

W 16	Water content (%)
N 13	SPT N-value (< > indicate corrected N-value from non-SPT Sampler)
q_c	CPT tip resistance (tsf)
	Measured groundwater level

Boring Soil Type

- Clay
- Silt
- Sand
- Gravel

CPT Soil Behavior Type

- Silty Clay to Clay
- Clayey Silt to Silty Clay
- Sandy Silt/Sand/Gravel
- Very Stiff Fine-grained Soil

20 ft
40 ft
Vertical Exaggeration = 2X

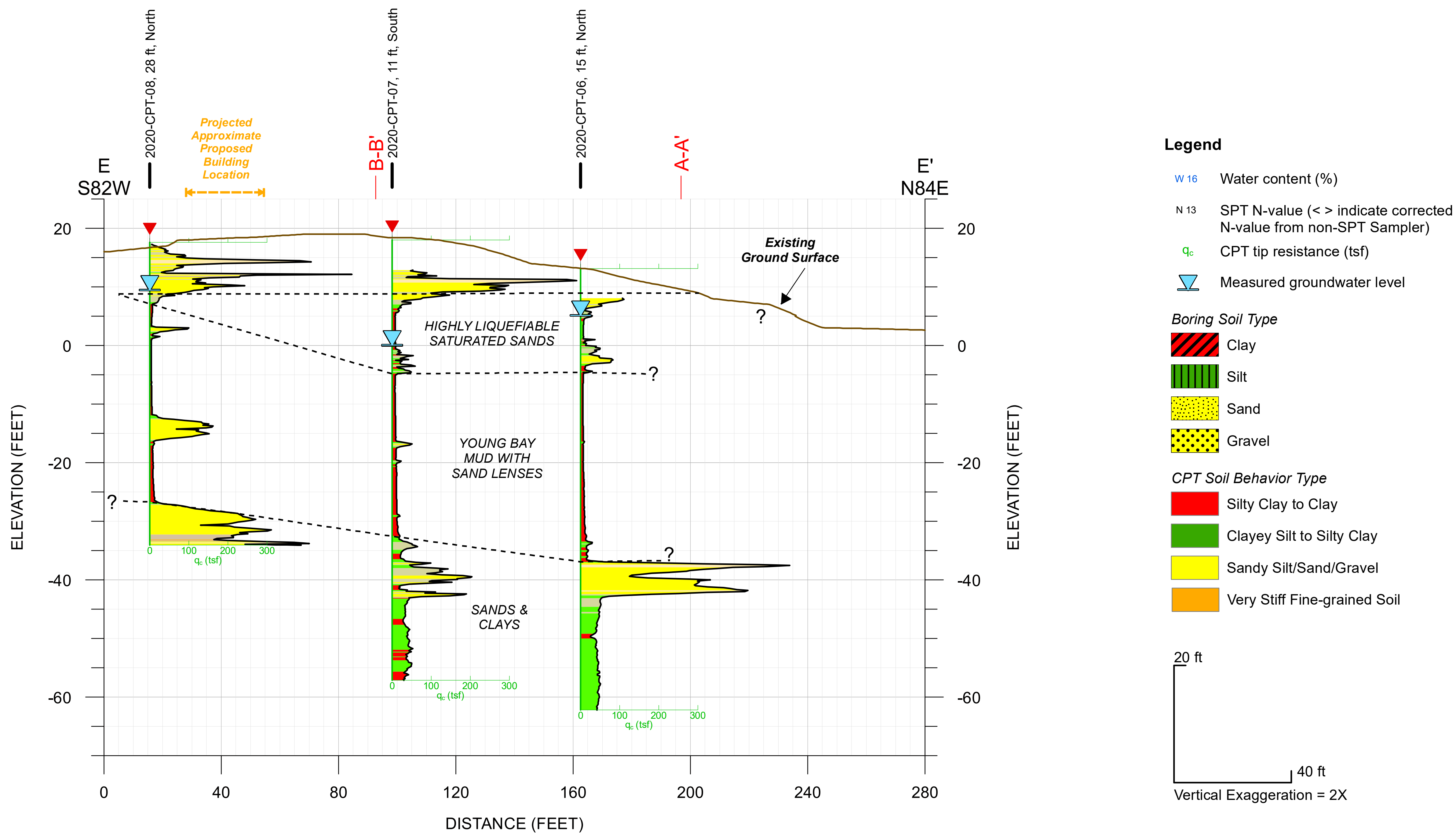
CROSS SECTION D-D'

PLATE 10

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\04_Outputs\2020_01_09_GeotechReport\MXD\10_CrossSection_D.mxd, 2/19/2020, m.srisabaranjan

Fugro Document No. 04.72190021

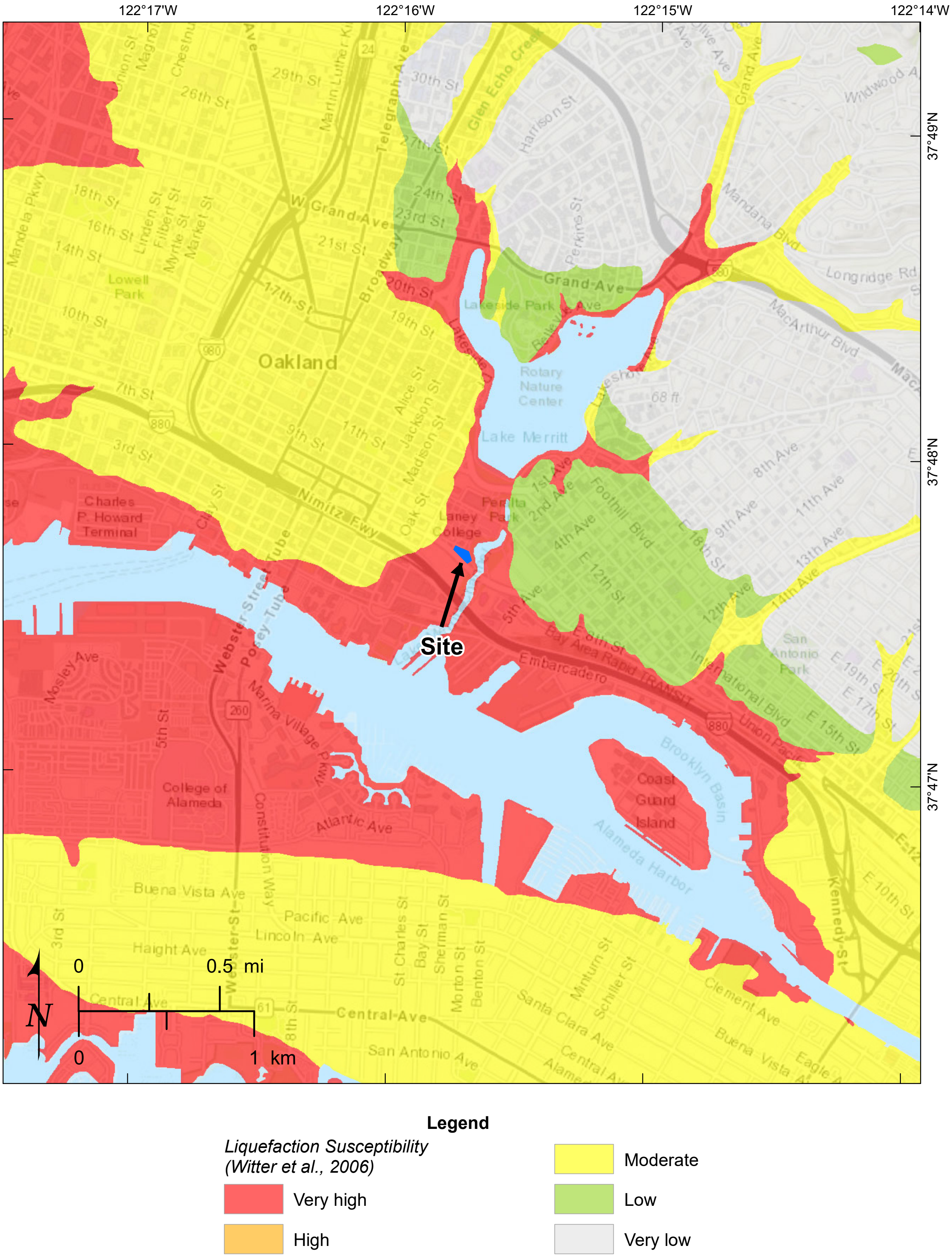
FUGRO



CROSS SECTION E-E'

PLATE 11

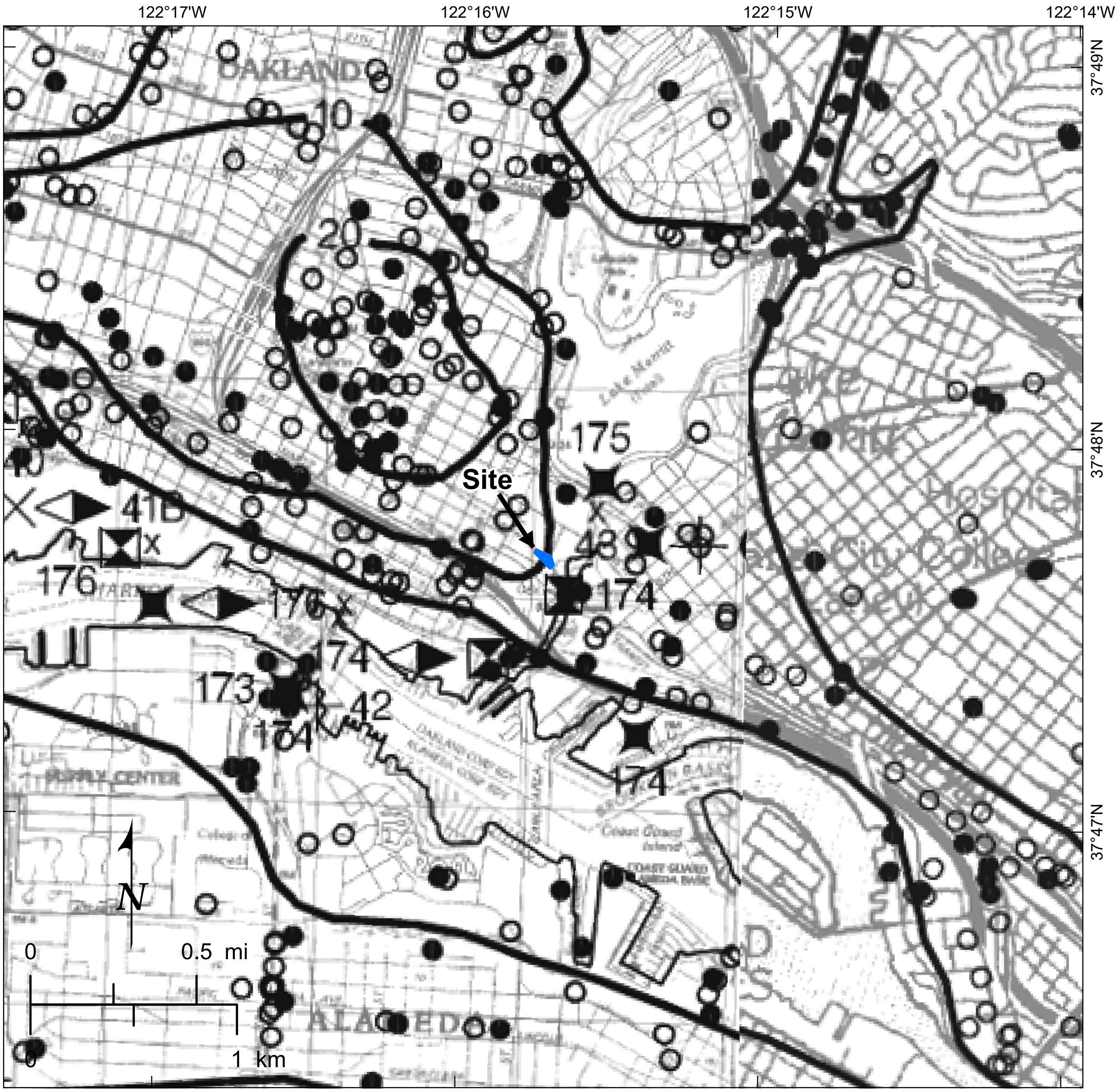
W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\04_Outputs\2020_01_09_GeotechReport\MXD\11_CrossSection_E.mxd, 2/19/2020, m.srisabaranjan



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

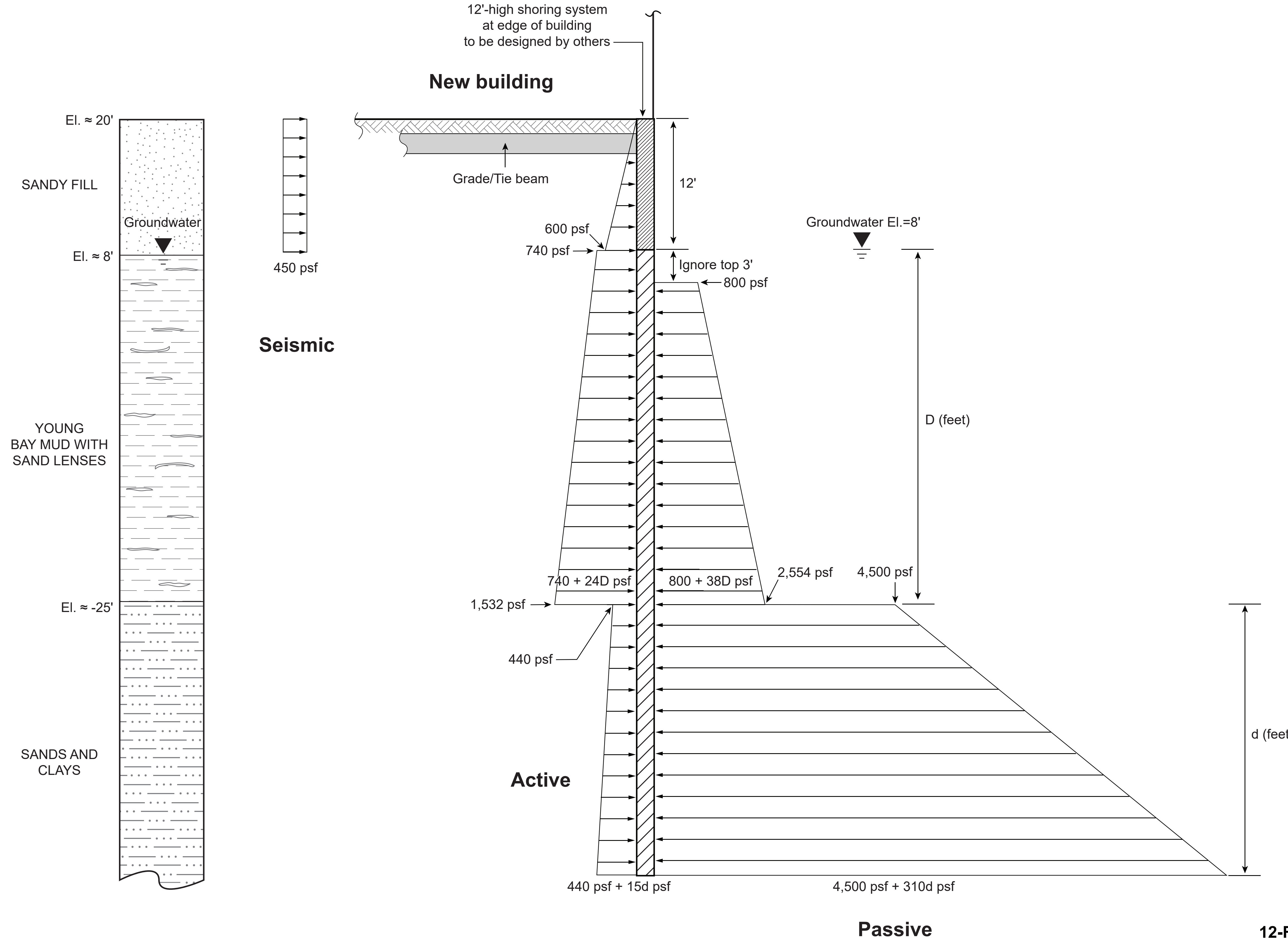
Historical Ground Failures (Knudsen et al., 2000)

- Miscellaneous effects
- Ground settlement
- Lateral Spread
- 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

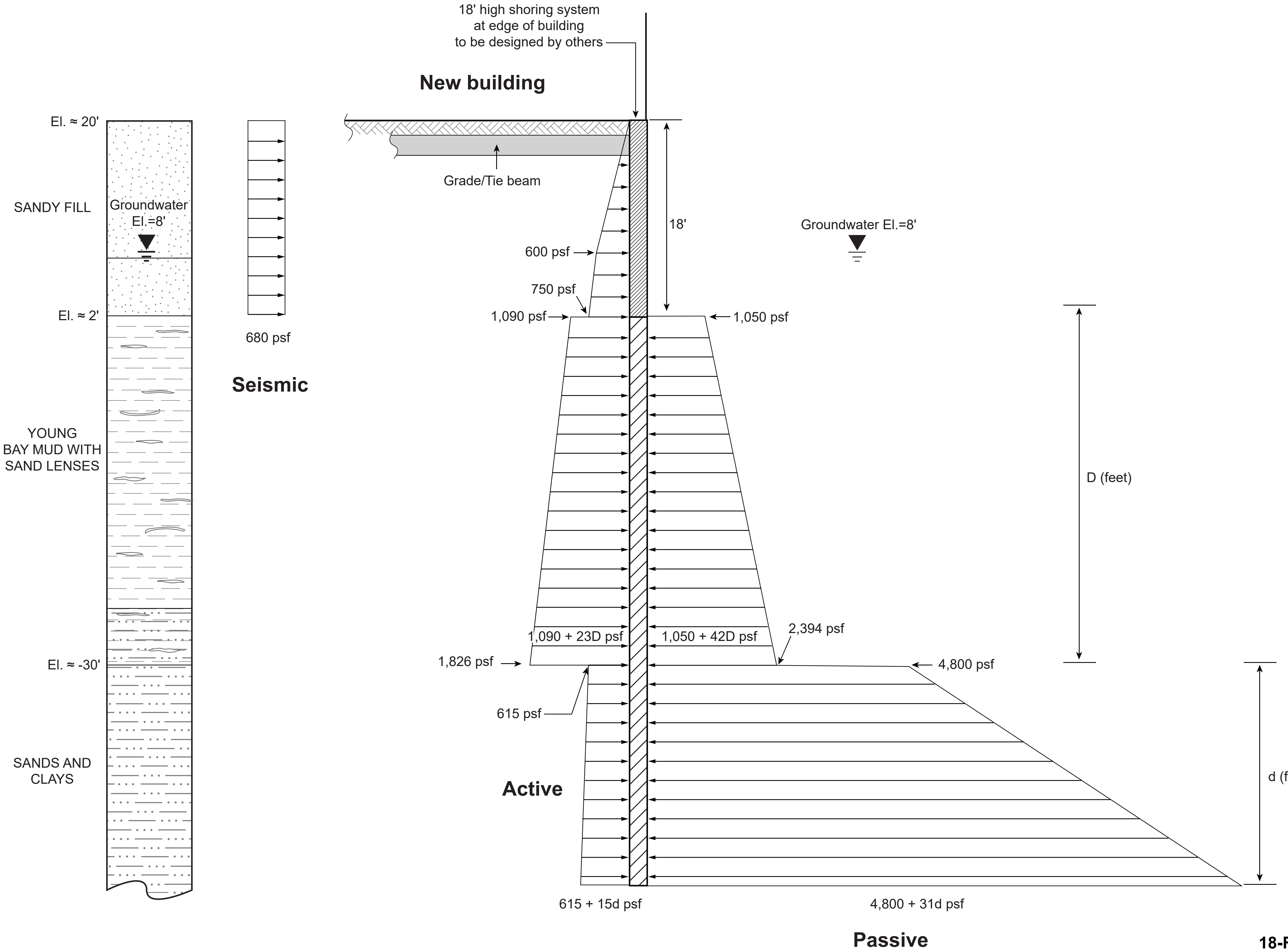
PLATE 13

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\04_Outputs\2019_05_01_GeotechReport\MXD\11_Historical_Liq_Sites.mxd; A.Ramirez; 6/14/2019



RECOMMENDED LATERAL PRESSURES FOR 12-FOOT HIGH SHORING SYSTEM

PLATE 14



RECOMMENDED LATERAL PRESSURES FOR 18-FOOT HIGH SHORING SYSTEM

PLATE 15

Appendix 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

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

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DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	LOCATION: N 37.794900+/- E 122.261959+/- WGS84 SURFACE EL: 14.1 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 with GRAVEL (SM): medium dense, light gray, dry, fine- to medium-grained, trace coarse-grained, silty, with gravel (fine to coarse, subangular to subrounded)							
						PEAT (PT): very soft to soft, black, dry, with organic odor.		55					
5						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

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

LOCATION:

N 37.794856+/- E 122.262089+/- WGS88
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
						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)							
15		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							
-10							52	83					Consol
	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

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Laney College Library Learning Resource Center
Oakland, California

FUGRO

LOCATION:

N 37.794856+/- E 122.262089+/- WGS84
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
			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.

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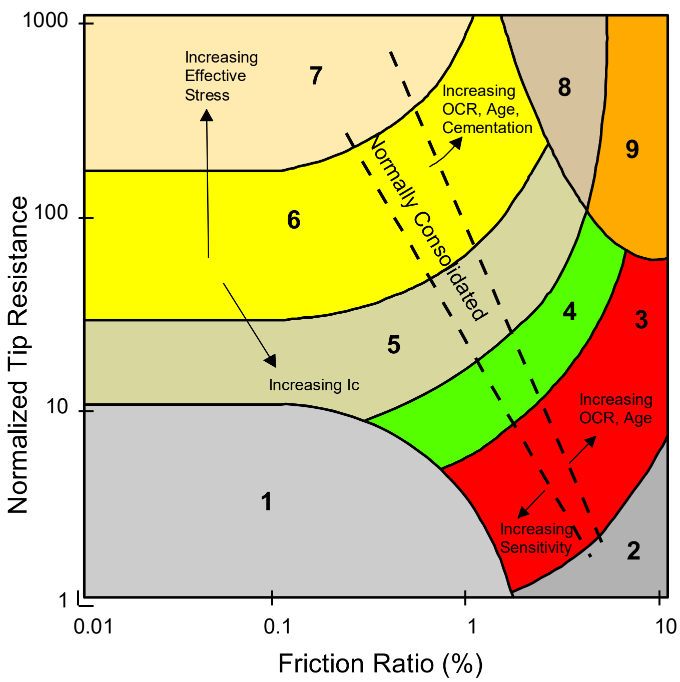
CPT CORRELATION CHART
(Robertson 1990)



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

KEY TO CPT INTERPRETATION

PLATE A-7

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\01_Explorations\CPT\2019\Processed\MXD\CPTproc_WB19C_Key.mxd, 4/30/2019, A.Ramirez

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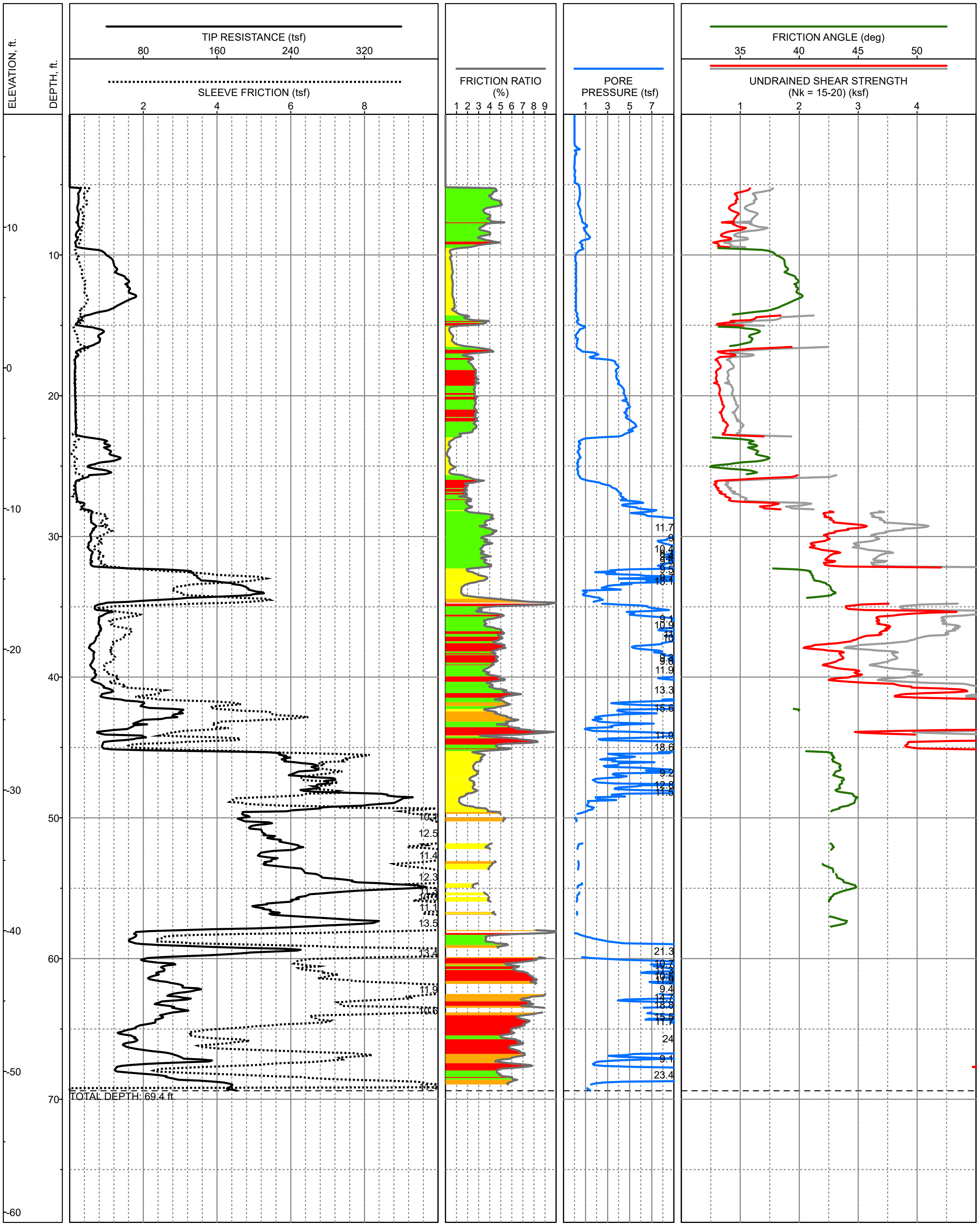
OAKLAND, CALIFORNIA



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: T.CHEN
CONE AREA RATIO: 0.59

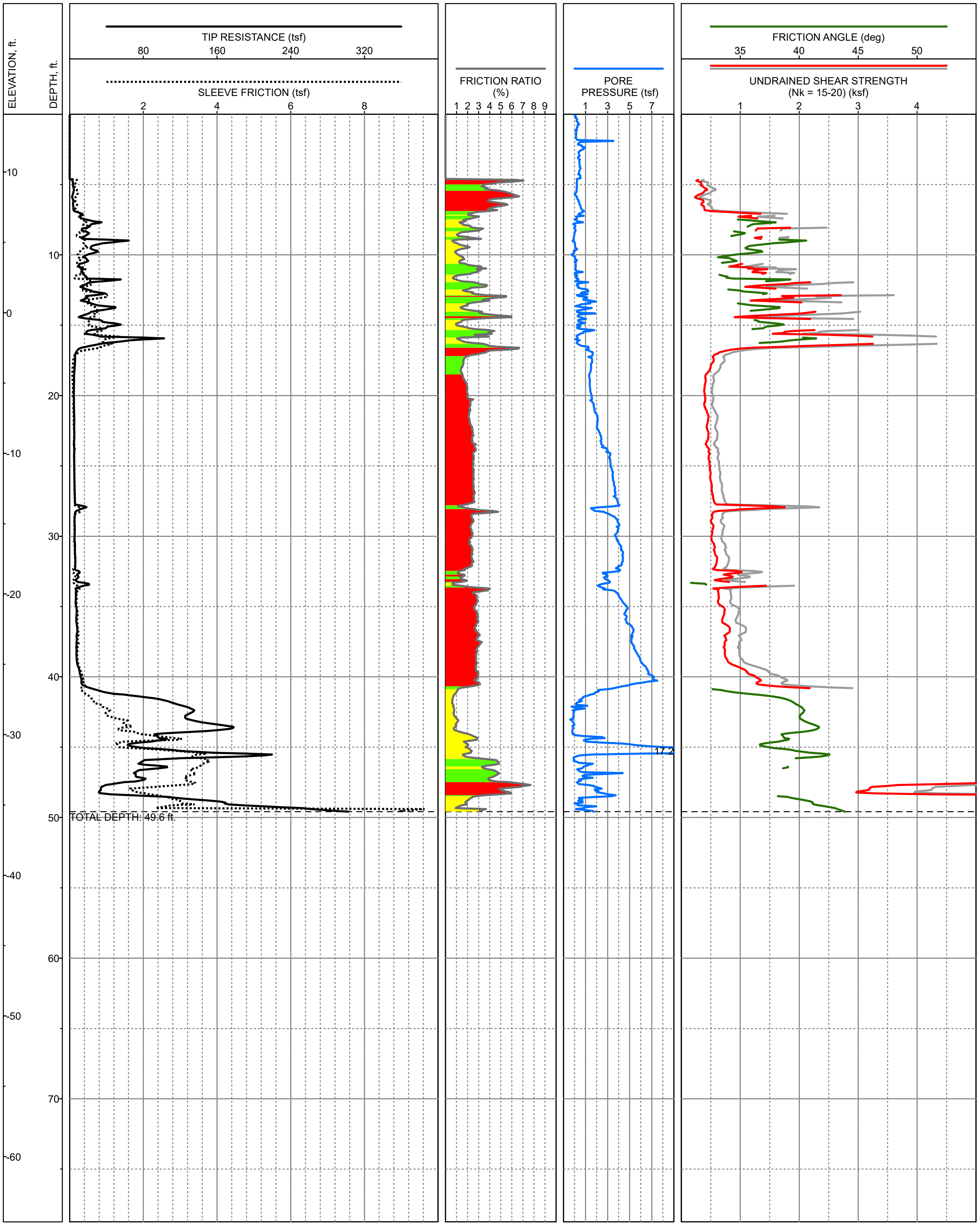
LOG OF 2019-CPT-01

PLATE A-8

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\01_Explorations\CPT\2019\Logs\2019_04-30_Logs_SuFr\MXD\CPTlogs_WB19C_SuFr,4/30/2019,A.Ramirez

PERALTA COMMUNITY COLLEGE DISTRICT
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER
OAKLAND, CALIFORNIA

FUGRO



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

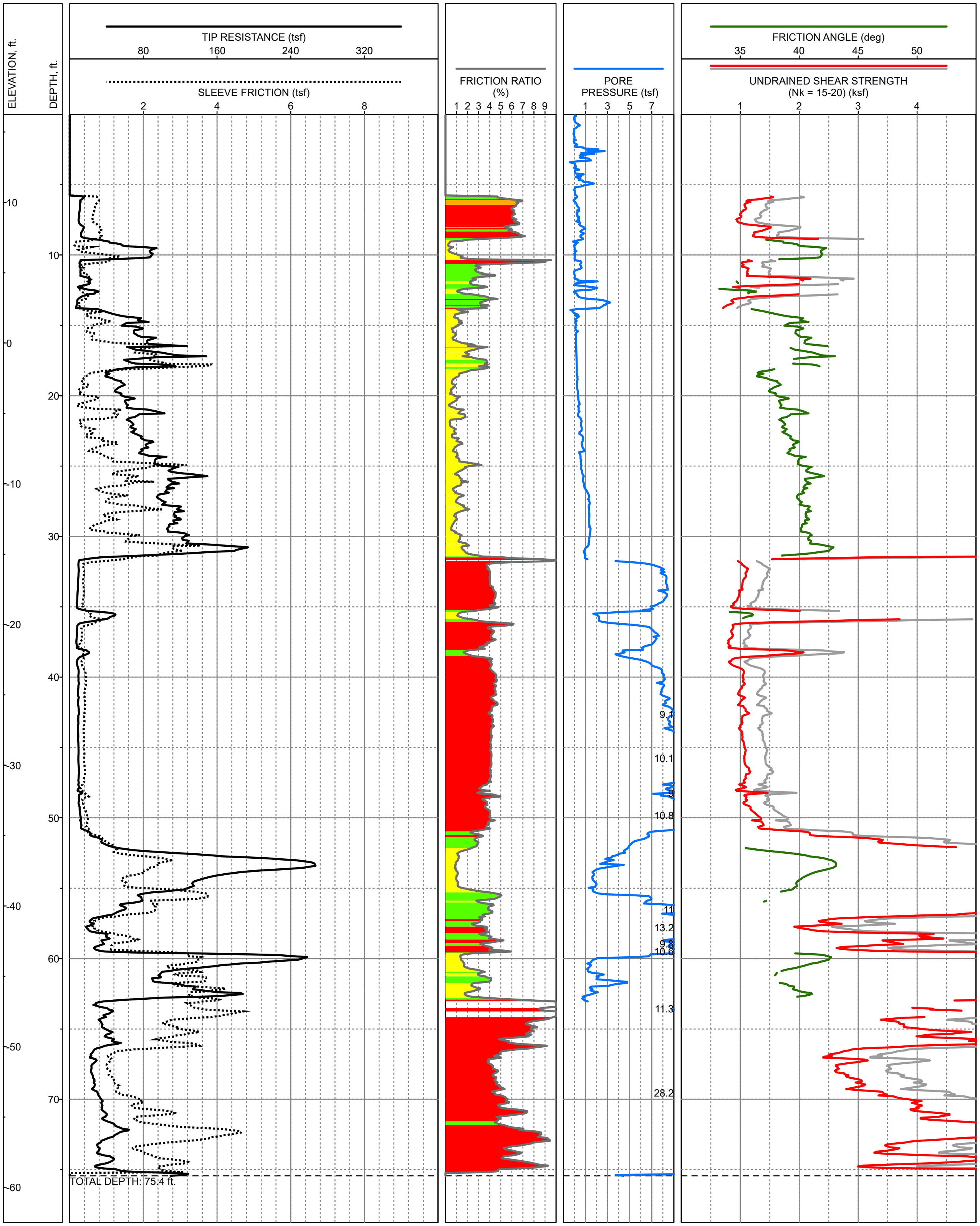
EXPLORATION METHOD: CPT
PERFORMED BY: FUGRO
REVIEWED BY: T.CHEN
CONE AREA RATIO: 0.59

LOG OF 2019-CPT-02

PLATE A-9

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\01_Explorations\CPT\2019\Logs\2019_04-30_Logs_SuFr\MXD\CPTlogs_WB19C_SuFr,4/30/2019,A.Ramirez

PERALTA COMMUNITY COLLEGE DISTRICT
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER
OAKLAND, CALIFORNIA



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: T.CHEN
CONE AREA RATIO: 0.59

LOG OF 2019-CPT-03

PLATE A-10

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PERALTA COMMUNITY COLLEGE DISTRICT
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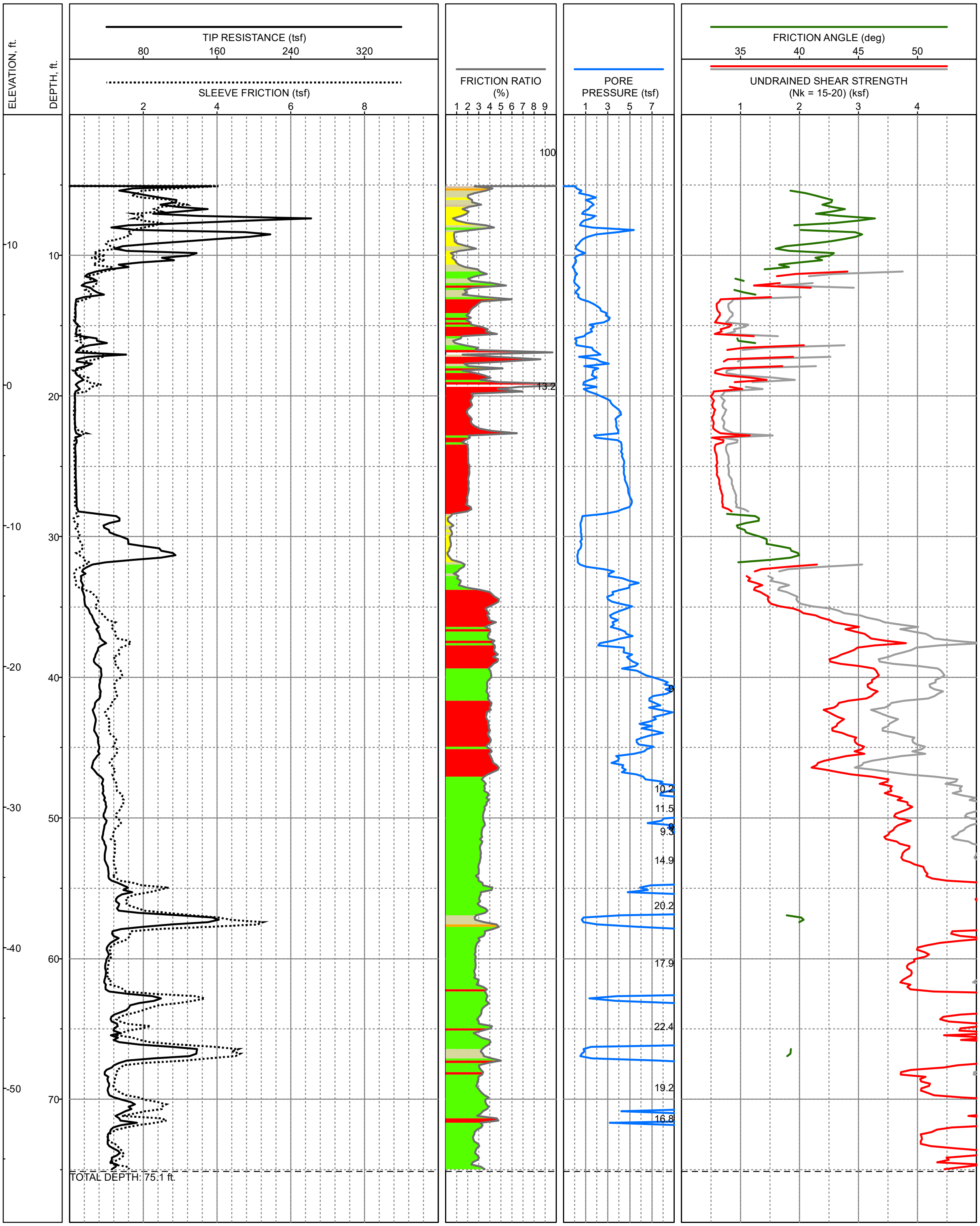
OAKLAND, CALIFORNIA



LOCATION: E6,052,487, N2,116,769, 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: T.CHEN
CONE AREA RATIO: 0.80

LOG OF 2020-CPT-04

PLATE A-11

Fugro Document No. 04.72190021

W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\01_Explorations\CPT\2020\Logs\2020_01_07_Logs_SuFr\MXD\CPTlogs_WB20C_SuFr,1/7/2020,A.Ramirez

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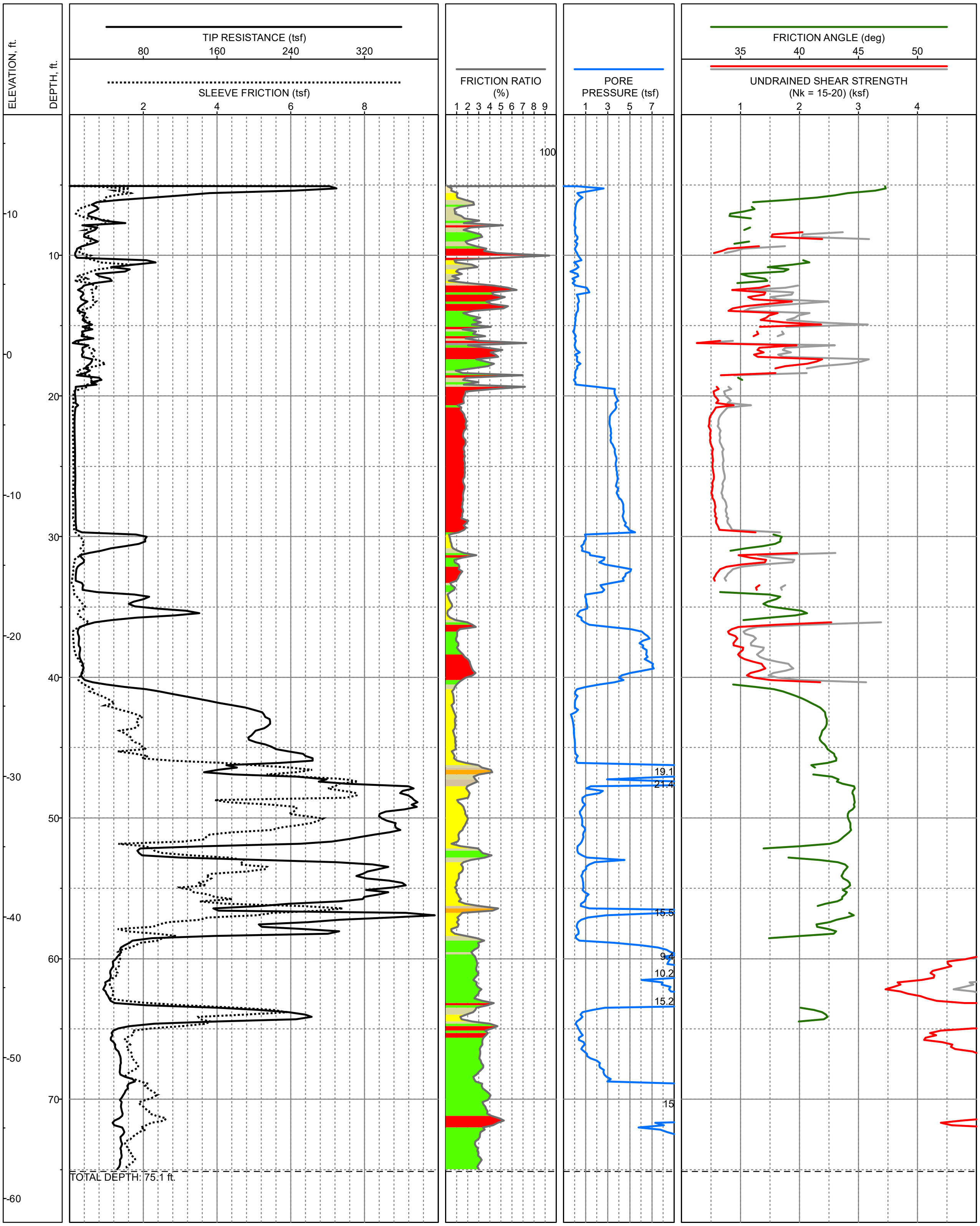
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER
OAKLAND, CALIFORNIA

FUGRO



LOCATION: E6,052,553, N2,116,736, 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: T.CHEN
CONE AREA RATIO: 0.80

LOG OF 2020-CPT-05

PLATE A-12

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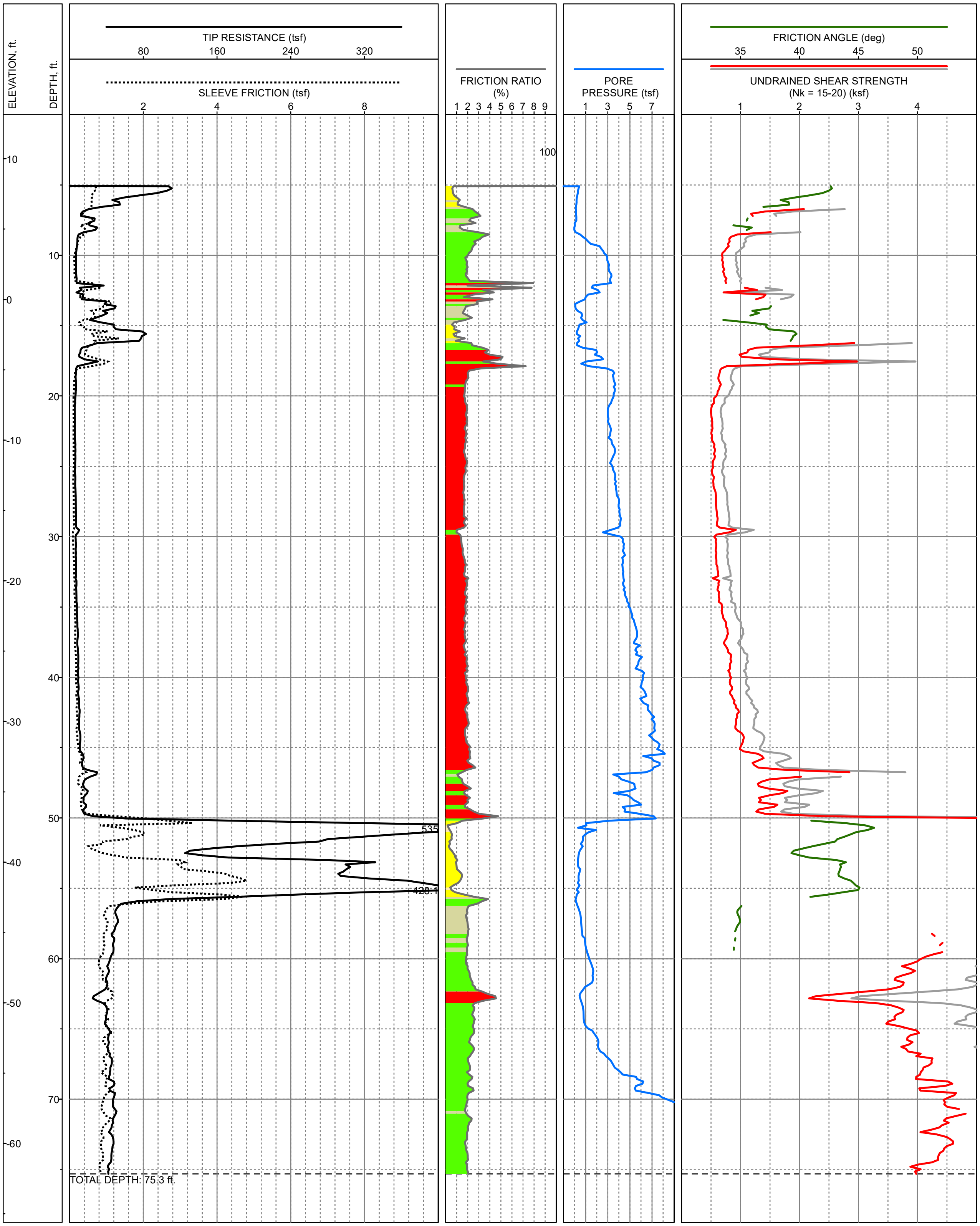
OAKLAND, CALIFORNIA



LOCATION: E6,052,629, N2,116,634, 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: T.CHEN
CONE AREA RATIO: 0.80

LOG OF 2020-CPT-06

PLATE A-13

Fugro Document No. 04.72190021

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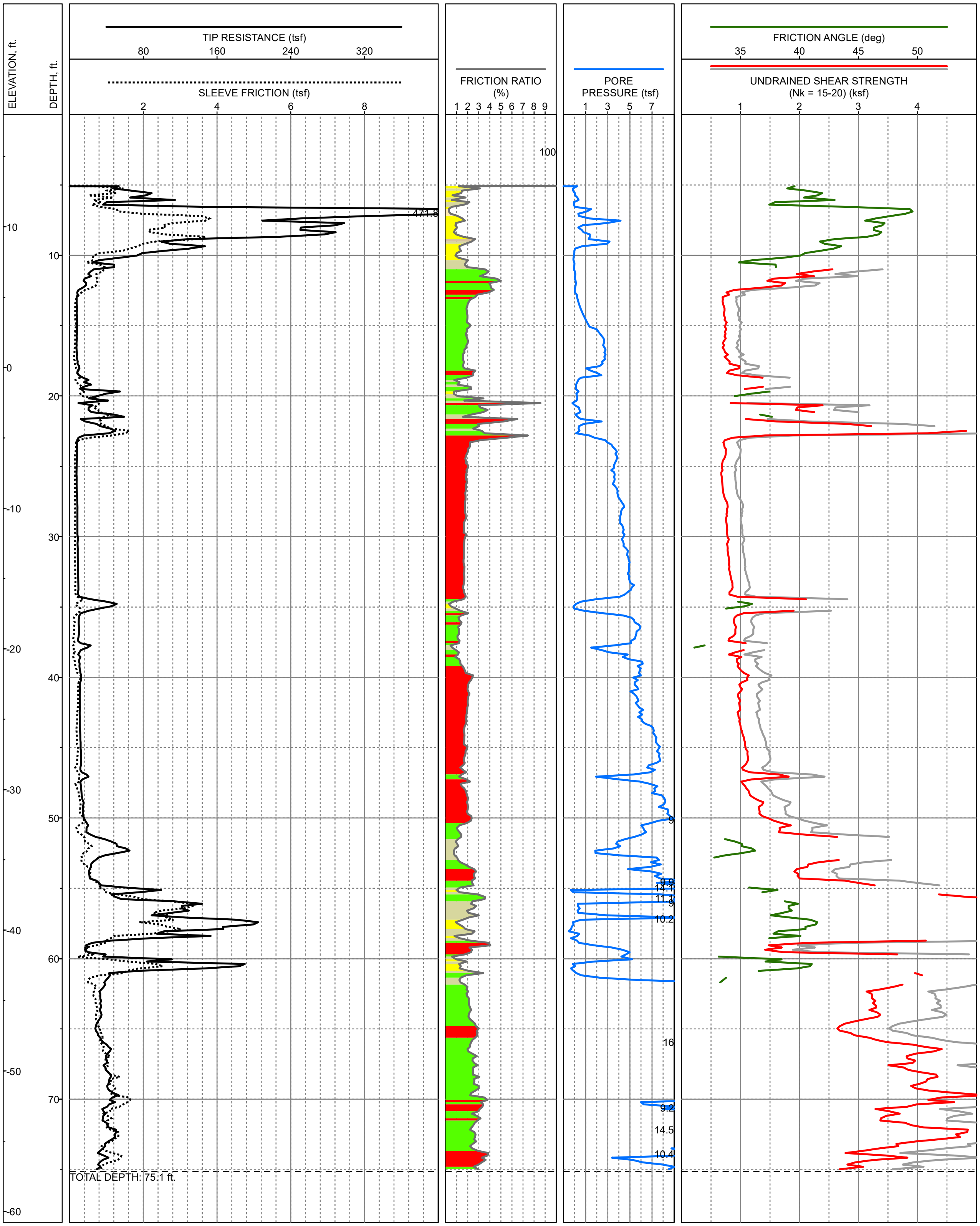
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER
OAKLAND, CALIFORNIA



LOCATION: E6,052,568, N2,116,600, 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: T.CHEN
CONE AREA RATIO: 0.80

LOG OF 2020-CPT-07

PLATE A-14

Fugro Document No. 04.72190021

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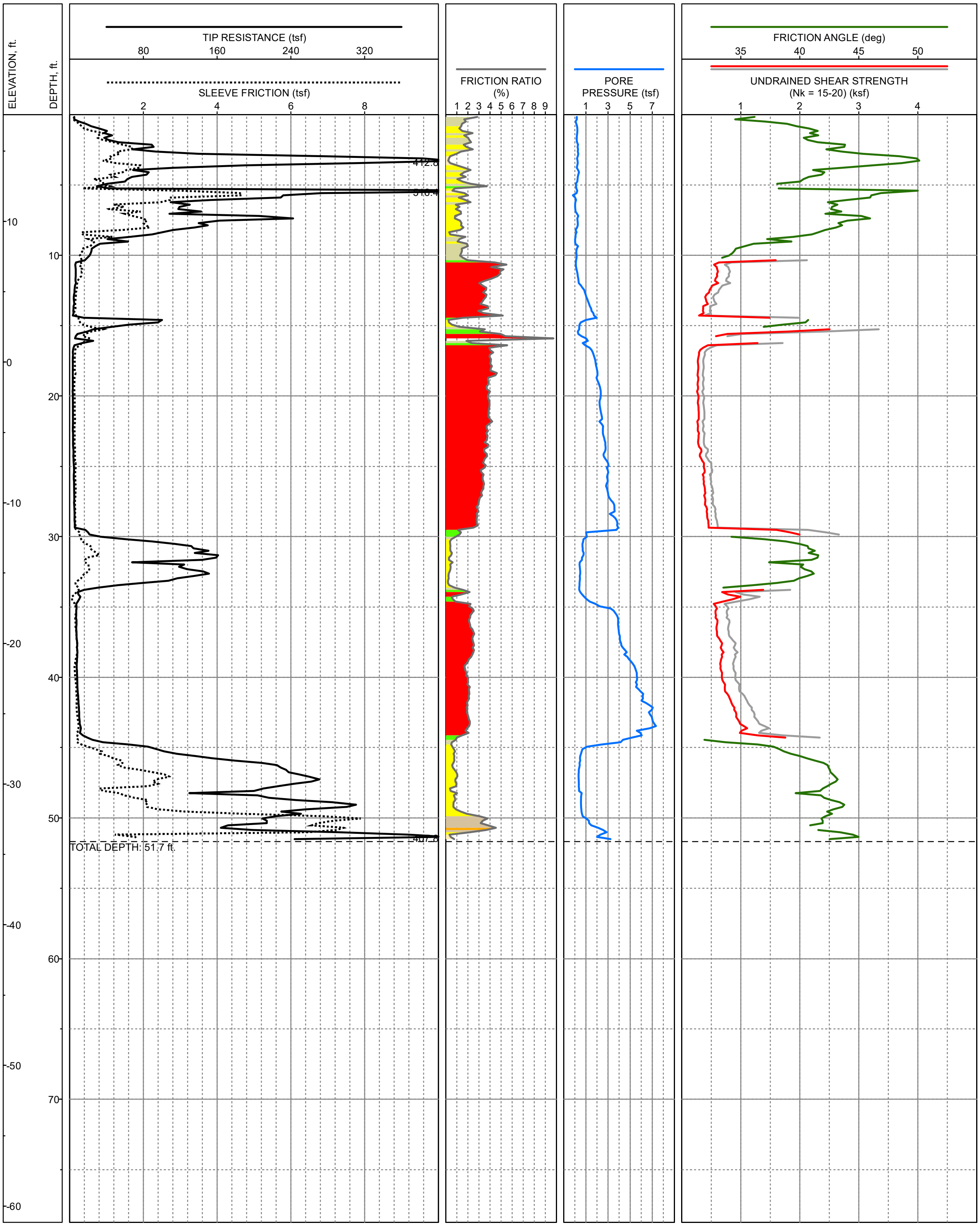
PERALTA COMMUNITY COLLEGE DISTRICT
LANEY COLLEGE LIBRARY LEARNING RESOURCE CENTER
OAKLAND, CALIFORNIA

FUGRO



LOCATION: E6,052,481, N2,116,627, 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: T.CHEN
CONE AREA RATIO: 0.80

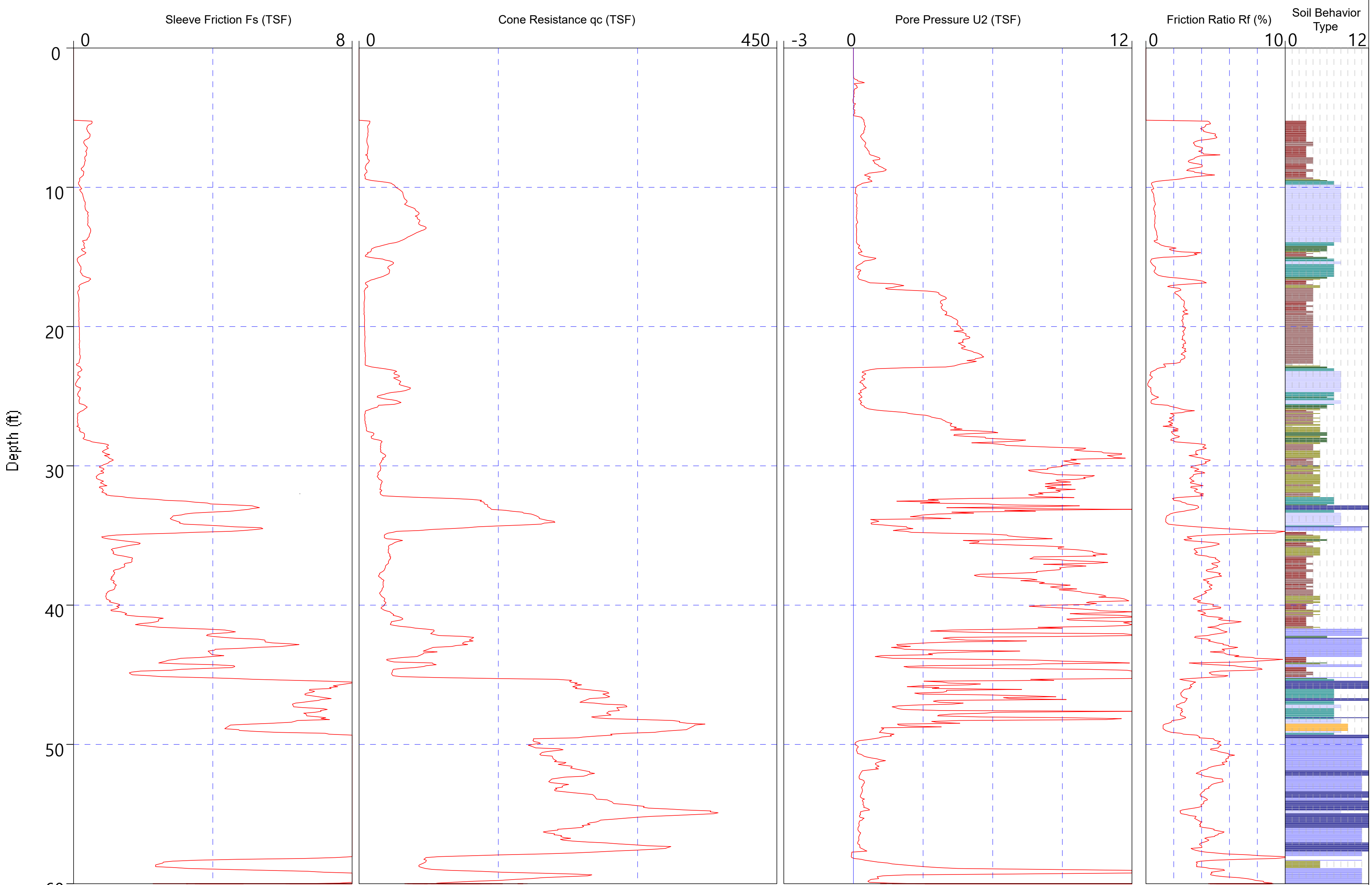
LOG OF 2020-CPT-08

PLATE A-15

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



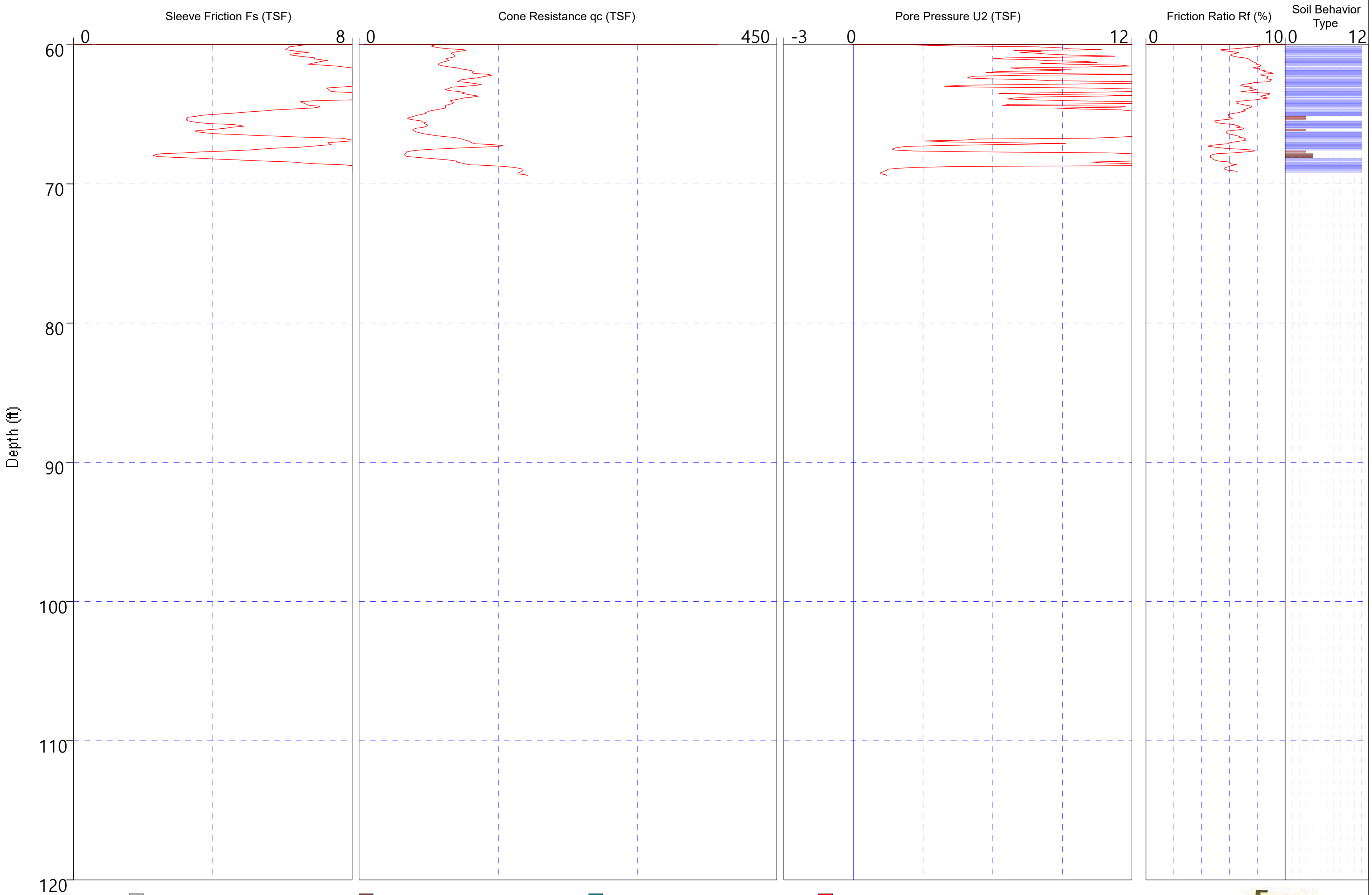
(1) sensitive fine grained (OL-CH)
(2) organic material (OL-OH)
(3) clay (CH)
(4) silty clay to clay (CL-CH)
(5) clayey silt to silty clay (MH-CL)
(6) sandy silt to clayey silt (ML-MH)
(7) silty sand to sandy silt (SM-ML)
(8) sand to silty sand (SM-SP)
(9) sand (SW-SP)
(10) gravel to gravelly sand (SW-GW)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

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



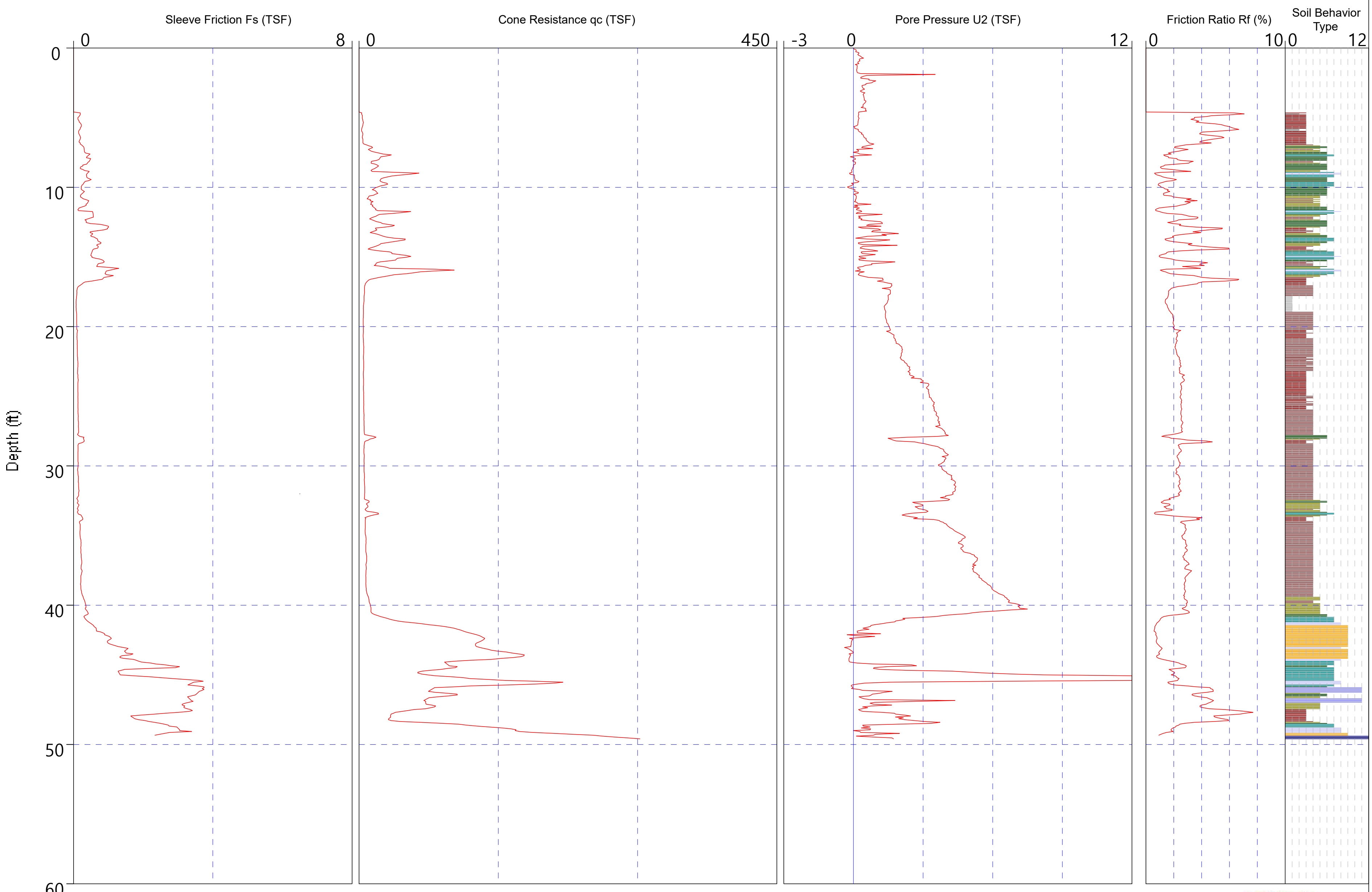
(1) sensitive fine grained (OL-CH)
(2) organic material (OL-OH)
(3) clay (CH)
(4) silty clay to clay (CL-CH)
(5) clayey silt to silty clay (MH-CL)
(6) sandy silt to clayey silt (ML-MH)
(7) silty sand to sandy silt (SM-ML)
(8) sand to silty sand (SM-SP)
(9) sand (SW-SP)
(10) gravel to gravelly sand (SW-GW)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

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



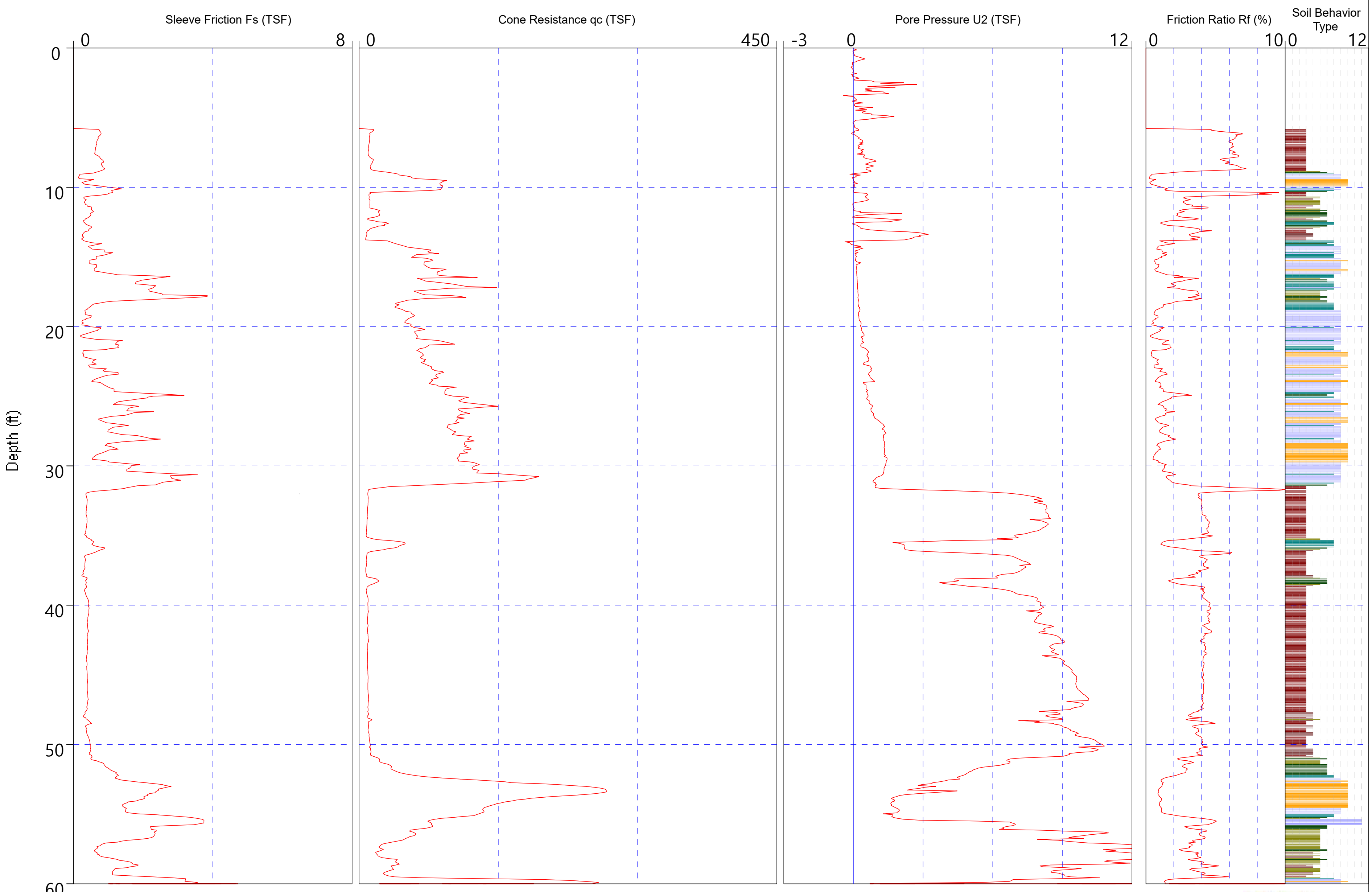
(1) sensitive fine grained (OL-CH)
(2) organic material (OL-OH)
(3) clay (CH)
(4) silty clay to clay (CL-CH)
(5) clayey silt to silty clay (MH-CL)
(6) sandy silt to clayey silt (ML-MH)
(7) silty sand to sandy silt (SM-ML)
(8) sand to silty sand (SM-SP)
(9) sand (SW-SP)
(10) gravel to gravelly sand (SW-GW)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

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



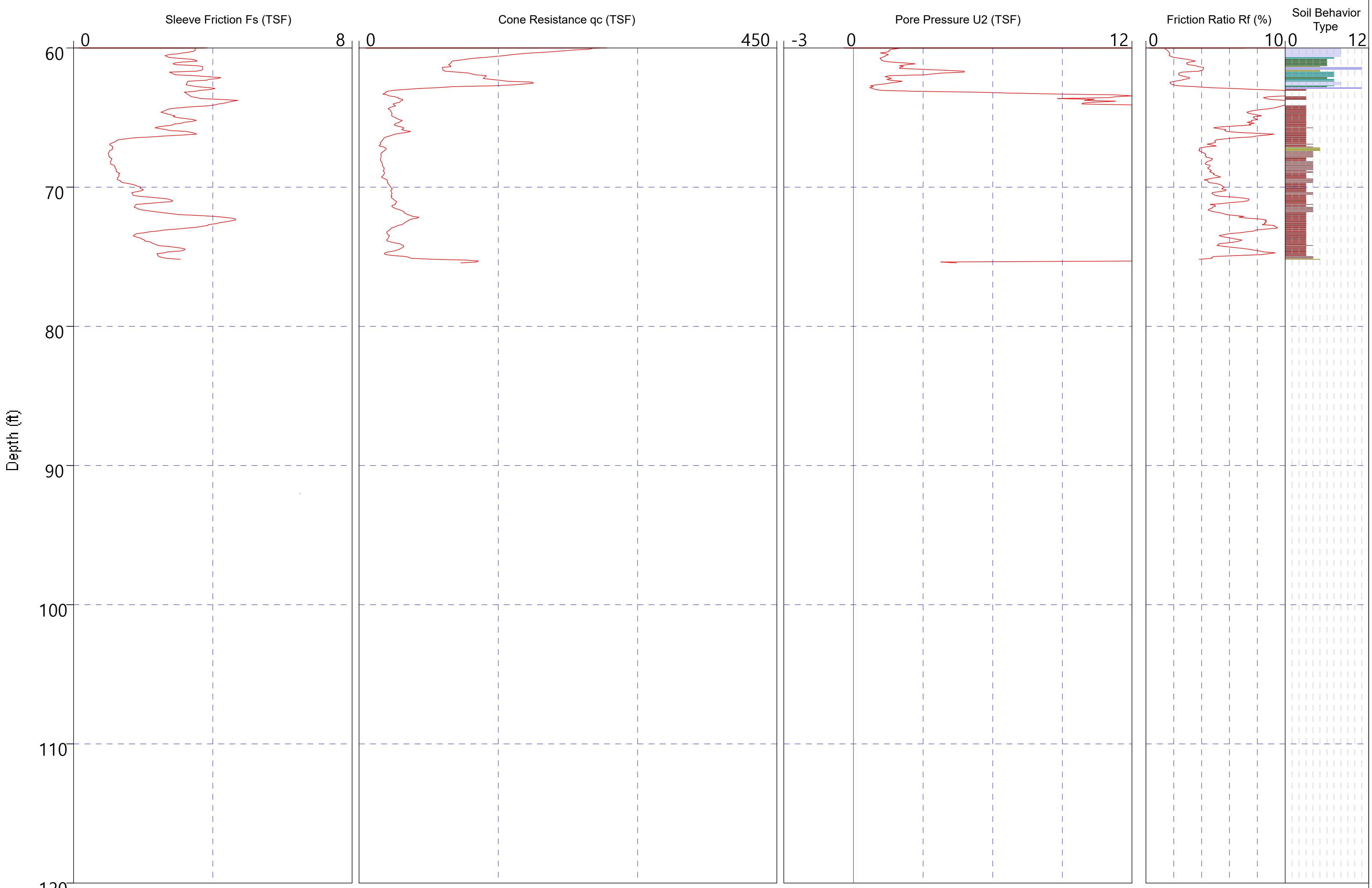
(1) sensitive fine grained (OL-CH)
(2) organic material (OL-OH)
(3) clay (CH)
(4) silty clay to clay (CL-CH)
(5) clayey silt to silty clay (MH-CL)
(6) sandy silt to clayey silt (ML-MH)
(7) silty sand to sandy silt (SM-ML)
(8) sand to silty sand (SM-SP)
(9) sand (SW-SP)
(10) gravel to gravelly sand (SW-GW)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

Robertson et al. 1986 *Overconsolidated or Cemented

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

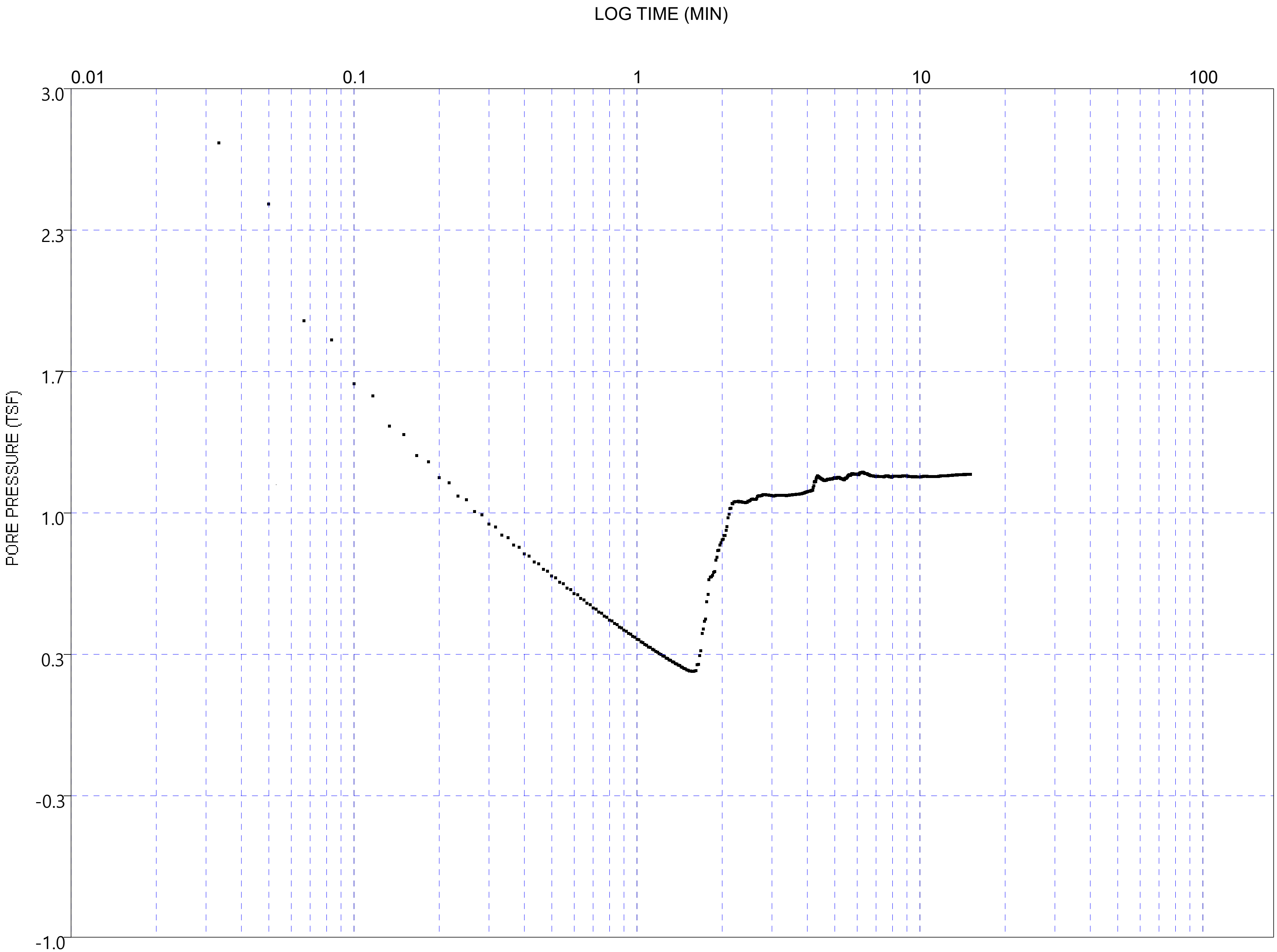
CPT Number: CPT-03
Date: 29-Mar-2019
Elevation: 0.00

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)

Robertson et al. 1986 *Overconsolidated or Cemented



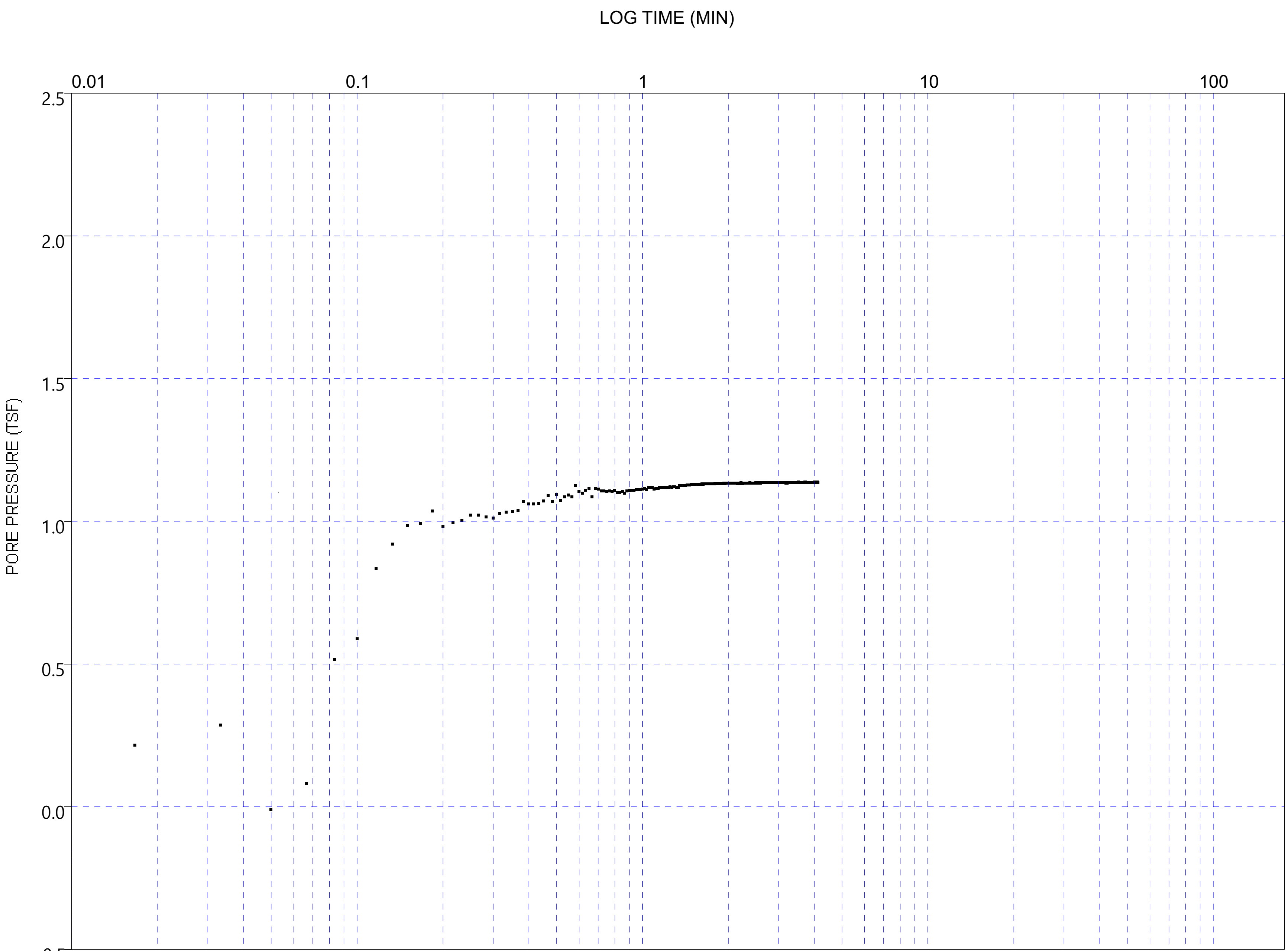
DISSIPATION TEST

CPT Number: CPT-01

Job Number: 04.72190021

Depth: 48.77

Date: 29-Mar-2019

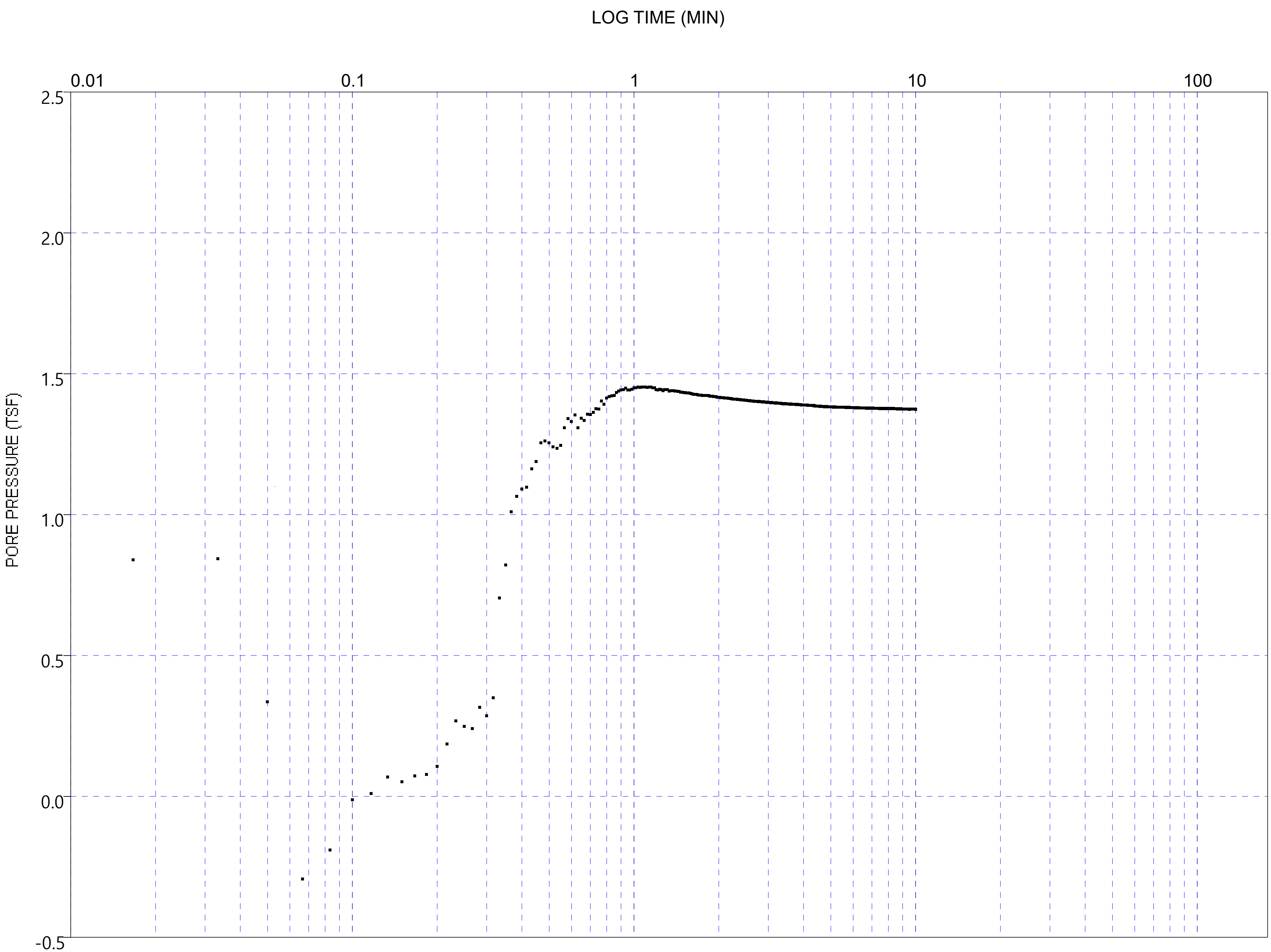


DISSIPATION TEST

CPT Number: CPT-02
Job Number: 04.72190021

Depth: 42.02
Date: 29-Mar-2019

FUGRO Consultants, Inc.



DISSIPATION TEST

CPT Number: CPT-02

Job Number: 04.72190021

Depth: 49.33

Date: 29-Mar-2019

FUGRO Consultants, Inc.

GREGG DRILLING, LLC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

January 7, 2020

Fugro
Attn: Reza Rahimnejad

Subject: CPT Site Investigation
Laney College
Oakland, California
GREGG Project Number: D2205001

Dear Mr. Rahimnejad:

The following report presents the results of GREGG Drilling Cone Penetration Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	☒
2	Pore Pressure Dissipation Tests	(PPD)	☒
3	Seismic Cone Penetration Tests	(SCPTU)	☒
4	UVOST Laser Induced Fluorescence	(UVOST)	☐
5	Groundwater Sampling	(GWS)	☐
6	Soil Sampling	(SS)	☐
7	Vapor Sampling	(VS)	☐
8	Pressuremeter Testing	(PMT)	☐
9	Vane Shear Testing	(VST)	☐
10	Dilatometer Testing	(DMT)	☐

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact me at 714-863-0988.

Sincerely,
Gregg Drilling, LLC.

CPT Reports Team
Gregg Drilling, LLC.

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

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

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950 Howe Road. • Martinez, California 94553 • (925) 313-5800 • FAX (925) 313-0302
www.greggdrilling.com

Bibliography

Lunne, T., Robertson, P.K. and Powell, J.J.M., “Cone Penetration Testing in Geotechnical Practice” E & FN Spon. ISBN 0 419 23750, 1997

Roberston, P.K., “Soil Classification using the Cone Penetration Test”, Canadian Geotechnical Journal, Vol. 27, 1990 pp. 151-158.

Mayne, P.W., “NHI (2002) Manual on Subsurface Investigations: Geotechnical Site Characterization”, available through www.ce.gatech.edu/~geosys/Faculty/Mayne/papers/index.html, Section 5.3, pp. 107-112.

Robertson, P.K., R.G. Campanella, D. Gillespie and A. Rice, “Seismic CPT to Measure In-Situ Shear Wave Velocity”, Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8, 1986 pp. 791-803.

Robertson, P.K., Sully, J., Woeller, D.J., Lunne, T., Powell, J.J.M., and Gillespie, D.J., "Guidelines for Estimating Consolidation Parameters in Soils from Piezocone Tests", Canadian Geotechnical Journal, Vol. 29, No. 4, August 1992, pp. 539-550.

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Campanella, R.G. and I. Weemees, “Development and Use of An Electrical Resistivity Cone for Groundwater Contamination Studies”, Canadian Geotechnical Journal, Vol. 27 No. 5, 1990 pp. 557-567.

DeGroot, D.J. and A.J. Lutenegger, “Reliability of Soil Gas Sampling and Characterization Techniques”, International Site Characterization Conference - Atlanta, 1998.

Woeller, D.J., P.K. Robertson, T.J. Boyd and Dave Thomas, “Detection of Polyaromatic Hydrocarbon Contaminants Using the UVIF-CPT”, 53rd Canadian Geotechnical Conference Montreal, QC October pp. 733-739, 2000.

Zemo, D.A., T.A. Delfino, J.D. Gallinatti, V.A. Baker and L.R. Hilpert, “Field Comparison of Analytical Results from Discrete-Depth Groundwater Samplers” BAT EnviroProbe and QED HydroPunch, Sixth national Outdoor Action Conference, Las Vegas, Nevada Proceedings, 1992, pp 299-312.

Copies of ASTM Standards are available through www.astm.org

Cone Penetration Testing Procedure (CPT)

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*.

The cone takes measurements of tip resistance (q_c), sleeve resistance (f_s), and penetration pore water pressure (u_2). Measurements are taken at either 2.5 or 5 cm intervals during penetration to provide a nearly continuous profile. CPT data reduction and basic interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored electronically for further analysis and reference. All CPT soundings are performed in accordance with revised ASTM standards (D 5778-12).

The 5mm thick porous plastic filter element is located directly behind the cone tip in the u_2 location. A new saturated filter element is used on each sounding to measure both penetration pore pressures as well as measurements during a dissipation test (*PPDT*). Prior to each test, the filter element is fully saturated with oil under vacuum pressure to improve accuracy.

When the sounding is completed, the test hole is backfilled according to client specifications. If grouting is used, the procedure generally consists of pushing a hollow tremie pipe with a "knock out" plug to the termination depth of the CPT hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

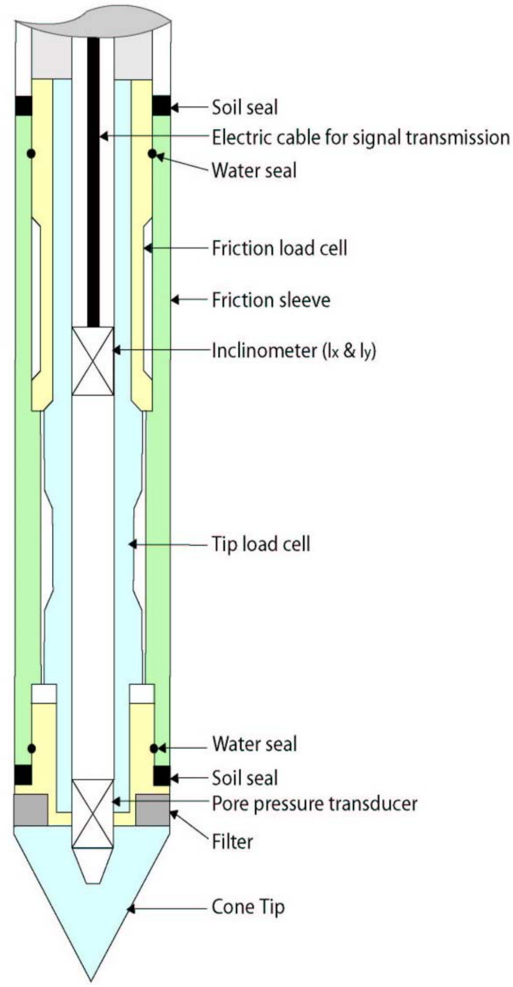


Figure CPT

Gregg $15cm^2$ Standard Cone Specifications

Dimensions	
Cone base area	15 cm^2
Sleeve surface area	225 cm^2
Cone net area ratio	0.80
Specifications	
Cone load cell	
Full scale range	180 kN (20 tons)
Overload capacity	150%
Full scale tip stress	120 MPa (1,200 tsf)
Repeatability	120 kPa (1.2 tsf)
Sleeve load cell	
Full scale range	31 kN (3.5 tons)
Overload capacity	150%
Full scale sleeve stress	1,400 kPa (15 tsf)
Repeatability	1.4 kPa (0.015 tsf)
Pore pressure transducer	
Full scale range	7,000 kPa (1,000 psi)
Overload capacity	150%
Repeatability	7 kPa (1 psi)

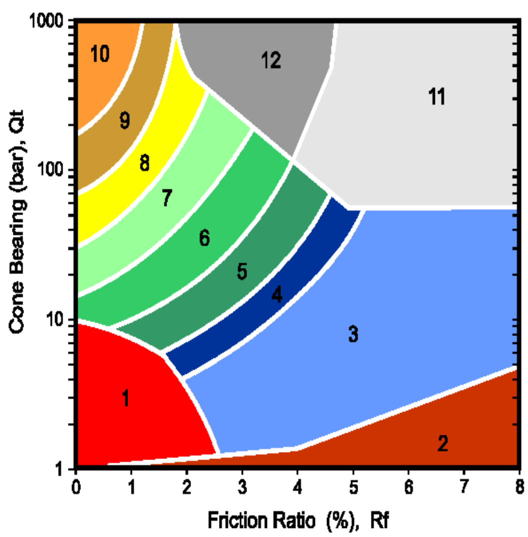
Note: The repeatability during field use will depend somewhat on ground conditions, abrasion, maintenance and zero load stability.

Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected are presented in graphical and electronic form in the report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the non-normalized charts of Robertson et al (1986). For CPT soundings deeper than 30m, we recommend the use of the normalized charts of Robertson (1990) which can be displayed as SBTn, upon request. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson (Guide to Cone Penetration Testing, 2015). The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling & Testing Inc. does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software. Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on q_t, f_s, and u_2. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.



ZONE		SBT
1		Sensitive, fine grained
2		Organic materials
3		Clay
4		Silty clay to clay
5		Clayey silt to silty clay
6		Sandy silt to clayey silt
7		Silty sand to sandy silt
8		Sand to silty sand
9		Sand
10		Gravely sand to sand
11		Very stiff fine grained*
12		Sand to clayey sand*

*over consolidated or cemented

Figure SBT (After Robertson et al., 1986) – Note: Colors may vary slightly compared to plots

Cone Penetration Test (CPT) Interpretation

Gregg uses a proprietary CPT interpretation and plotting software. The software takes the CPT data and performs basic interpretation in terms of soil behavior type (SBT) and various geotechnical parameters using current published empirical correlations based on the comprehensive review by Lunne, Robertson and Powell (1997). The interpretation is presented in tabular format using MS Excel. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

The following provides a summary of the methods used for the interpretation. Many of the empirical correlations to estimate geotechnical parameters have constants that have a range of values depending on soil type, geologic origin and other factors. The software uses 'default' values that have been selected to provide, in general, conservatively low estimates of the various geotechnical parameters.

Input:

1. Units for display (Imperial or metric) (atm. pressure, p_a = 0.96 tsf or 0.1 MPa)
2. Depth interval to average results (ft or m). Data are collected at either 0.02 or 0.05m and can be averaged every 1, 3 or 5 intervals.
3. Elevation of ground surface (ft or m)
4. Depth to water table, z_w (ft or m) – input required
5. Net area ratio for cone, a (default to 0.80)
6. Relative Density constant, C_{Dr} (default to 350)
7. Young's modulus number for sands, α (default to 5)
8. Small strain shear modulus number
   a. for sands, S_G (default to 180 for SBT_n 5, 6, 7)
   b. for clays, C_G (default to 50 for SBT_n 1, 2, 3 & 4)
9. Undrained shear strength cone factor for clays, N_{kt} (default to 15)
10. Over Consolidation ratio number, k_{ocr} (default to 0.3)
11. Unit weight of water, (default to γ_w = 62.4 lb/ft^3 or 9.81 kN/m^3)

Column

1. Depth, z, (m) – CPT data is collected in meters
2. Depth (ft)
3. Cone resistance, q_c (tsf or MPa)
4. Sleeve resistance, f_s (tsf or MPa)
5. Penetration pore pressure, u (psi or MPa), measured behind the cone (i.e. u_2)
6. Other – any additional data
7. Total cone resistance, q_t (tsf or MPa) $q_t = q_c + u\,(1-a)$

8	Friction Ratio, R_f (%)	$R_f = (f_s/q_t) \times 100\%$
9	Soil Behavior Type (non-normalized), SBT	see note
10	Unit weight, γ (pcf or kN/m^3)	based on SBT, see note
11	Total overburden stress, σ_v (tsf)	$\sigma_{vo} = \sigma\, z$
12	In-situ pore pressure, u_o (tsf)	$u_o = \gamma_w\,(z - z_w)$
13	Effective overburden stress, σ'_{vo} (tsf)	$\sigma'_{vo} = \sigma_{vo} - u_o$
14	Normalized cone resistance, Q_{t1}	$Q_{t1} = (q_t - \sigma_{vo}) / \sigma'_{vo}$
15	Normalized friction ratio, F_r (%)	$F_r = f_s / (q_t - \sigma_{vo}) \times 100\%$
16	Normalized Pore Pressure ratio, B_q	$B_q = u - u_o / (q_t - \sigma_{vo})$
17	Soil Behavior Type (normalized), SBT_n	see note
18	SBT_n Index, I_c	see note
19	Normalized Cone resistance, Q_{tn} (n varies with I_c)	see note
20	Estimated permeability, k_{SBT} (cm/sec or ft/sec)	see note
21	Equivalent SPT N_{60}, blows/ft	see note
22	Equivalent SPT $(N_1)_{60}$ blows/ft	see note
23	Estimated Relative Density, D_r, (%)	see note
24	Estimated Friction Angle, ϕ', (degrees)	see note
25	Estimated Young's modulus, E_s (tsf)	see note
26	Estimated small strain Shear modulus, Go (tsf)	see note
27	Estimated Undrained shear strength, s_u (tsf)	see note
28	Estimated Undrained strength ratio	s_u/σ_v'
29	Estimated Over Consolidation ratio, OCR	see note

Notes:

1 Soil Behavior Type (non-normalized), SBT (Lunne et al., 1997 and table below)

2 Unit weight, γ either constant at 119 pcf or based on Non-normalized SBT (Lunne et al., 1997 and table below)

3 Soil Behavior Type (Normalized), SBT_n Lunne et al. (1997)

4 SBT_n Index, I_c $I_c = ((3.47 - \log Q_{t1})^2 + (\log F_r + 1.22)^2)^{0.5}$

5 Normalized Cone resistance, Q_{tn} (n varies with Ic)

$Q_{tn} = ((q_t - \sigma_{vo})/pa)\,(pa/(\sigma'_{vo})^n$ and recalculate I_c, then iterate:

When $I_c < 1.64$, n = 0.5 (clean sand)
When $I_c > 3.30$, n = 1.0 (clays)
When $1.64 < I_c < 3.30$, $n = (I_c - 1.64)0.3 + 0.5$
Iterate until the change in n, $\Delta n < 0.01$

6 Estimated permeability, k_{SBT} based on Normalized SBT_n (Lunne et al., 1997 and table below)

7 Equivalent SPT N_{60}, blows/ft Lunne et al. (1997)

$$\frac{(q_t/p_a)}{N_{60}} = 8.5\left(1 - \frac{I_c}{4.6}\right)$$

8 Equivalent SPT $(N_1)_{60}$ blows/ft $(N_1)_{60} = N_{60}\,C_{N,}$
where $C_N = (pa/\sigma'_{vo})^{0.5}$

9 Relative Density, D_r, (%) $D_r^2 = Q_{tn} / C_{Dr}$
Only SBT_n 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

10 Friction Angle, ϕ', (degrees) $\tan\phi' = \frac{1}{2.68}\left[\log\left(\frac{q_c}{\sigma'_{vo}}\right) + 0.29\right]$

Only SBT_n 5, 6, 7 & 8 *Show'N/A' in zones 1, 2, 3, 4 & 9*

11 Young's modulus, E_s $E_s = \alpha\, q_t$
Only SBT_n 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

12 Small strain shear modulus, Go
a. $G_o = S_G (q_t\ \sigma'_{vo}\ pa)^{1/3}$ *For SBT_n 5, 6, 7*
b. $G_o = C_G\, q_t$ *For SBT_n 1, 2, 3& 4*
Show 'N/A' in zones 8 & 9

13 Undrained shear strength, s_u $s_u = (q_t - \sigma_{vo}) / N_{kt}$
Only SBT_n 1, 2, 3, 4 & 9 *Show 'N/A' in zones 5, 6, 7 & 8*

14 Over Consolidation ratio, OCR $OCR = k_{ocr}\, Q_{t1}$
Only SBT_n 1, 2, 3, 4 & 9 *Show 'N/A' in zones 5, 6, 7 & 8*

The following updated and simplified SBT descriptions have been used in the software:

SBT Zones		**SBT_n Zones**	
1	sensitive fine grained	1	sensitive fine grained
2	organic soil	2	organic soil
3	clay	3	clay
4	clay & silty clay	4	clay & silty clay
5	clay & silty clay		
6	sandy silt & clayey silt		

7	silty sand & sandy silt	5	silty sand & sandy silt
8	sand & silty sand	6	sand & silty sand
9	sand		
10	sand	7	sand
11	very dense/stiff soil*	8	very dense/stiff soil*
12	very dense/stiff soil*	9	very dense/stiff soil*

*heavily overconsolidated and/or cemented

Track when soils fall with zones of same description and print that description (i.e. if soils fall only within SBT zones 4 & 5, print ‘clays & silty clays’)

Estimated Permeability (see Lunne et al., 1997)

SBT_n	Permeability (ft/sec)	(m/sec)
1	$3x\ 10^{-8}$	$1x\ 10^{-8}$
2	$3x\ 10^{-7}$	$1x\ 10^{-7}$
3	$1x\ 10^{-9}$	$3x\ 10^{-10}$
4	$3x\ 10^{-8}$	$1x\ 10^{-8}$
5	$3x\ 10^{-6}$	$1x\ 10^{-6}$
6	$3x\ 10^{-4}$	$1x\ 10^{-4}$
7	$3x\ 10^{-2}$	$1x\ 10^{-2}$
8	$3x\ 10^{-6}$	$1x\ 10^{-6}$
9	$1x\ 10^{-8}$	$3x\ 10^{-9}$

Estimated Unit Weight (see Lunne et al., 1997)

SBT	Approximate Unit Weight (lb/ft^3)	(kN/m^3)
1	111.4	17.5
2	79.6	12.5
3	111.4	17.5
4	114.6	18.0
5	114.6	18.0
6	114.6	18.0
7	117.8	18.5
8	120.9	19.0
9	124.1	19.5
10	127.3	20.0
11	130.5	20.5
12	120.9	19.0

Pore Pressure Dissipation Tests (PPDT)

Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals can be used to measure equilibrium water pressure (at the time of the CPT). If conditions are hydrostatic, the equilibrium water pressure can be used to determine the approximate depth of the ground water table. A PPDT is conducted when penetration is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure (*u*) with time is measured behind the tip of the cone and recorded.

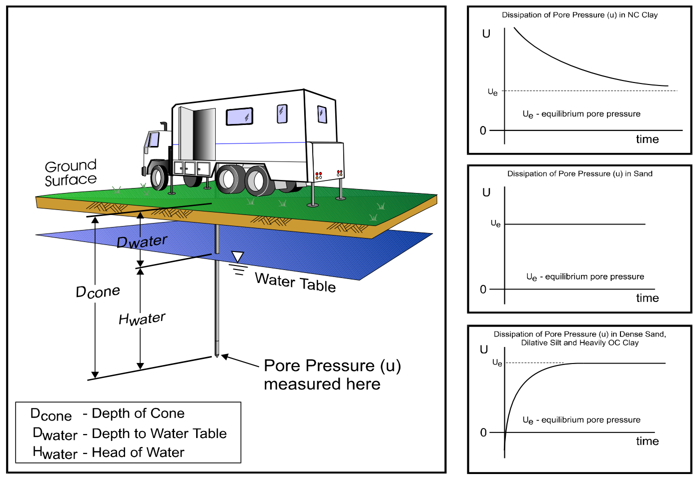
Pore pressure dissipation data can be interpreted to provide estimates of:

- Equilibrium piezometric pressure
- Phreatic Surface
- In situ horizontal coefficient of consolidation (c_h)
- In situ horizontal coefficient of permeability (k_h)

In order to correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until it reaches equilibrium, *Figure PPDT*. This time is commonly referred to as t_{100}, the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1992 and Lunne et al. 1997.

A summary of the pore pressure dissipation tests are summarized in Table 1.



Water Table Calculation

$$D_{water} = D_{cone} - H_{water}$$

where $H_{water} = U_e$ (depth units)

Useful Conversion Factors: 1psi = 0.704m = 2.31 feet (water)
1tsf = 0.958 bar = 13.9 psi
1m = 3.28 feet

Figure PPDT

Seismic Cone Penetration Testing (SCPT)

Seismic Cone Penetration Testing (SCPT) can be conducted at various intervals during the Cone Penetration Test. Shear wave velocity (Vs) can then be calculated over a specified interval with depth. A small interval for seismic testing, such as 1-1.5m (3-5ft) allows for a detailed look at the shear wave profile with depth. Conversely, a larger interval such as 3-6m (10-20ft) allows for a more average shear wave velocity to be calculated. Gregg's cones have a horizontally active geophone located 0.2m (0.66ft) behind the tip.

To conduct the seismic shear wave test, the penetration of the cone is stopped and the rods are decoupled from the rig. An automatic hammer is triggered to send a shear wave into the soil. The distance from the source to the cone is calculated knowing the total depth of the cone and the horizontal offset distance between the source and the cone. To calculate an interval velocity, a minimum of two tests must be performed at two different depths. The arrival times between the two wave traces are compared to obtain the difference in time (Δt). The difference in depth is calculated (Δd) and velocity can be determined using the simple equation: $v = \Delta d/\Delta t$

Multiple wave traces can be recorded at the same depth to improve quality of the data.

A complete reference on seismic cone penetration tests is presented by Robertson et al. 1986 and Lunne et al. 1997.

A summary the shear wave velocities, arrival times and wave traces are provided with the report.

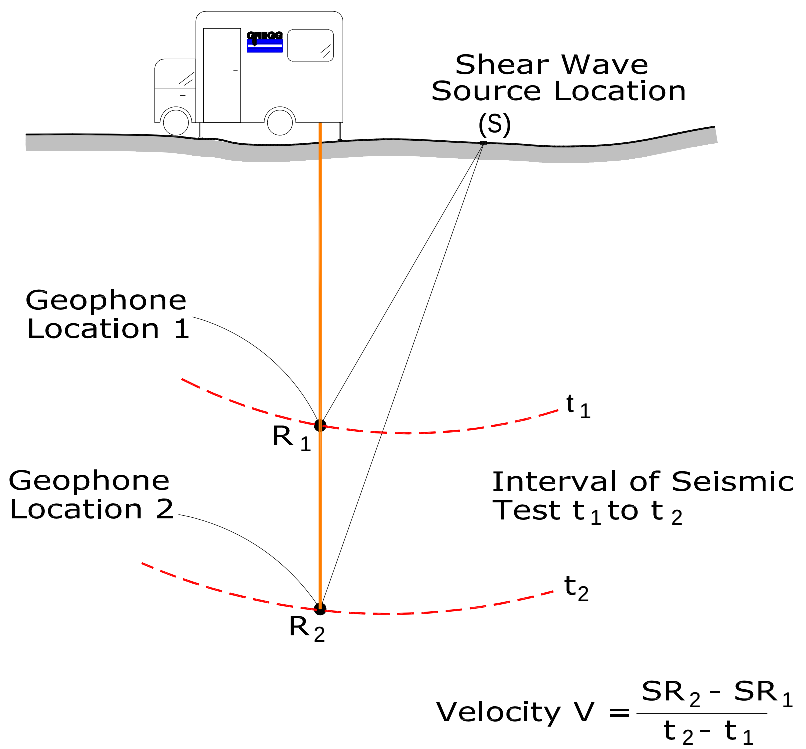


Figure SCPT

Groundwater Sampling

Gregg Drilling & Testing, Inc. conducts groundwater sampling using a sampler as shown in *Figure GWS*. The groundwater sampler has a retrievable stainless steel or disposable PVC screen with steel drop off tip. This allows for samples to be taken at multiple depth intervals within the same sounding location. In areas of slower water recharge, provisions may be made to set temporary PVC well screens during sampling to allow the pushing equipment to advance to the next sample location while the groundwater is allowed to infiltrate.

The groundwater sampler operates by advancing 44.5mm (1¾ inch) hollow push rods with the filter tip in a closed configuration to the base of the desired sampling interval. Once at the desired sample depth, the push rods are retracted; exposing the encased filter screen and allowing groundwater to infiltrate hydrostatically from the formation into the inlet screen. A small diameter bailer (approximately ½ or ¾ inch) is lowered through the push rods into the screen section for sample collection. The number of downhole trips with the bailer and time necessary to complete the sample collection at each depth interval is a function of sampling protocols, volume requirements, and the yield characteristics and storage capacity of the formation. Upon completion of sample collection, the push rods and sampler, with the exception of the PVC screen and steel drop off tip are retrieved to the ground surface, decontaminated and prepared for the next sampling event.

For a detailed reference on direct push groundwater sampling, refer to Zemo et. al., 1992.

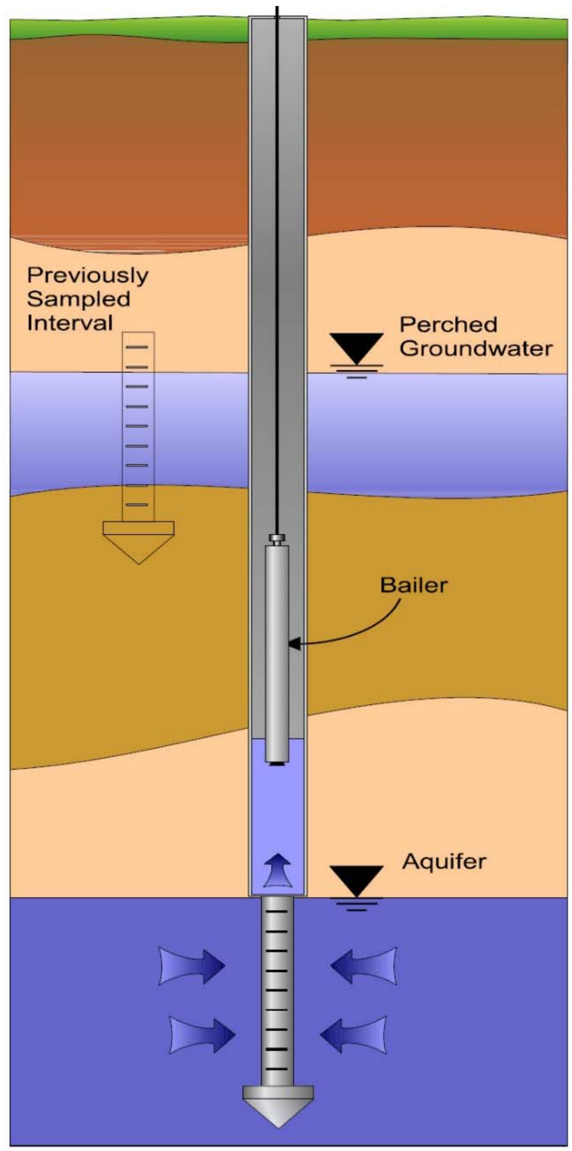


Figure GWS

Soil Sampling

Gregg Drilling & Testing, Inc. uses a piston-type push-in sampler to obtain small soil samples without generating any soil cuttings, *Figure SS*. Two different types of samplers (12 and 18 inch) are used depending on the soil type and density. The soil sampler is initially pushed in a "closed" position to the desired sampling interval using the CPT pushing equipment. Keeping the sampler closed minimizes the potential of cross contamination. The inner tip of the sampler is then retracted leaving a hollow soil sampler with inner 1¼” diameter sample tubes. The hollow sampler is then pushed in a locked "open" position to collect a soil sample. The filled sampler and push rods are then retrieved to the ground surface. Because the soil enters the sampler at a constant rate, the opportunity for 100% recovery is increased. For environmental analysis, the soil sample tube ends are sealed with Teflon and plastic caps. Often, a longer "split tube" can be used for geotechnical sampling.

For a detailed reference on direct push soil sampling, refer to Robertson et al, 1998.

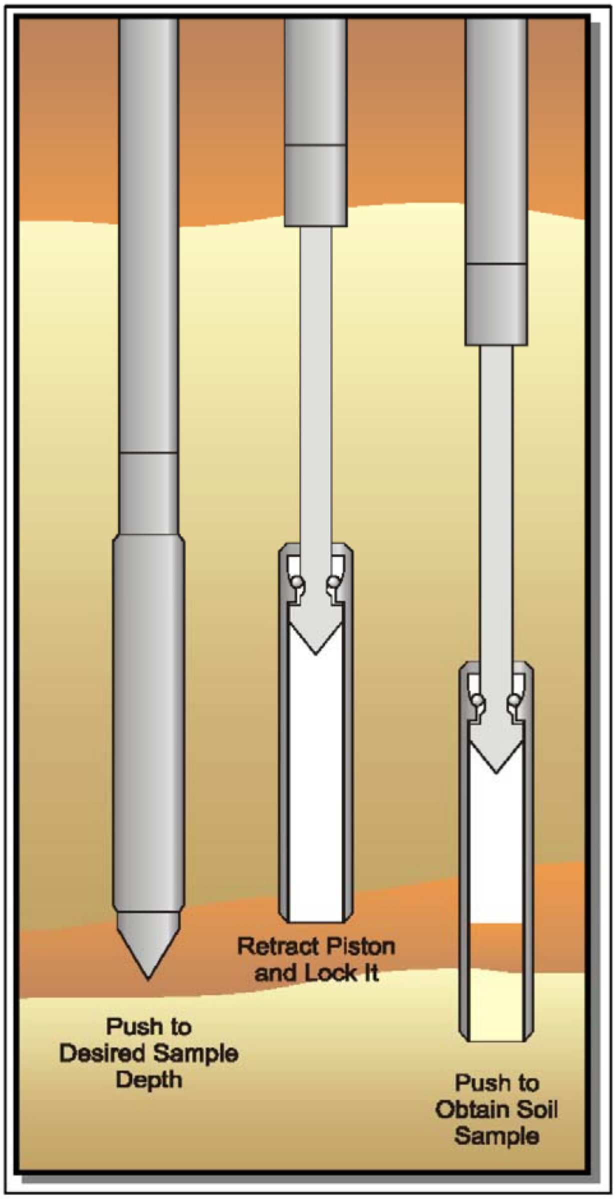


Figure SS

Gregg Drilling, LLC
www.greggdrilling.com

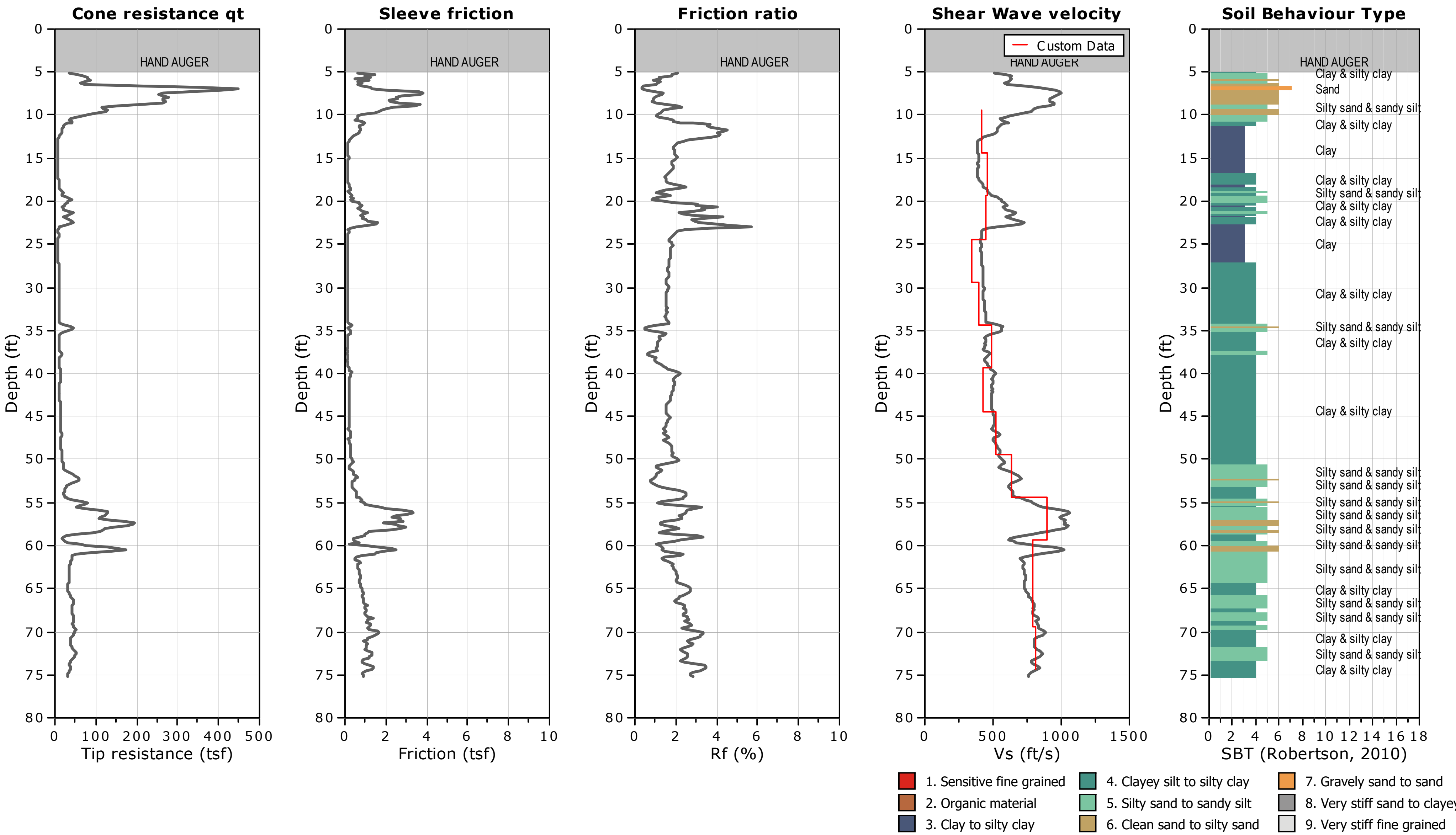
CPT: SCPT-07

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

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



Gregg Drilling, LLC
www.greggdrilling.com

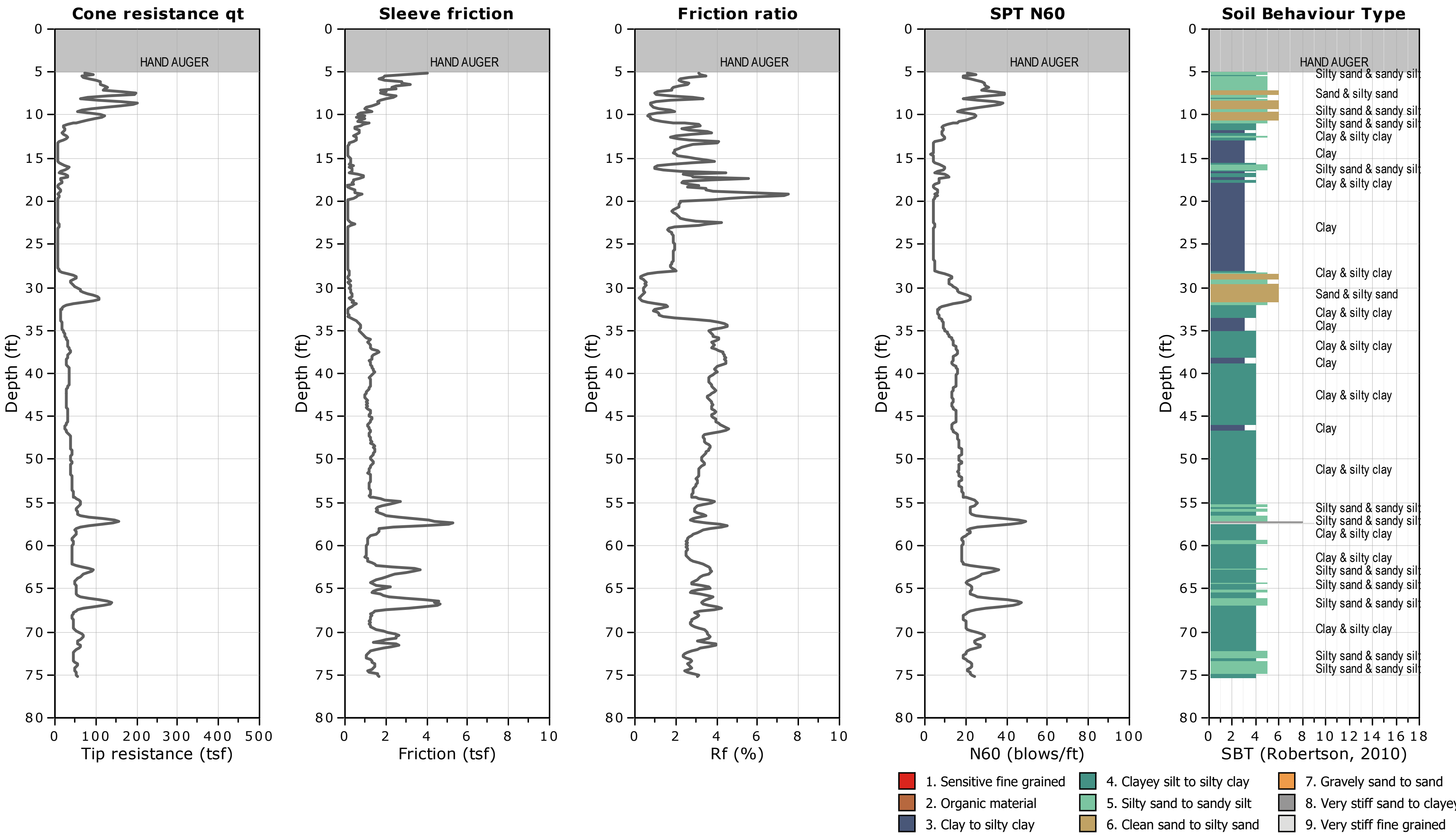
CPT: CPT-04

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

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



Gregg Drilling, LLC
www.greggdrilling.com

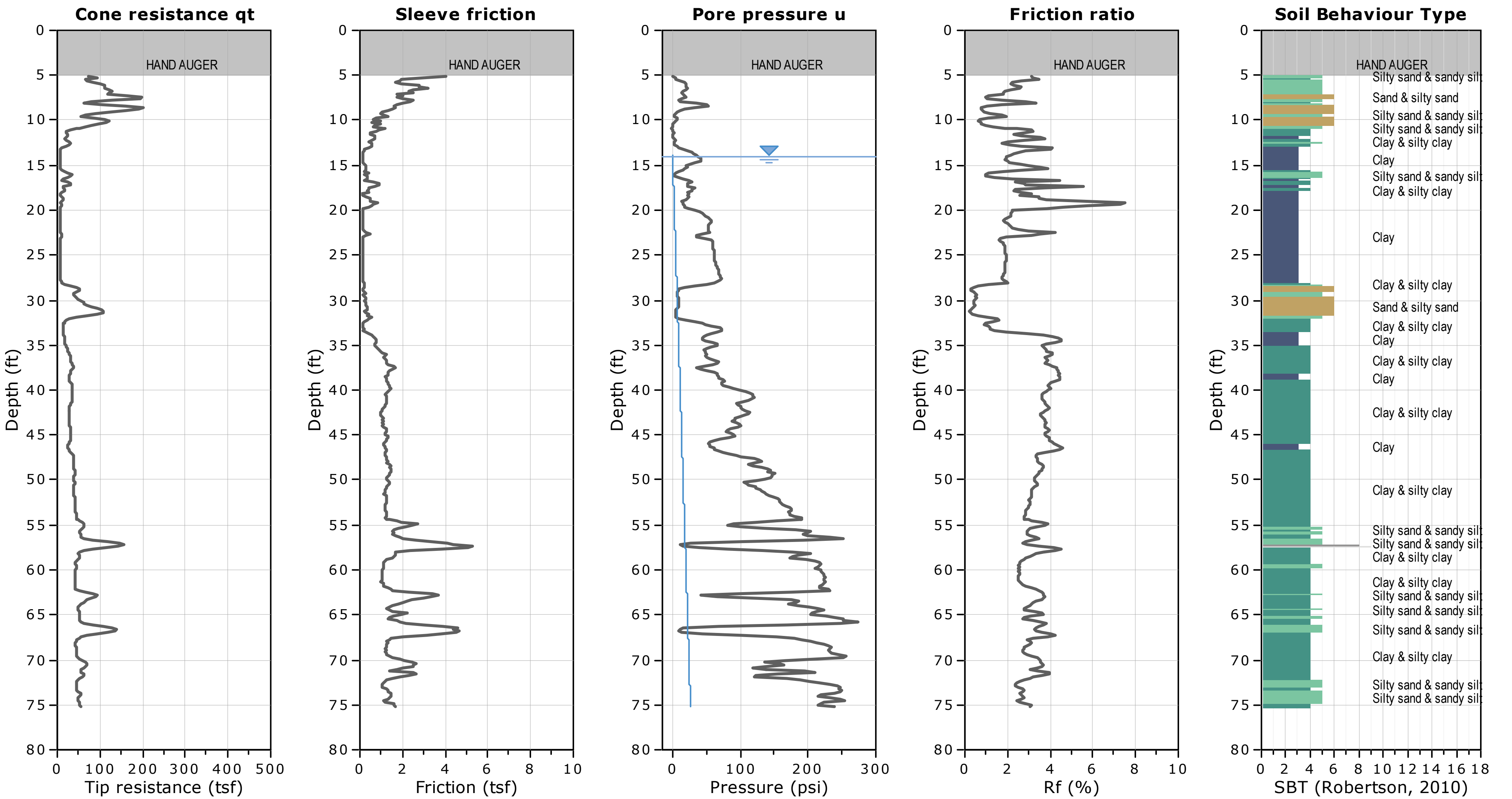
CPT: CPT-04

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

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



1. Sensitive fine grained
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

Gregg Drilling, LLC
www.greggdrilling.com

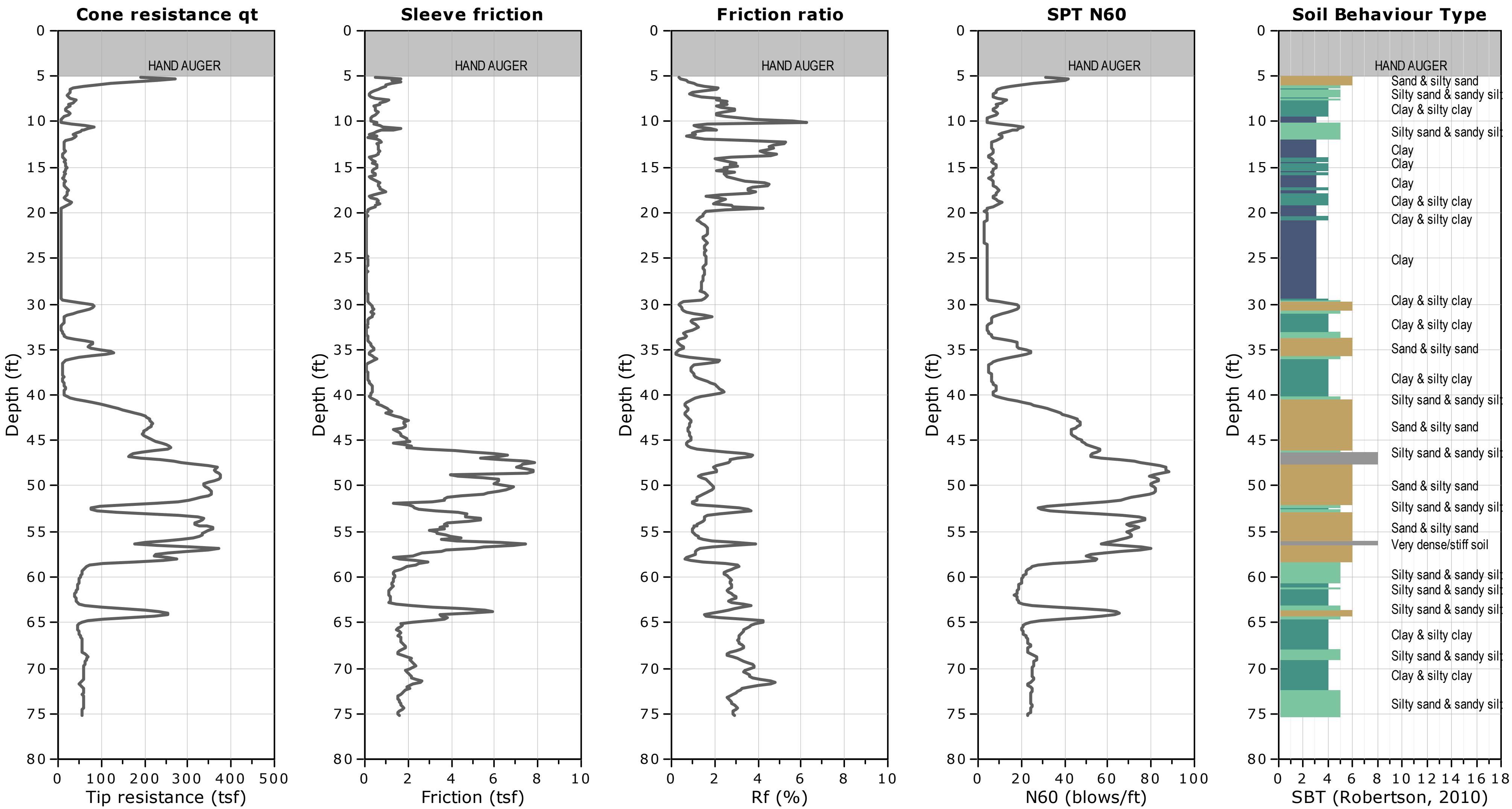
CPT: CPT-05

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

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



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

Gregg Drilling, LLC
www.greggdrilling.com

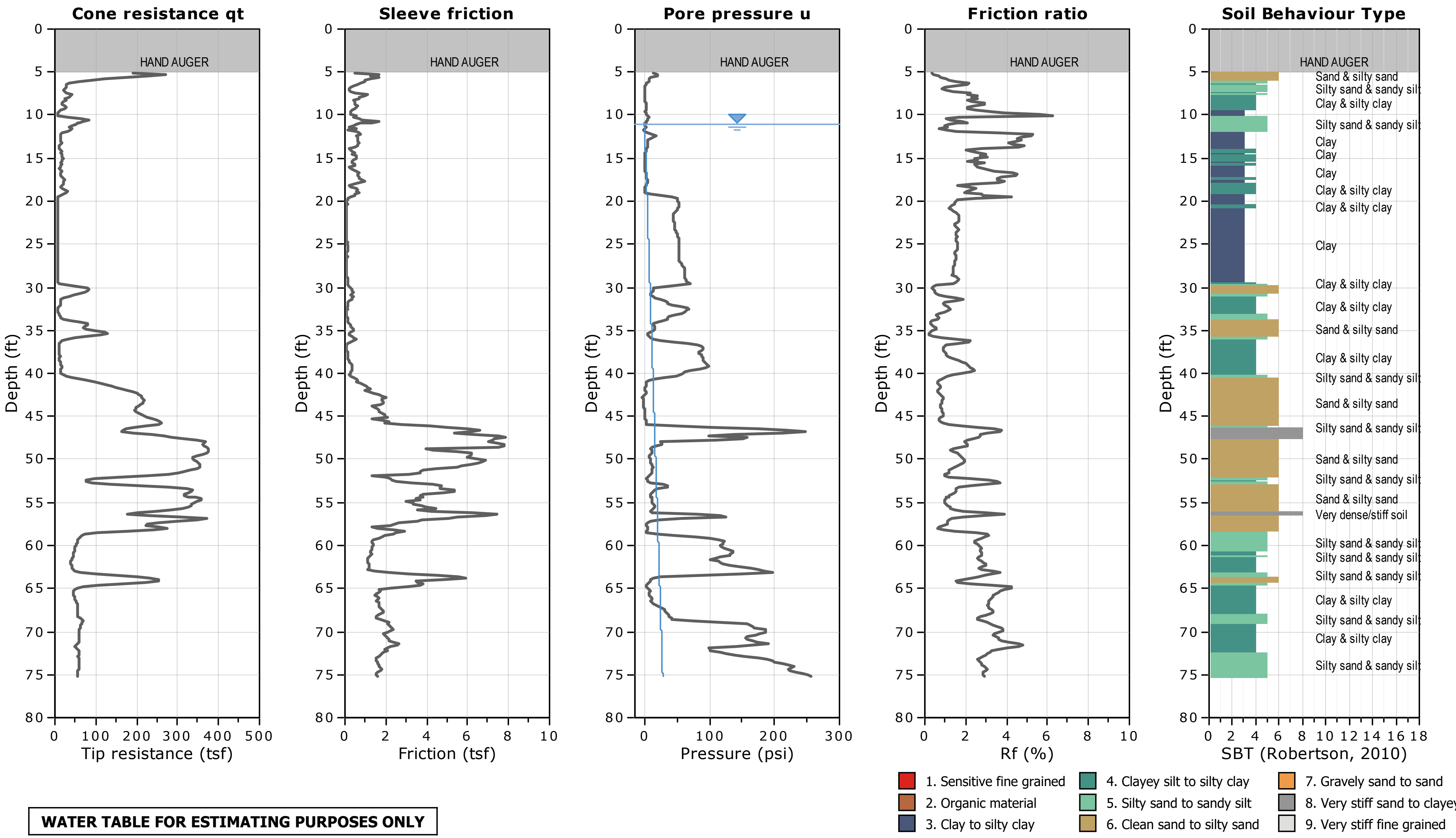
CPT: CPT-05

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

Gregg Drilling, LLC
www.greggdrilling.com

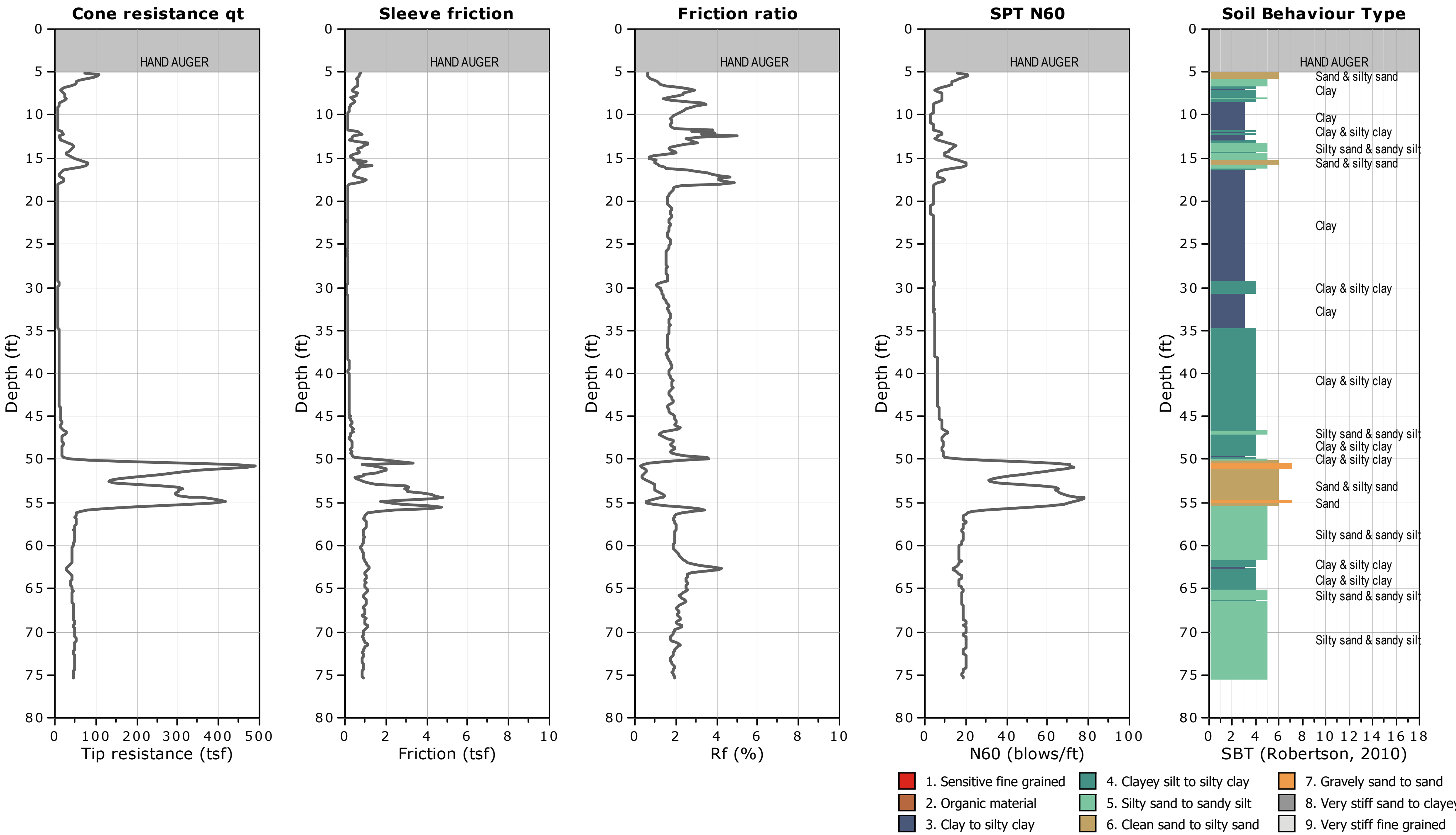
CPT: CPT-06

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

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



Gregg Drilling, LLC
www.greggdrilling.com

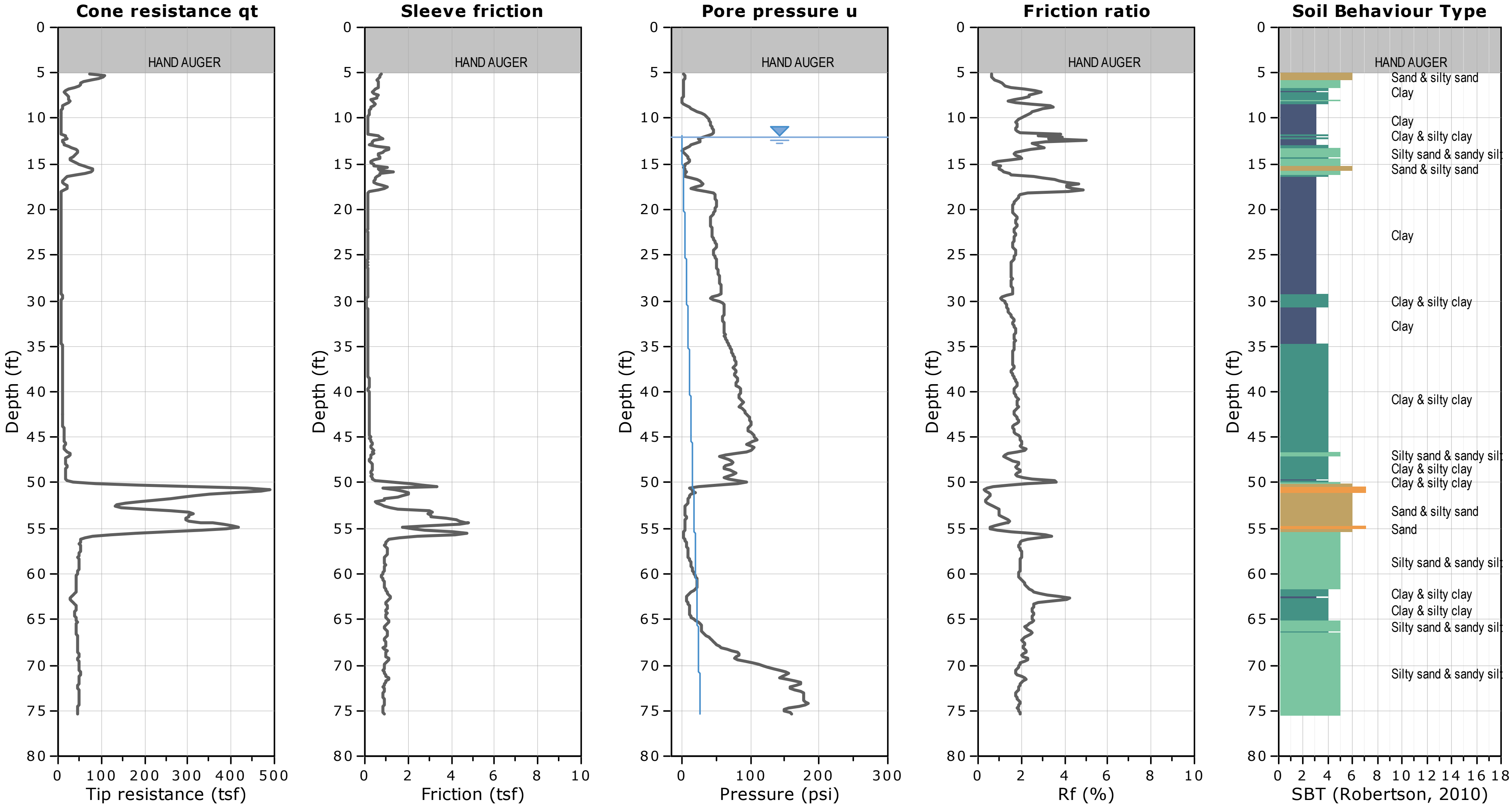
CPT: CPT-06

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

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



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

Gregg Drilling, LLC
www.greggdrilling.com

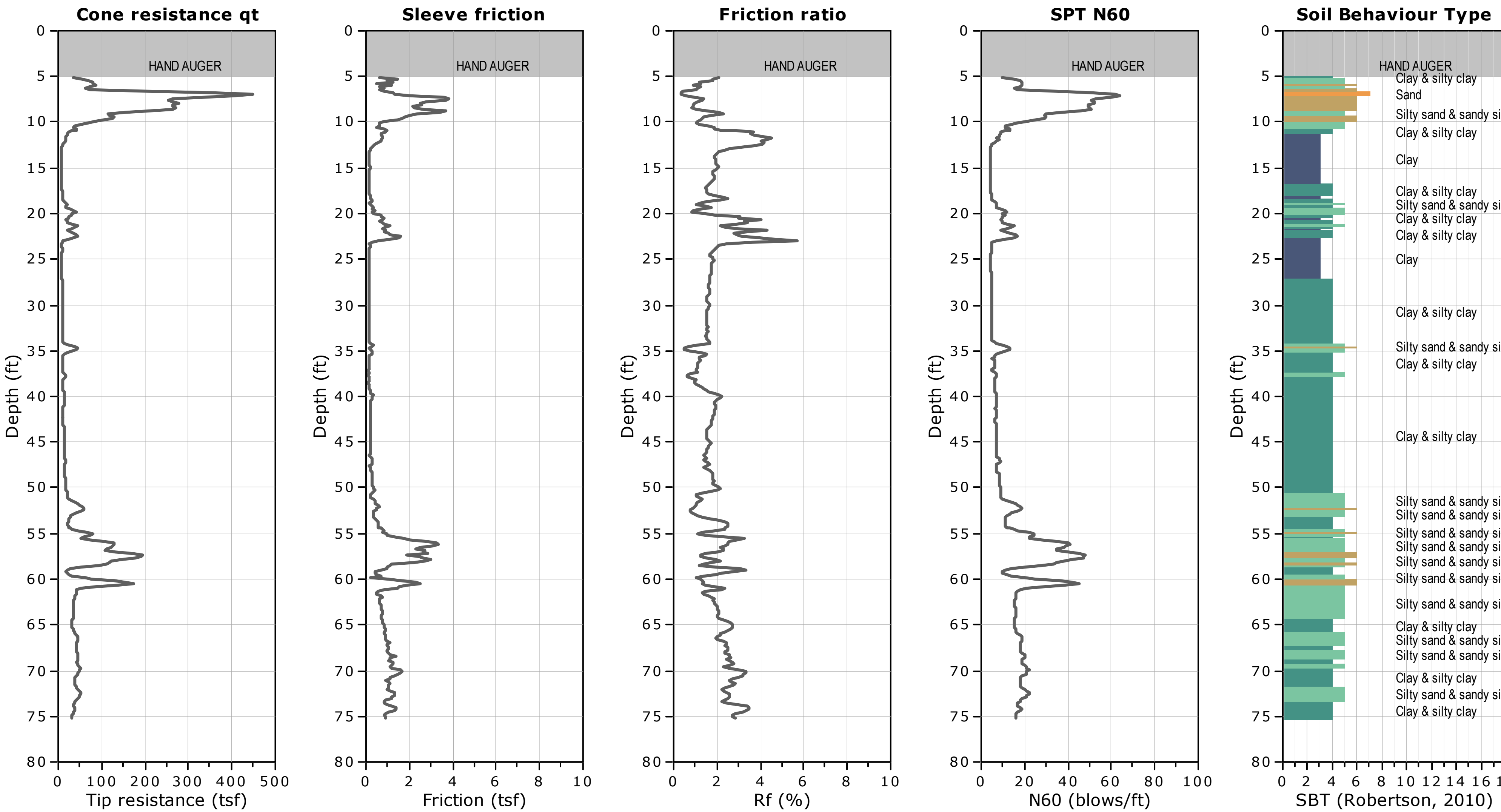
CPT: SCPT-07

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

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



1. Sensitive fine grained	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

Gregg Drilling, LLC
www.greggdrilling.com

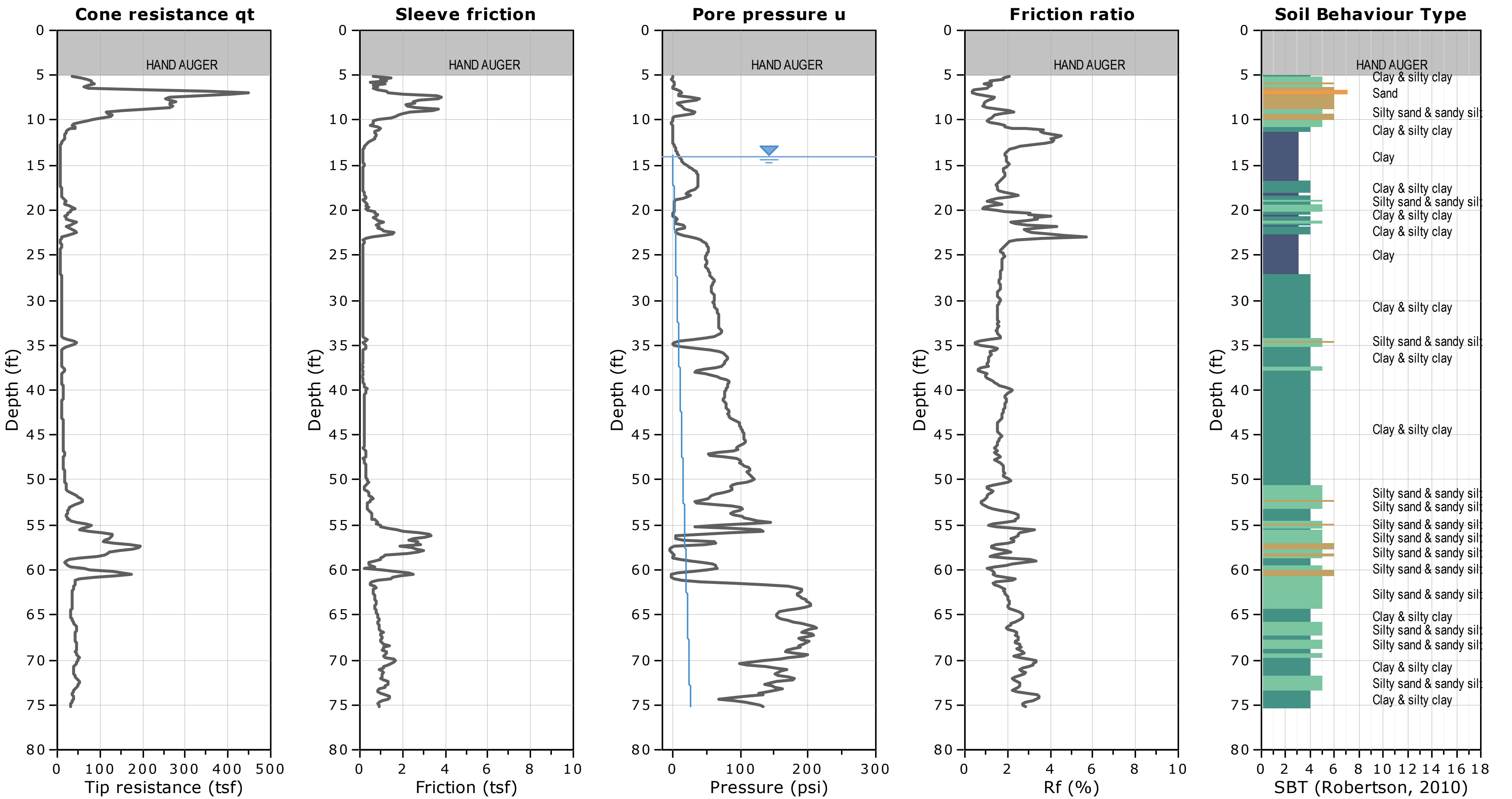
CPT: SCPT-07

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

Gregg Drilling, LLC
www.greggdrilling.com

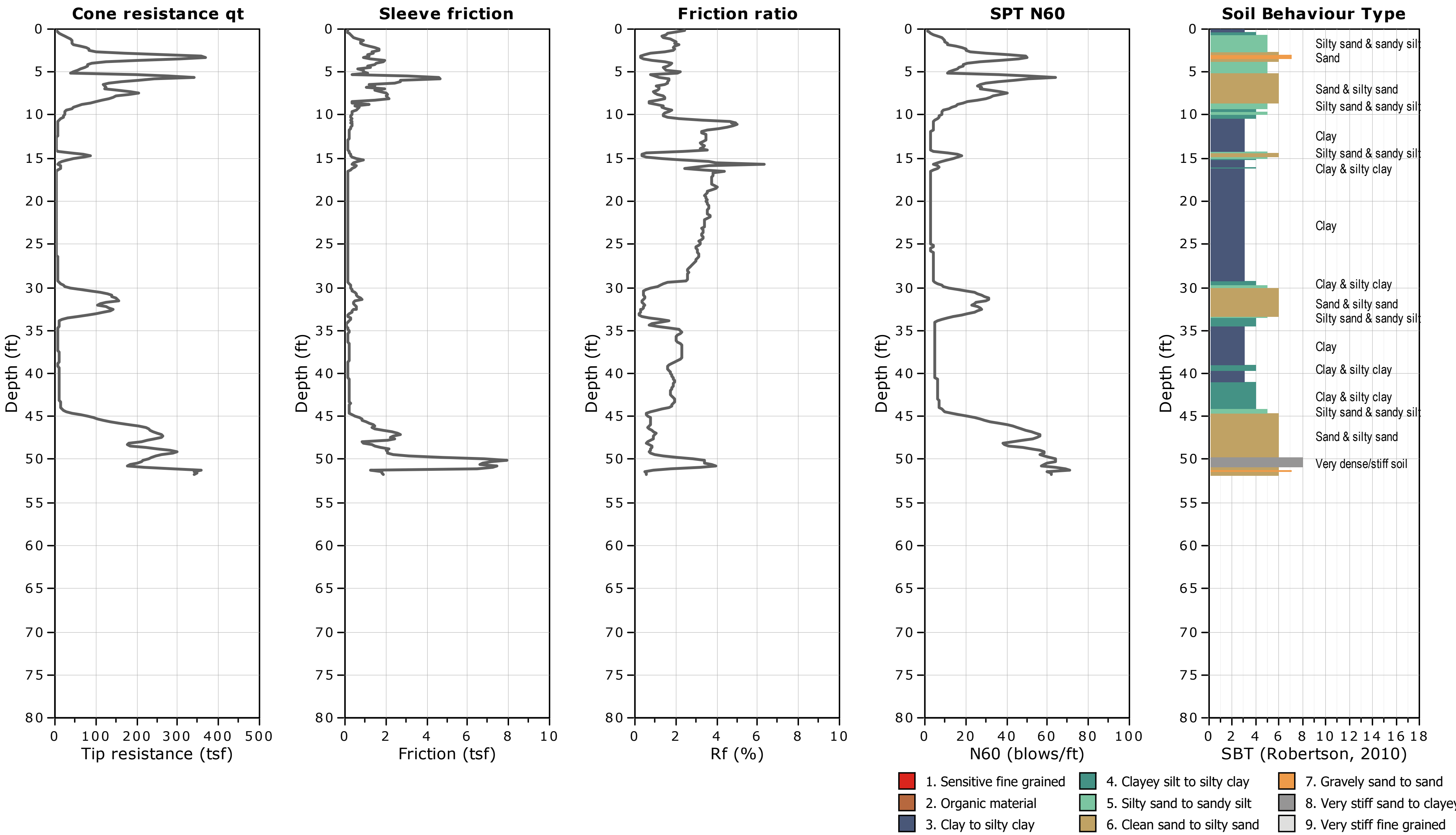
CPT: CPT-08

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

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



Gregg Drilling, LLC
www.greggdrilling.com

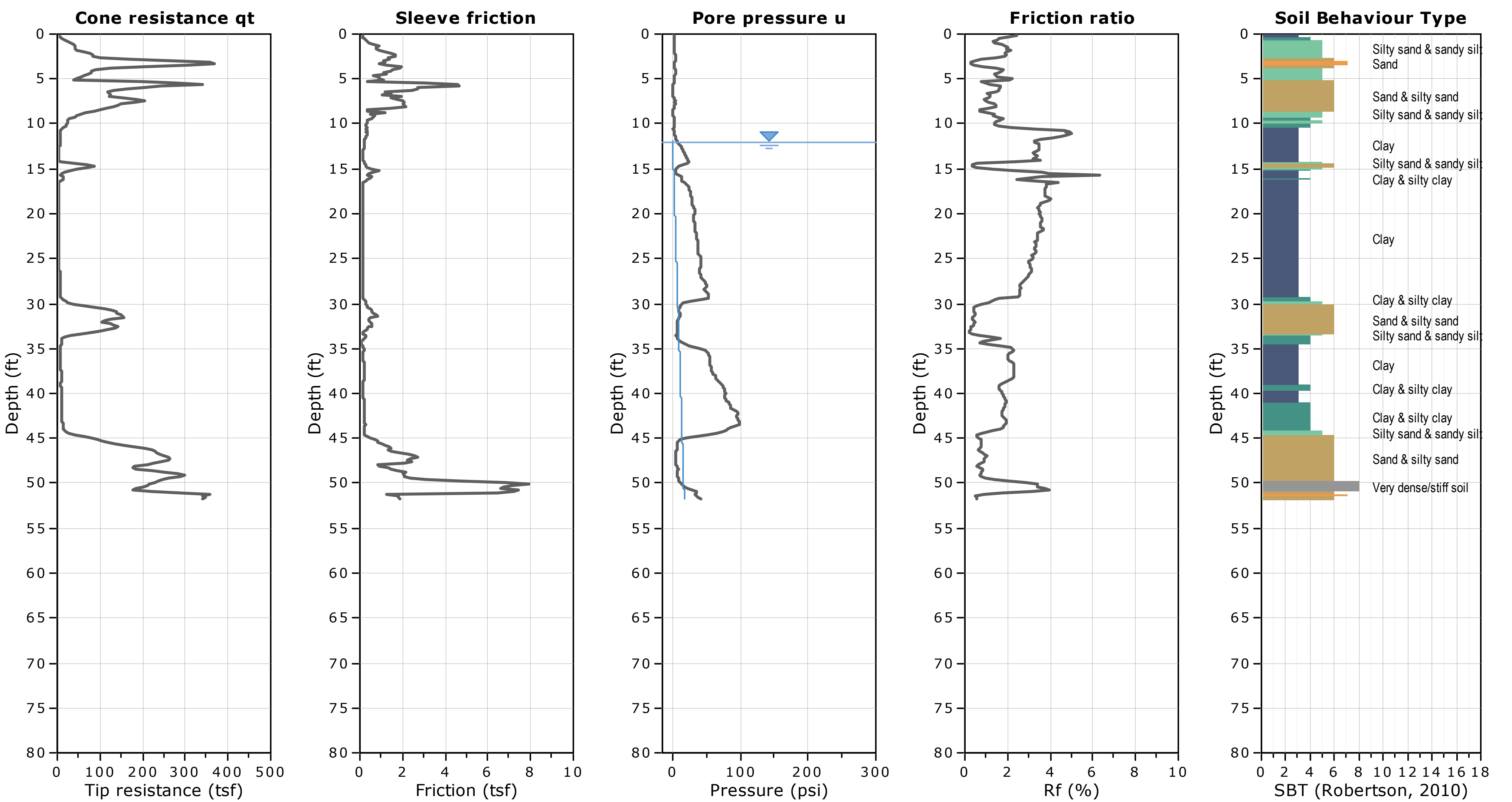
CPT: CPT-08

CLIENT: FUGRO

SITE: LANEY COLLEGE, OAKLAND, CA

FIELD REP: REZA RAHIMNEJAD

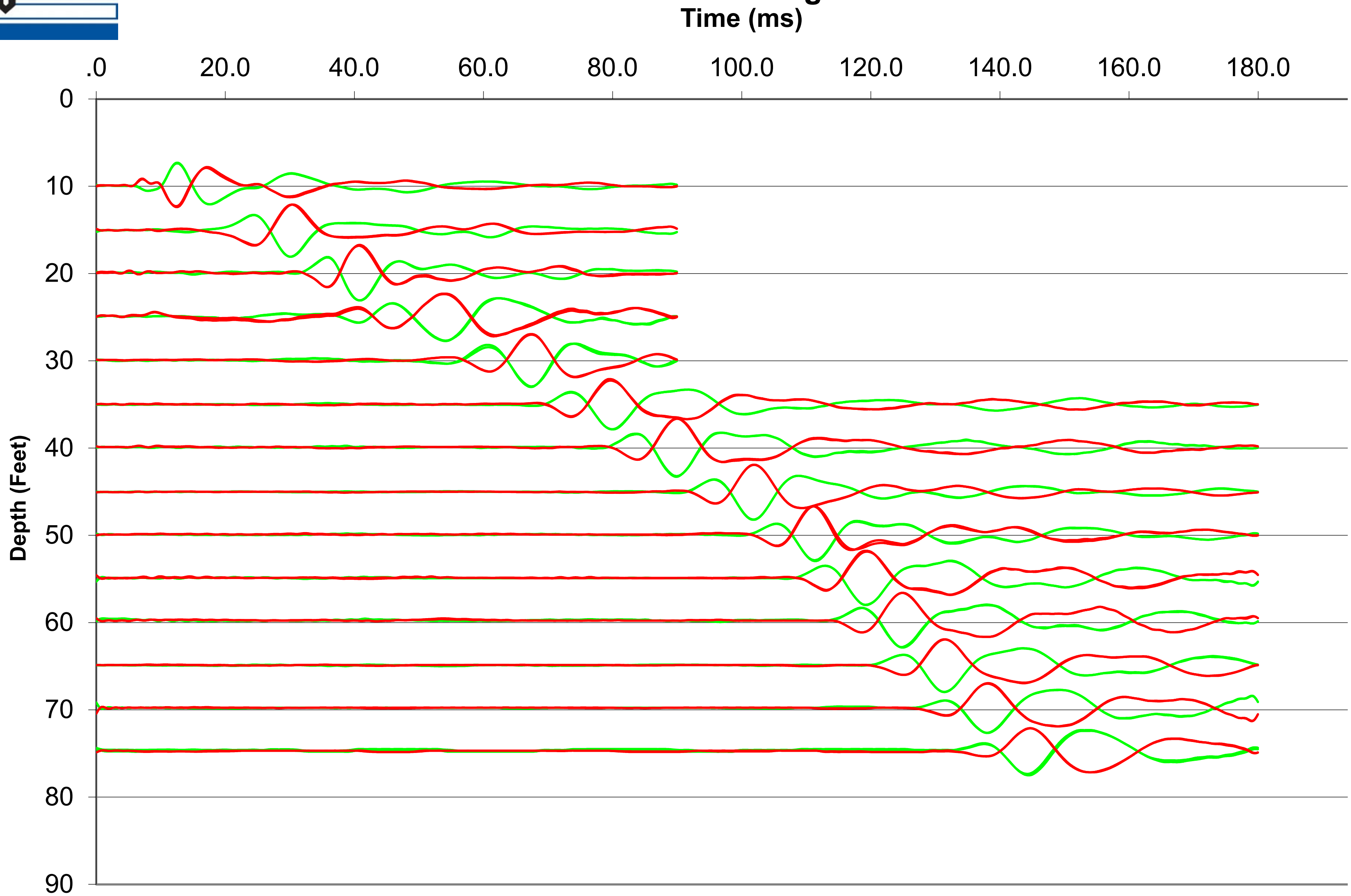
Total depth: 51.67 ft, Date: 1/2/2020



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

Waveforms for Sounding SCPT-07



Shear Wave Velocity Calculations

Laney College

SCPT-07

Geophone Offset: 0.66 Feet
Source Offset: 1.67 Feet

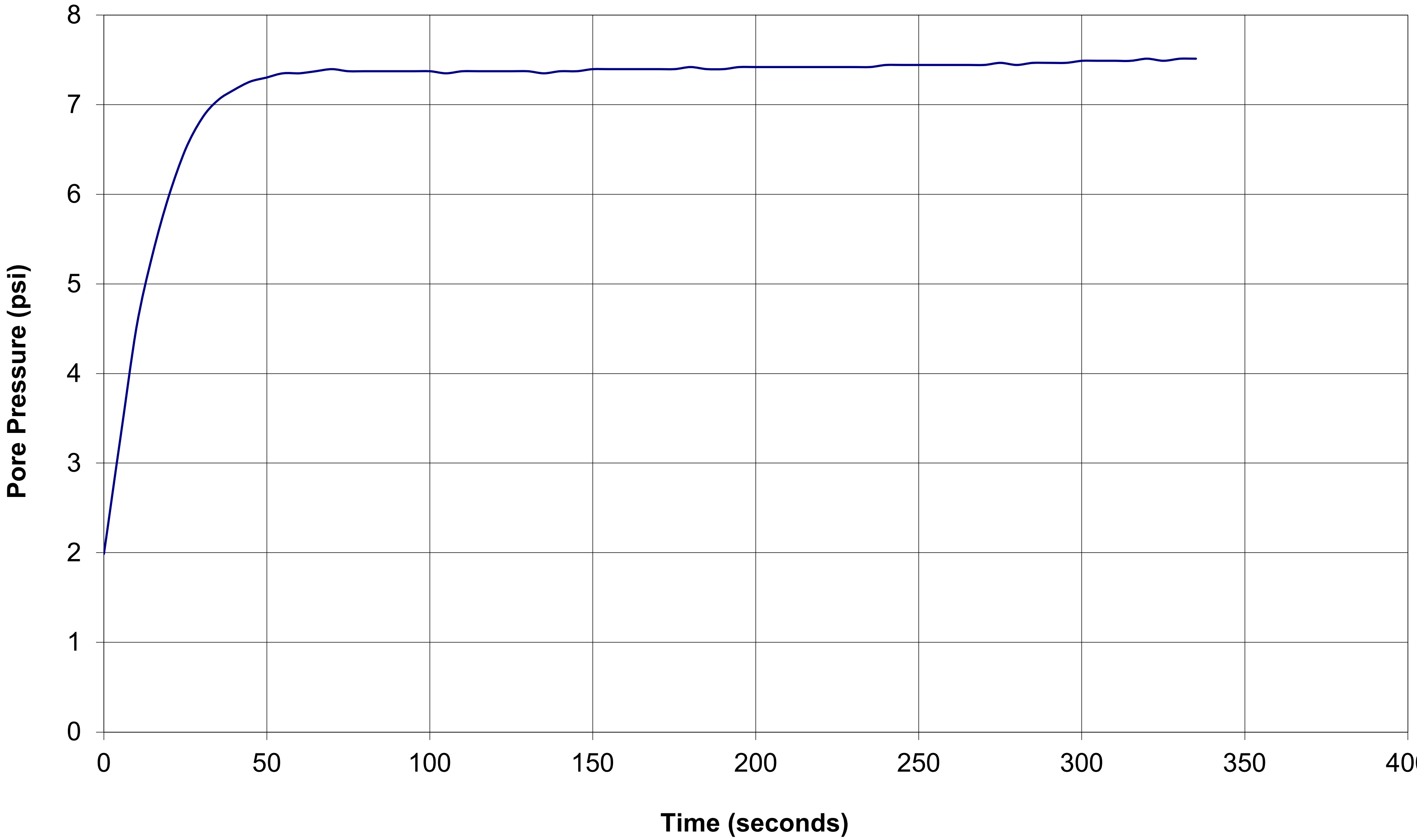
01/03/20

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

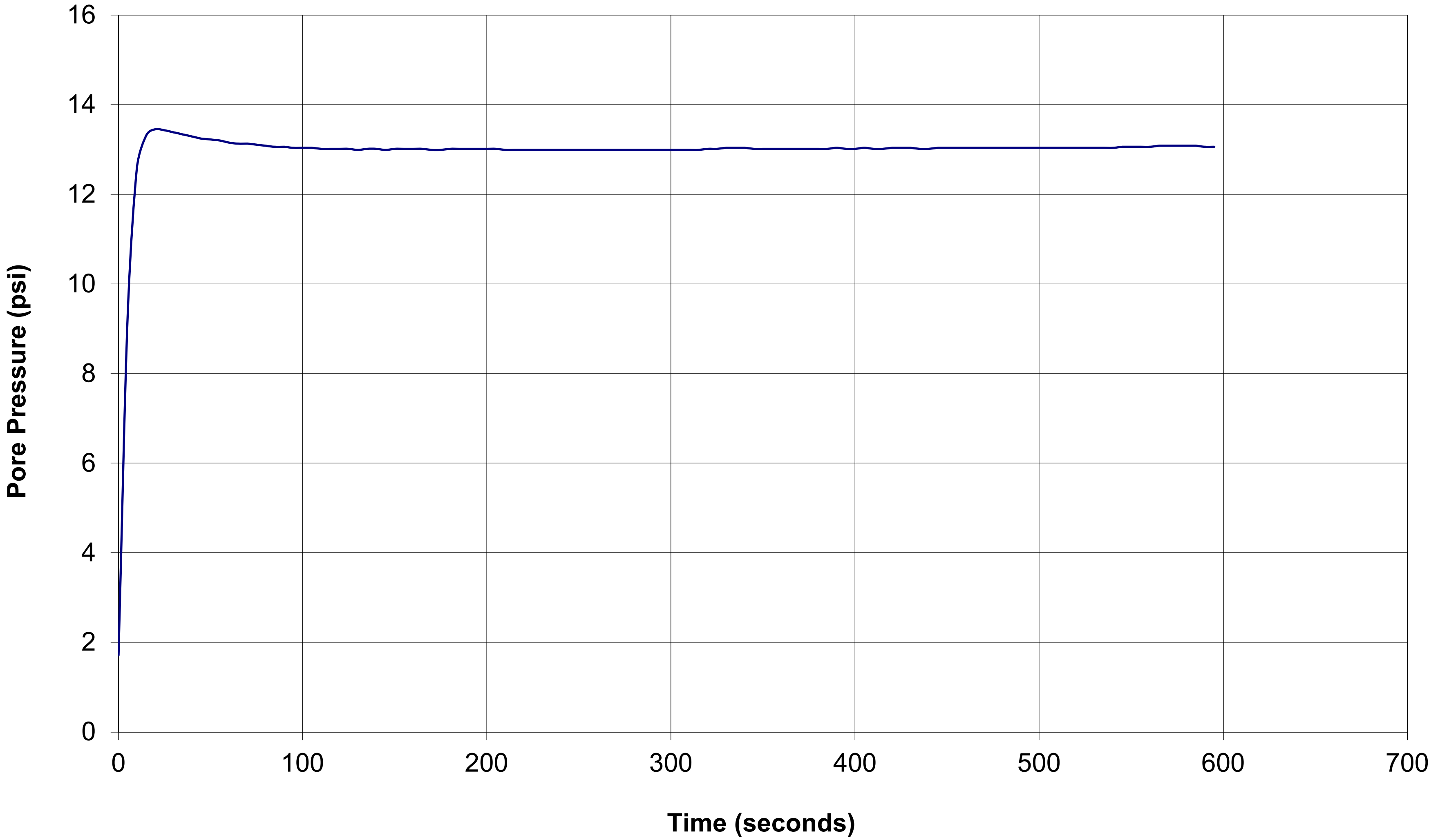
Sounding:	CPT-04
Depth (ft):	31.33
Site:	Laney College
Engineer:	Reza Rahimnejad



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

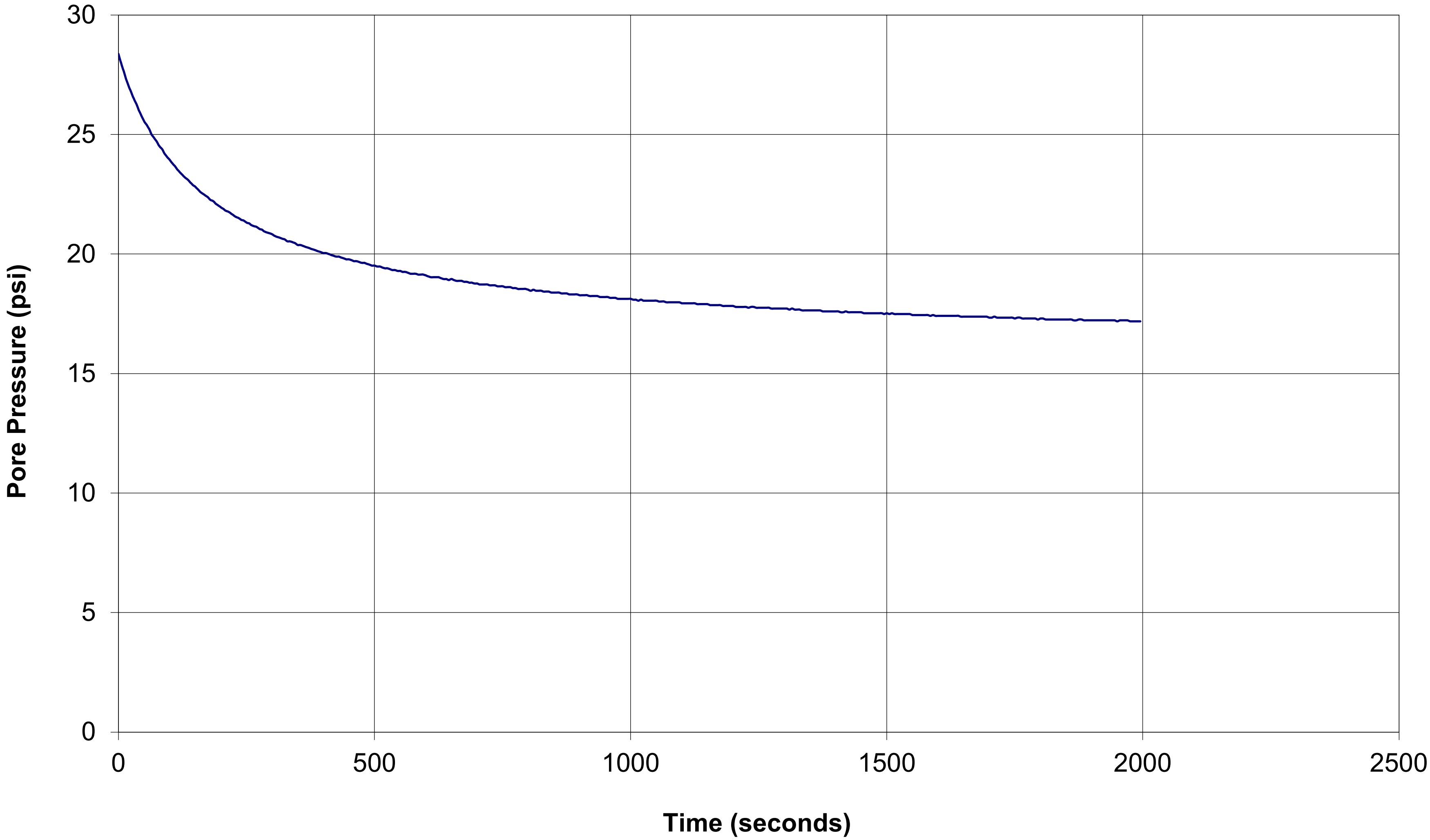
Sounding: CPT-05
Depth (ft): 41.17
Site: Laney College
Engineer: Reza Rahimnejad



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

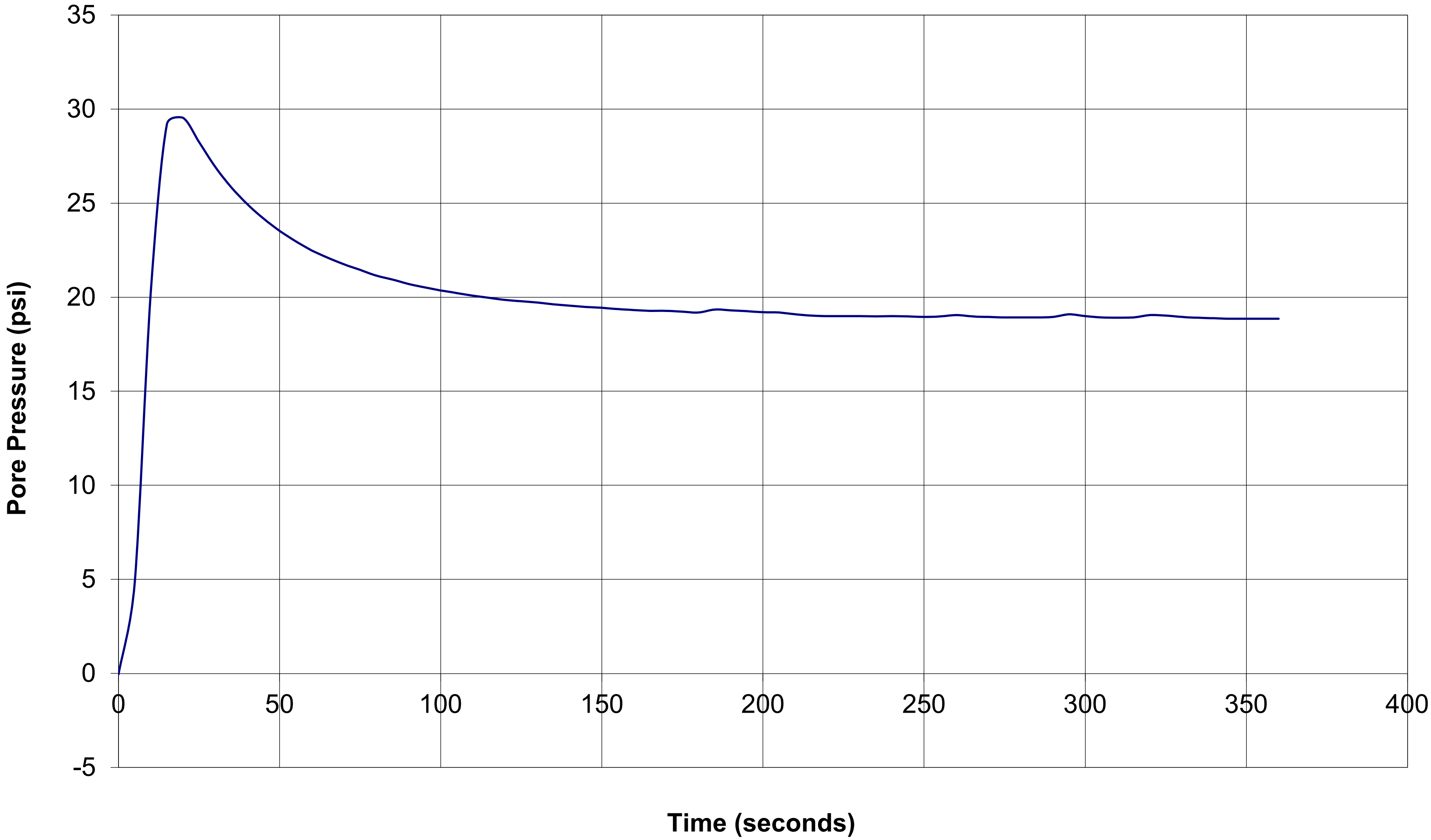
Sounding: CPT-08
Depth (ft): 51.67
Site: Laney College
Engineer: Reza Rahimnejad



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

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



Appendix B

Laboratory Testing Program

Peralta Community College District
Project No. 04.72190021

FUGRO

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

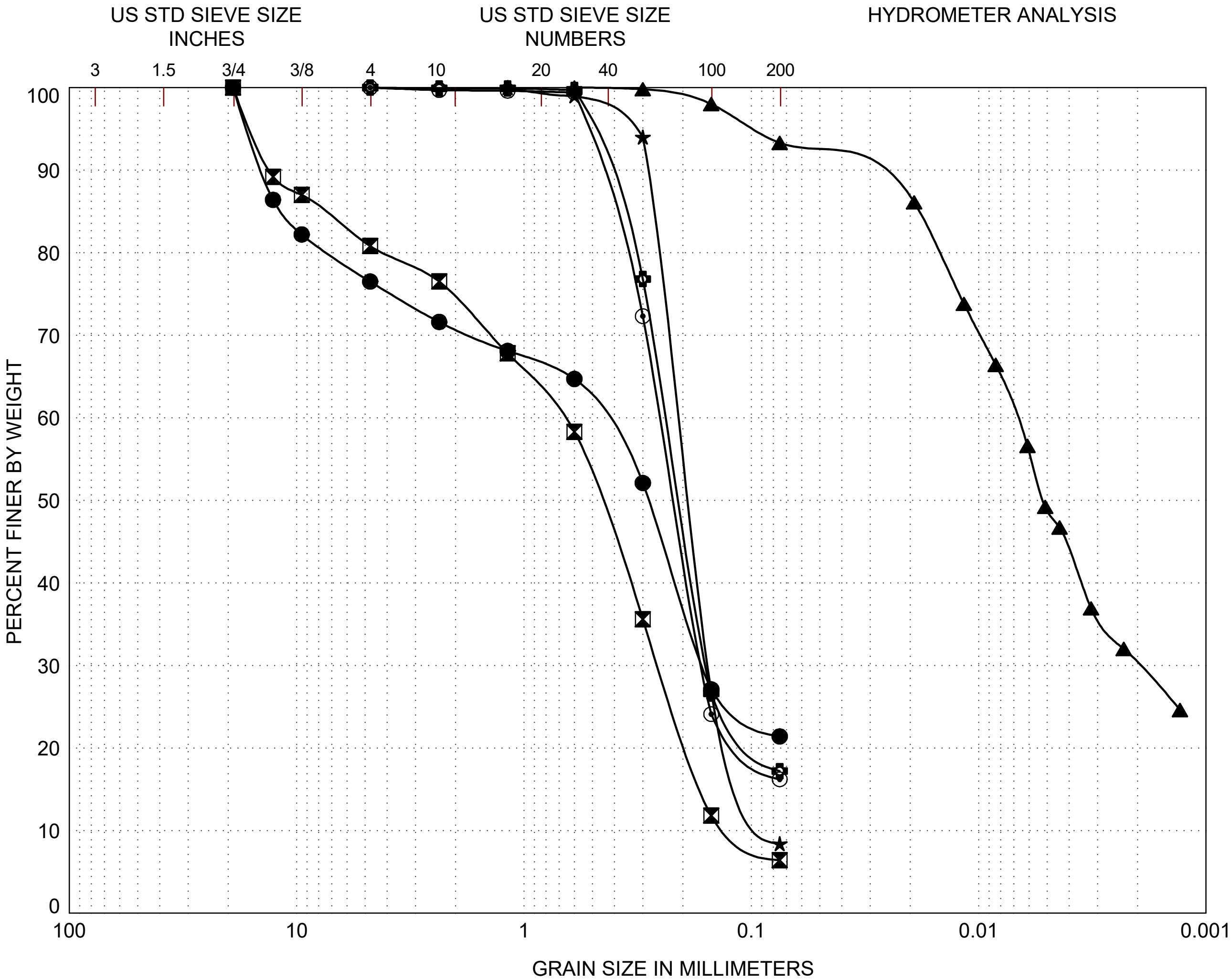
Test Listing Abbreviations
M = Moisture Content
T = Total & Dry Unit Weight
S = Sieve Analysis
FC = % Passing #200 Sieve
H = Hydrometer Analysis
A = Atterberg Limits
P = Compaction Test
D = Direct Shear Test
C = Consolidation Test
Co = Corrosivity Tests
CU = CU Triaxial
U = UU Triaxial
R = R-Value
SE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

SUMMARY OF LABORATORY TEST RESULTS

Laney College Library Learning Resource Center
Oakland, California

PLATE B-1

Peralta Community College District
Project No. 04.72190021

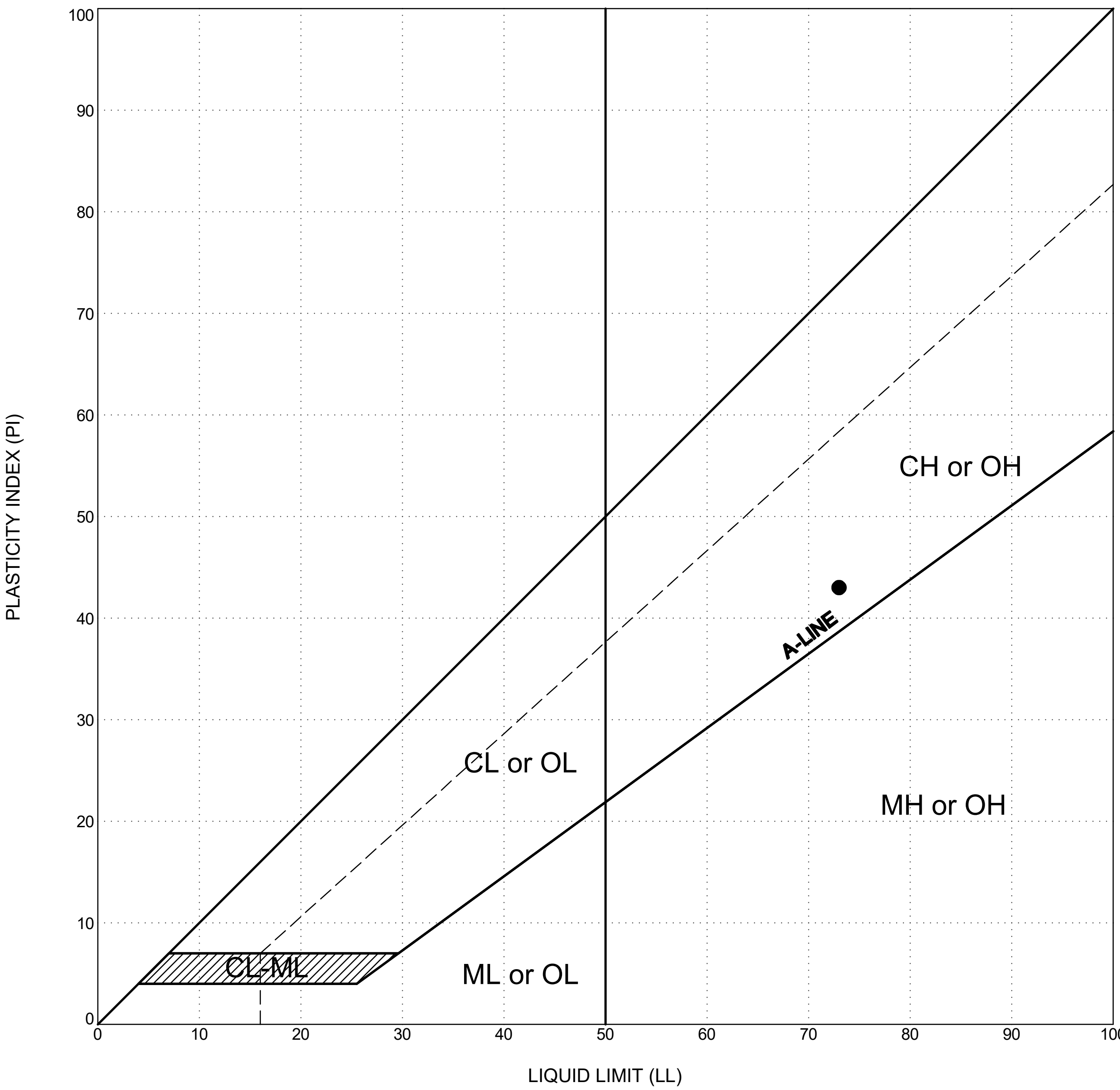


GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

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

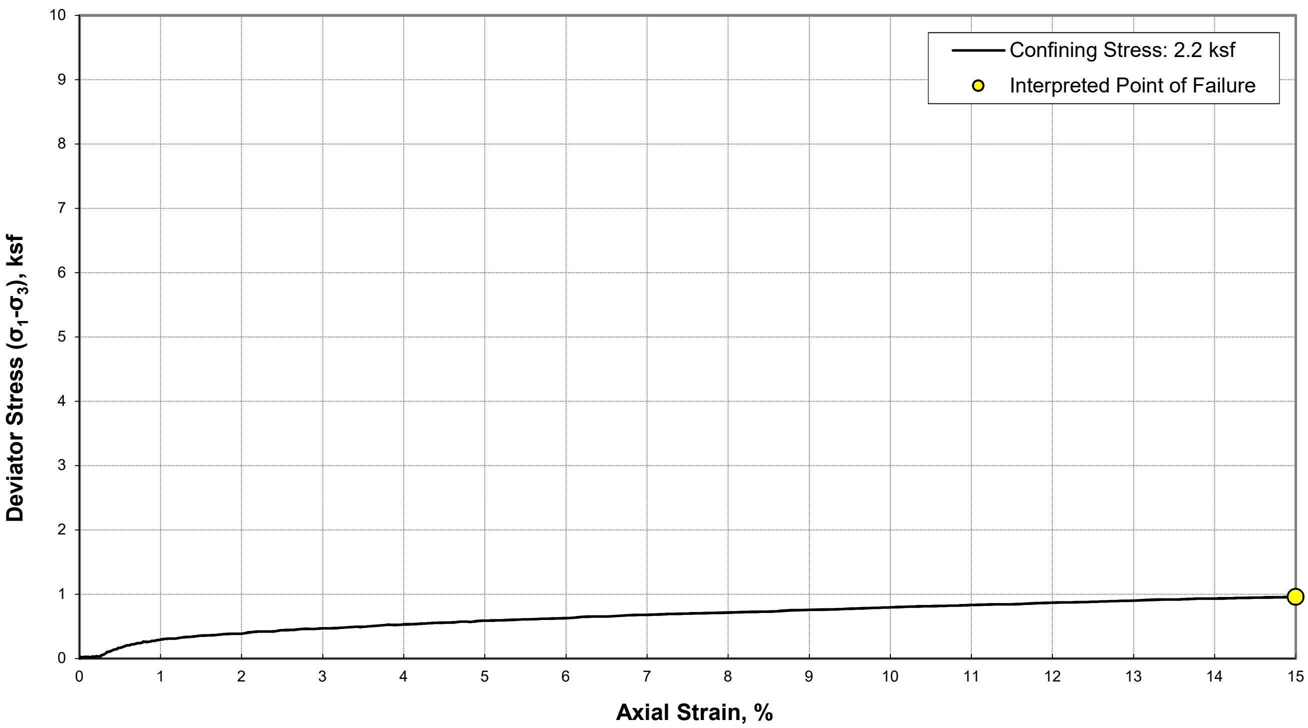


ATTERBERG LIMITS TEST RESULTS

LEGEND location	depth, ft	CLASSIFICATION	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



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

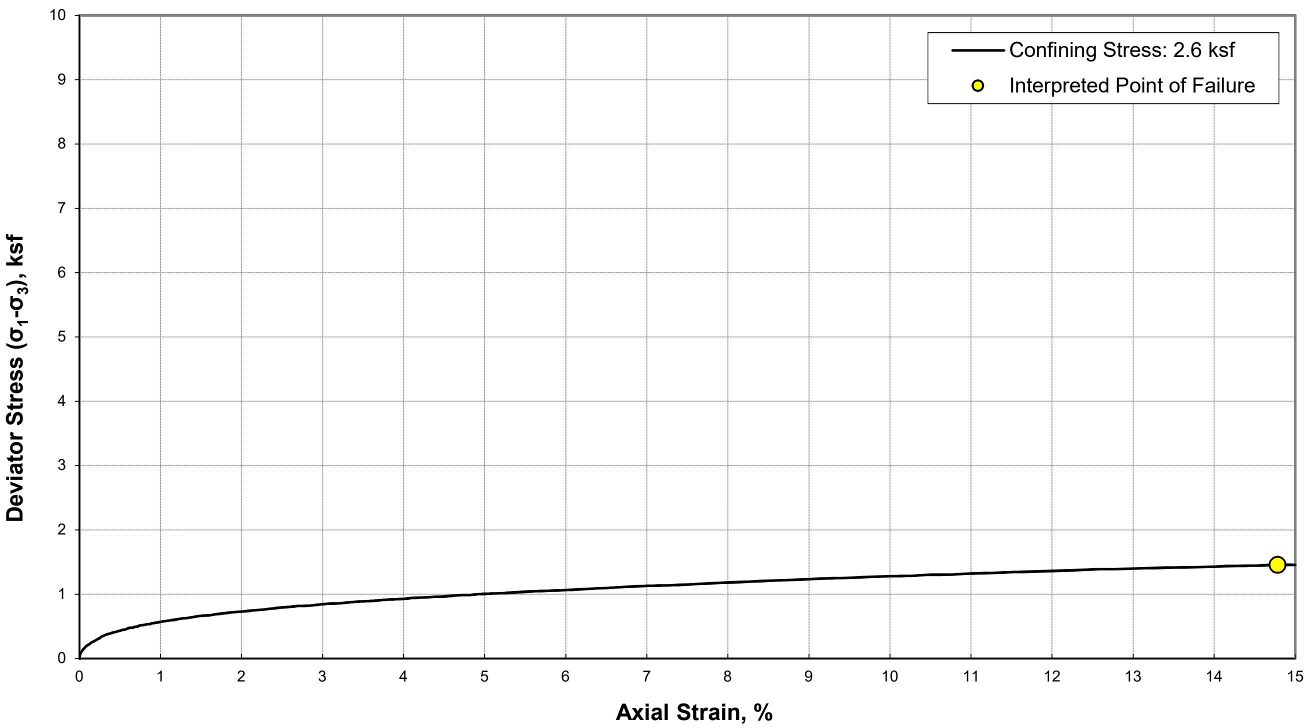
REMARKS

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	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	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

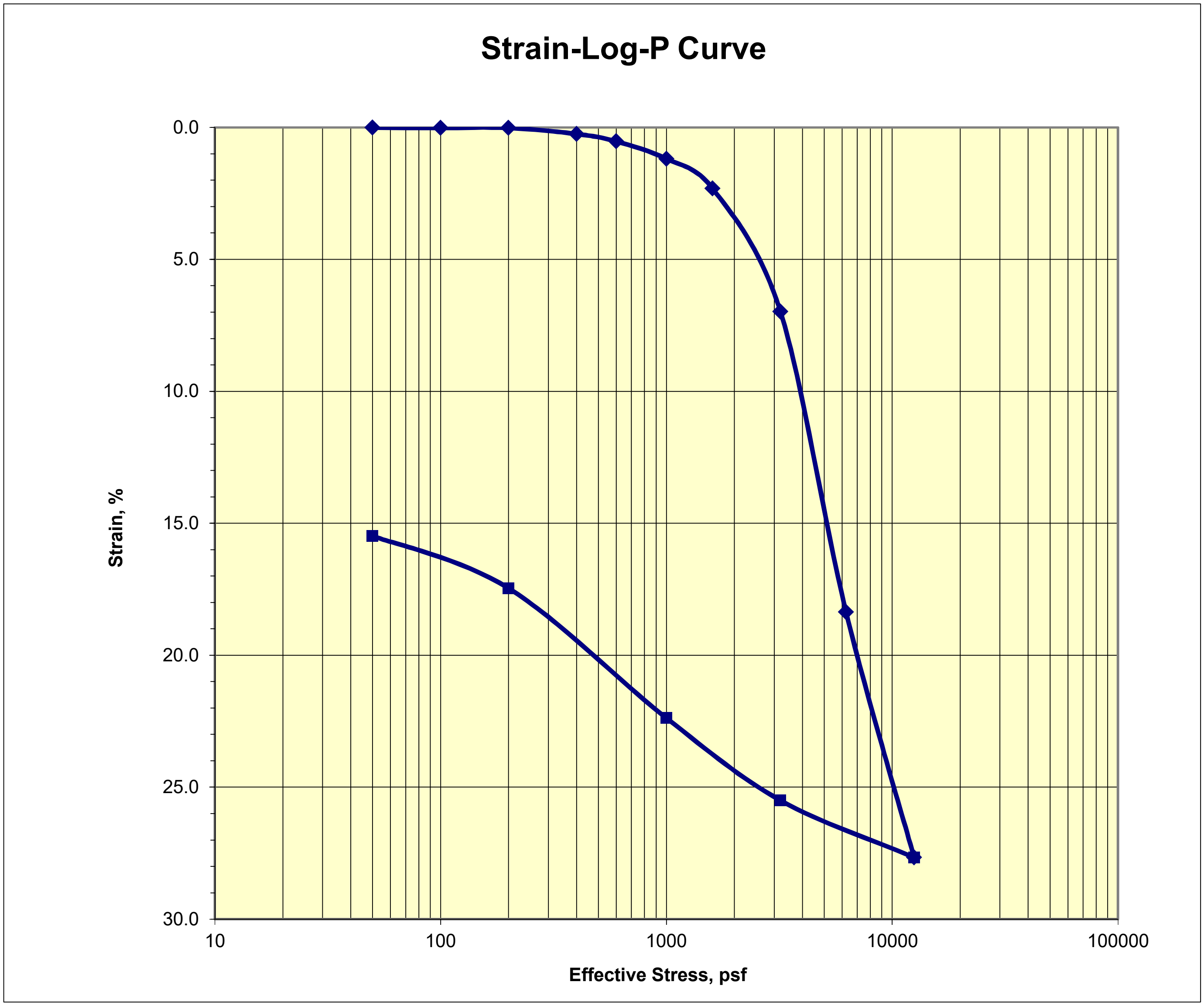
Sample No.	Depth (ft)	Sample Description	Water Content (%)	Ash Content (%)	Organic Content (%)
B-01	16	Poorly Graded SAND with SILT (SP - SM)	25.9	95.0	5.0
B-01	17	Organic CLAY with SAND (OH)	82.5	78.8	21.2
B-01	21	Fat CLAY (CH)	53.4	93.4	6.6

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.

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

JDH/jdl
Enclosure

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

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

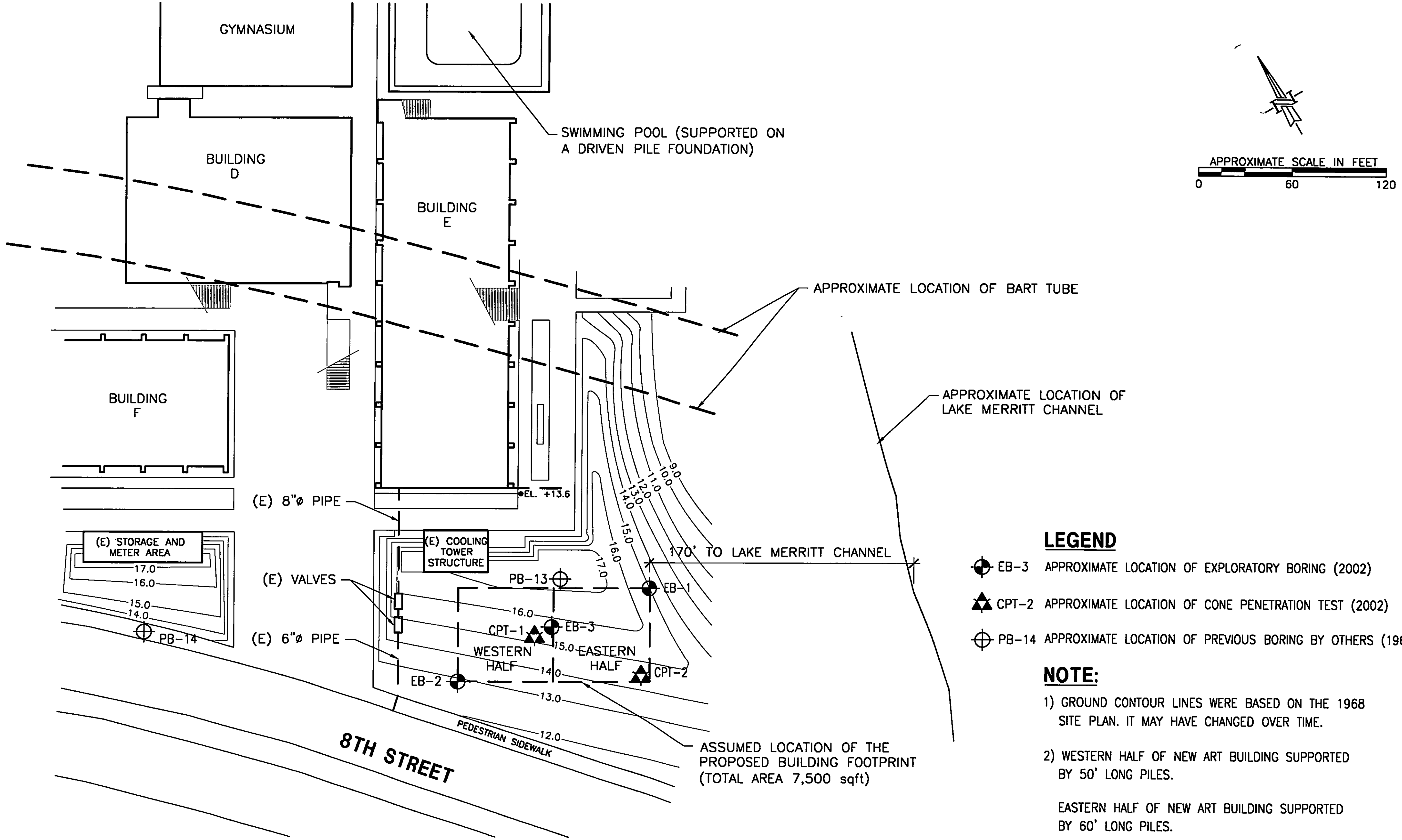
Quality Control Summary - All laboratory quality control parameters were found to be within established limits

Appendix 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.

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

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

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
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
			20						
strong hydrocarbon odor, with high amount of wood fragment, metal pieces, and other debris at 20 feet									
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				

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		61	22	105	2.3	% of Passing #200 Sieve = 43 PP = 4.0
			70		67				PP = 4.5
			75		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.

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	**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
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	(FEET)						TESTS
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

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-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		30		57	17	114	9.1	
SAND (SP/SM), light brown, medium- to coarse-grained, trace gravel (subangular to subrounded) and silt, wet	Dense		35		37				
			40		32				% of Passing #200 Sieve = 19 between 29 feet to 59 feet
			45		32	▼			
					37				

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-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
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).

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-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 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
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	X	33	15	119	3.2	% of Passing #200 Sieve = 42
hard drilling due to encountering concrete or brick chunk			10	X	50/4"				No Recovery
BAY MUD: CLAY (CH), black, mottled shades of blue-gray, silty, mild hydrocarbon odor, moist	Very Soft		15						
			20	X	1	▽			PP < 0.5
			25						

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

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-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).

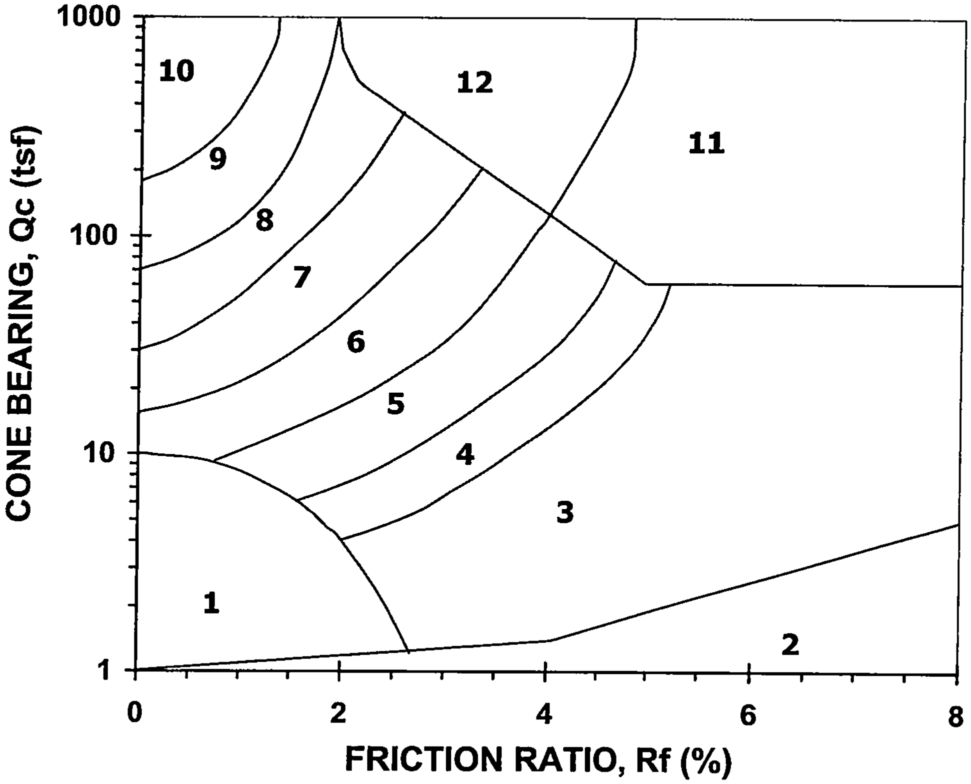
FUGRO
425 Roland Way
Oakland, CA 94621

EXPLORATORY BORING LOG

NEW ART BUILDING AT LANEY COLLEGE
Oakland, CA

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

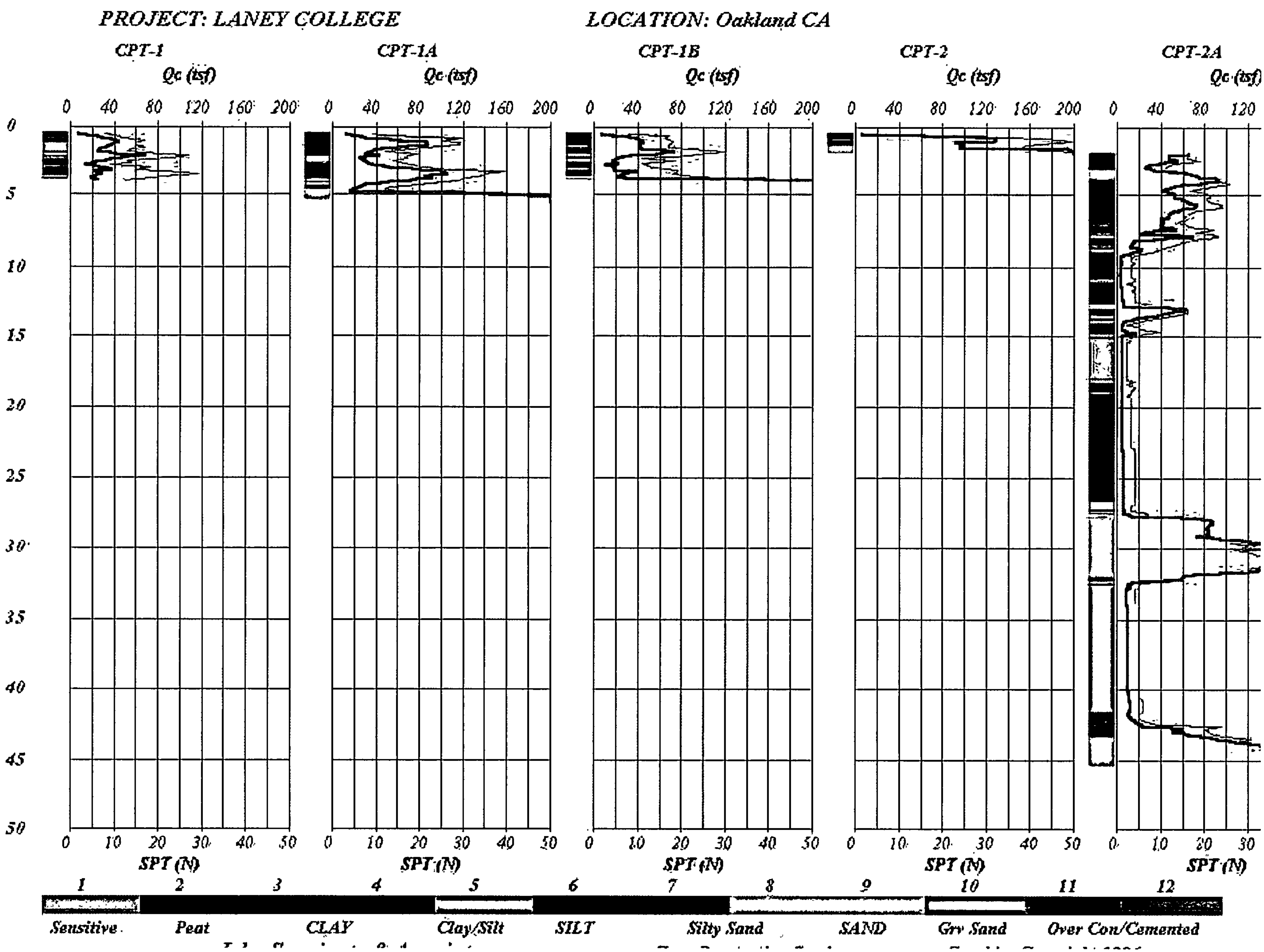
(*) Overconsolidated or Cemented

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

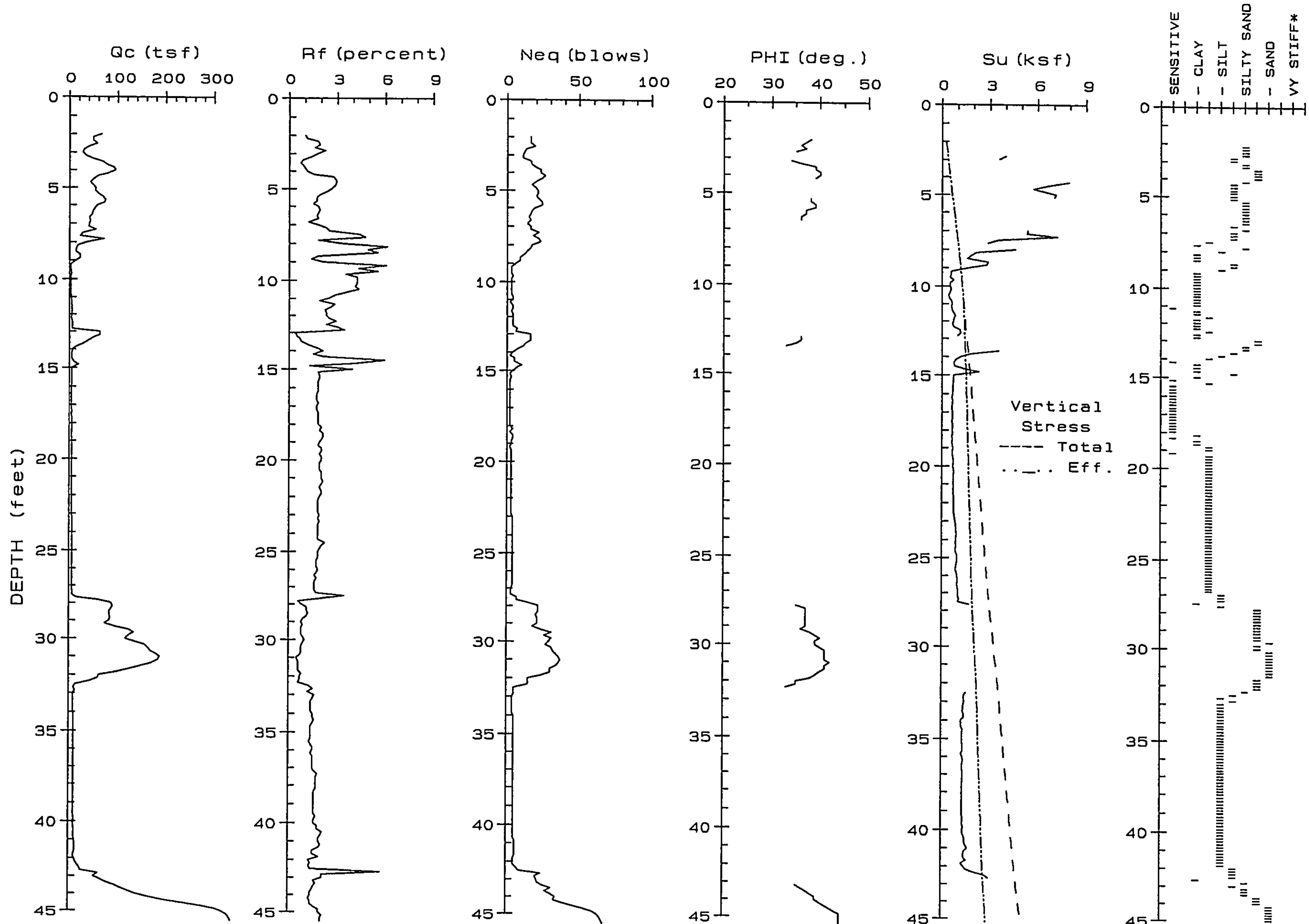
References: [1]Robertson, 1986, Olsen, 1988
[2]Bonaparte & Mitchell, 1979 (young bay mud Qc <= 9)
[2]Estimated from local experience (fine grained soils Qc > 9)

Note: Testing performed in accordance with ASTM D3441

John Sarmiento & Associates
Cone Penetrometer Testing Services



RESULTS OF CONE PENETROMETER TESTS (CPTs)



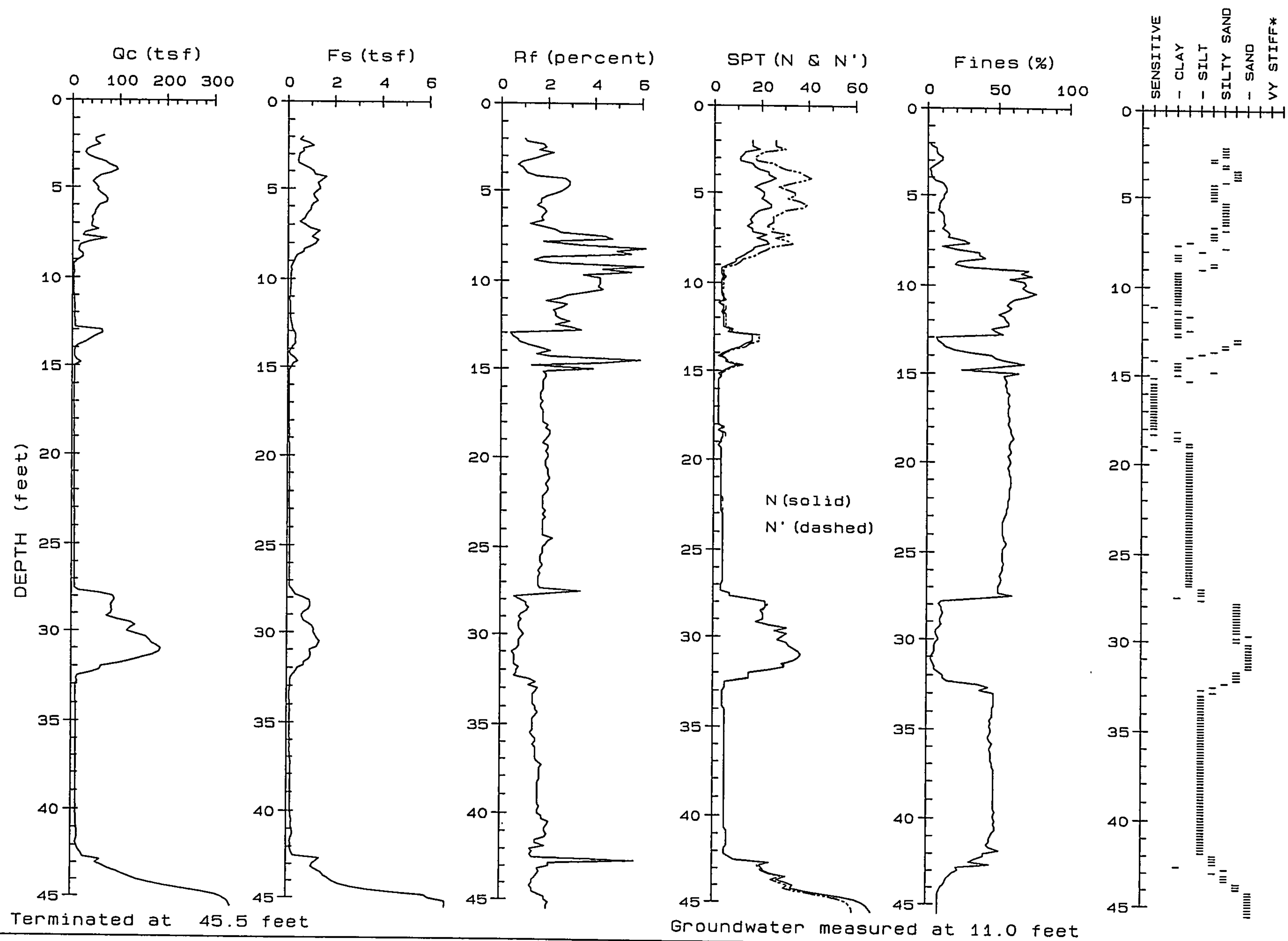
Terminated at 45.5 feet

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

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

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)
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

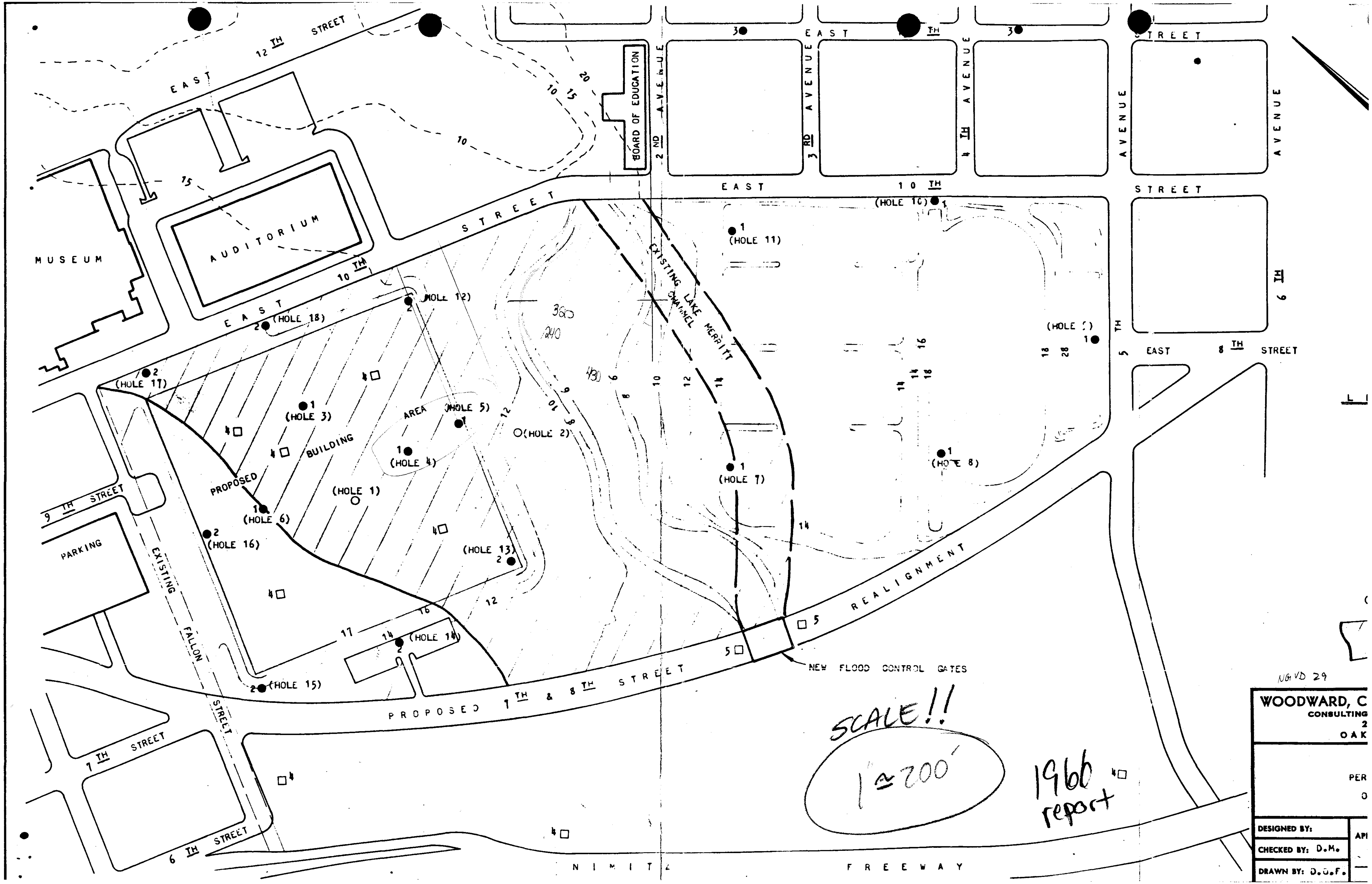
DEPTH (feet)	Qc (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT (N')	TotHzStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
34.50	9.78	0.143	1.5	5	5	3.75	----	1.32	"	"
35.00	9.60	0.138	1.4	5	5	3.80	----	1.28	"	"
35.50	9.93	0.126	1.3	5	5	3.85	----	1.33	"	"
36.00	9.83	0.147	1.5	5	5	3.90	----	1.31	"	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

John Sarmiento & Associates
Cone Penetration Testing Service

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



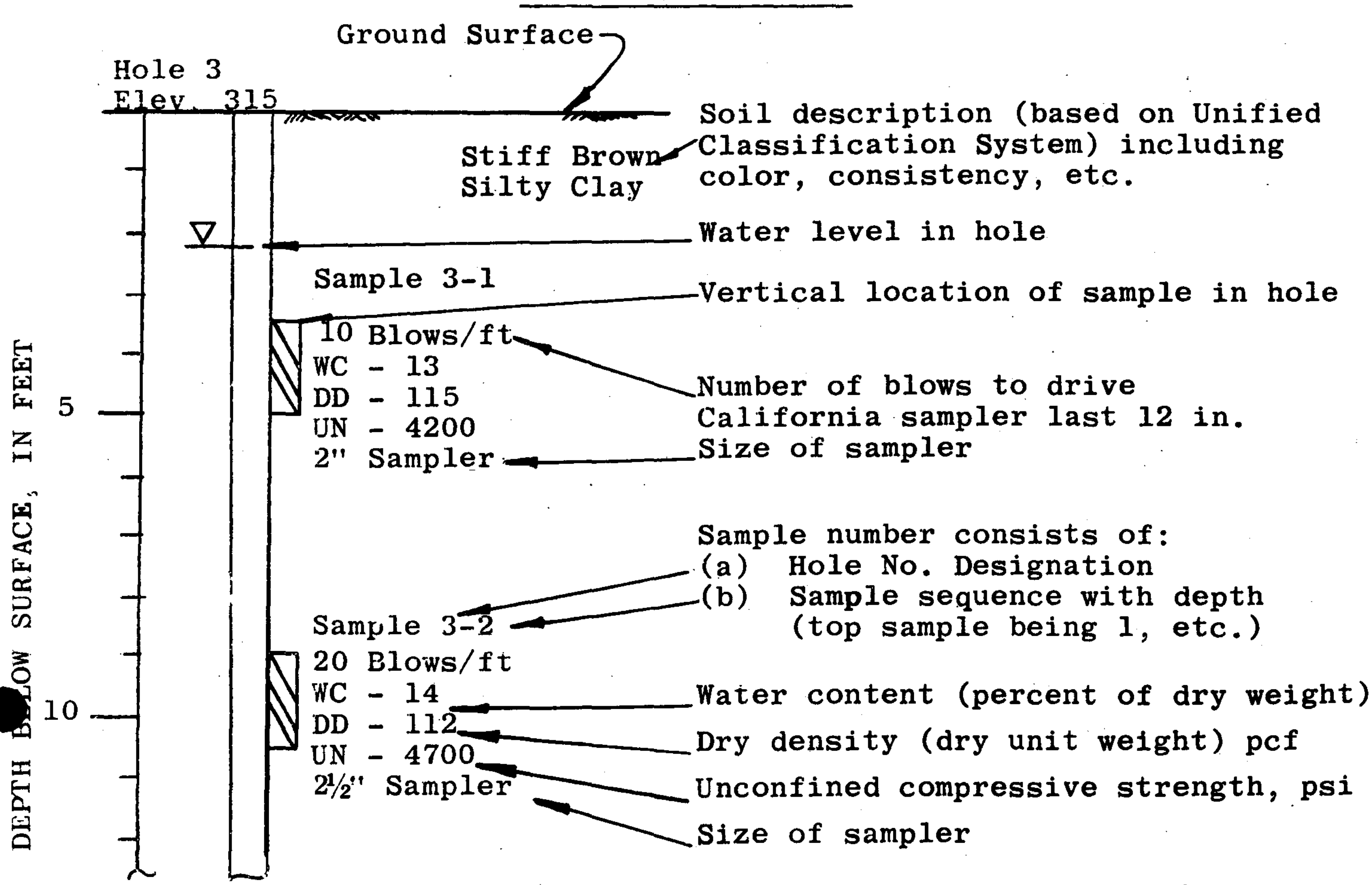
EAST 12 TH STREET
BOARD OF EDUCATION
MUSEUM
AUDITORIUM
EAST 10 TH STREET
2 ND AVENUE
3 RD AVENUE
4 TH AVENUE
AVENUE
6 TH AVENUE
EAST TH STREET
EAST 10 TH STREET
EAST 8 TH STREET
5 TH
EXISTING LAKE MERRITT CHANNEL
(HOLE 10)
(HOLE 11)
(HOLE 12)
(HOLE 18)
(HOLE 17)
(HOLE 3)
(HOLE 5)
(HOLE 2)
(HOLE 4)
(HOLE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

A P P E N D I X

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



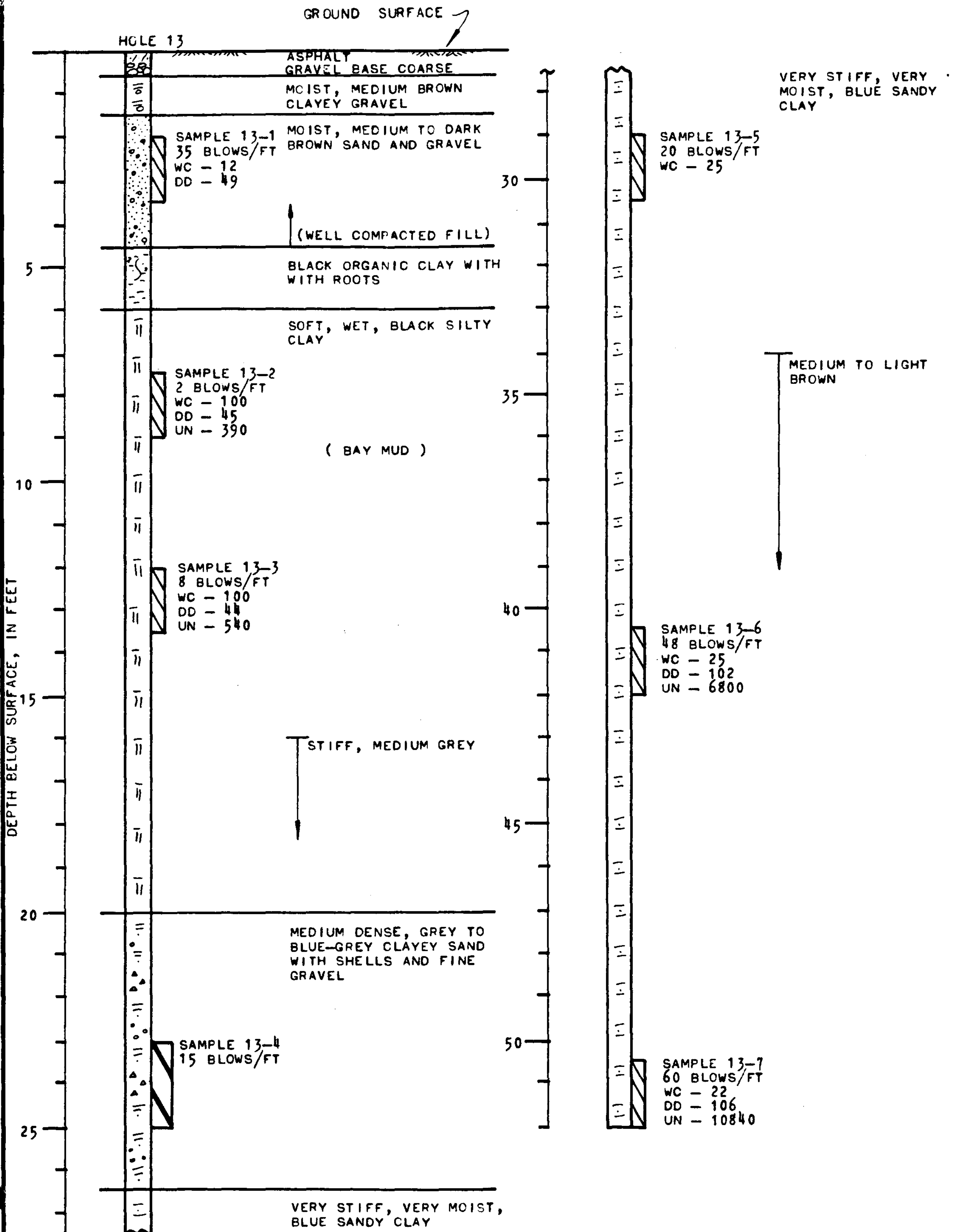


FIG. 20 — LOGS OF BORINGS

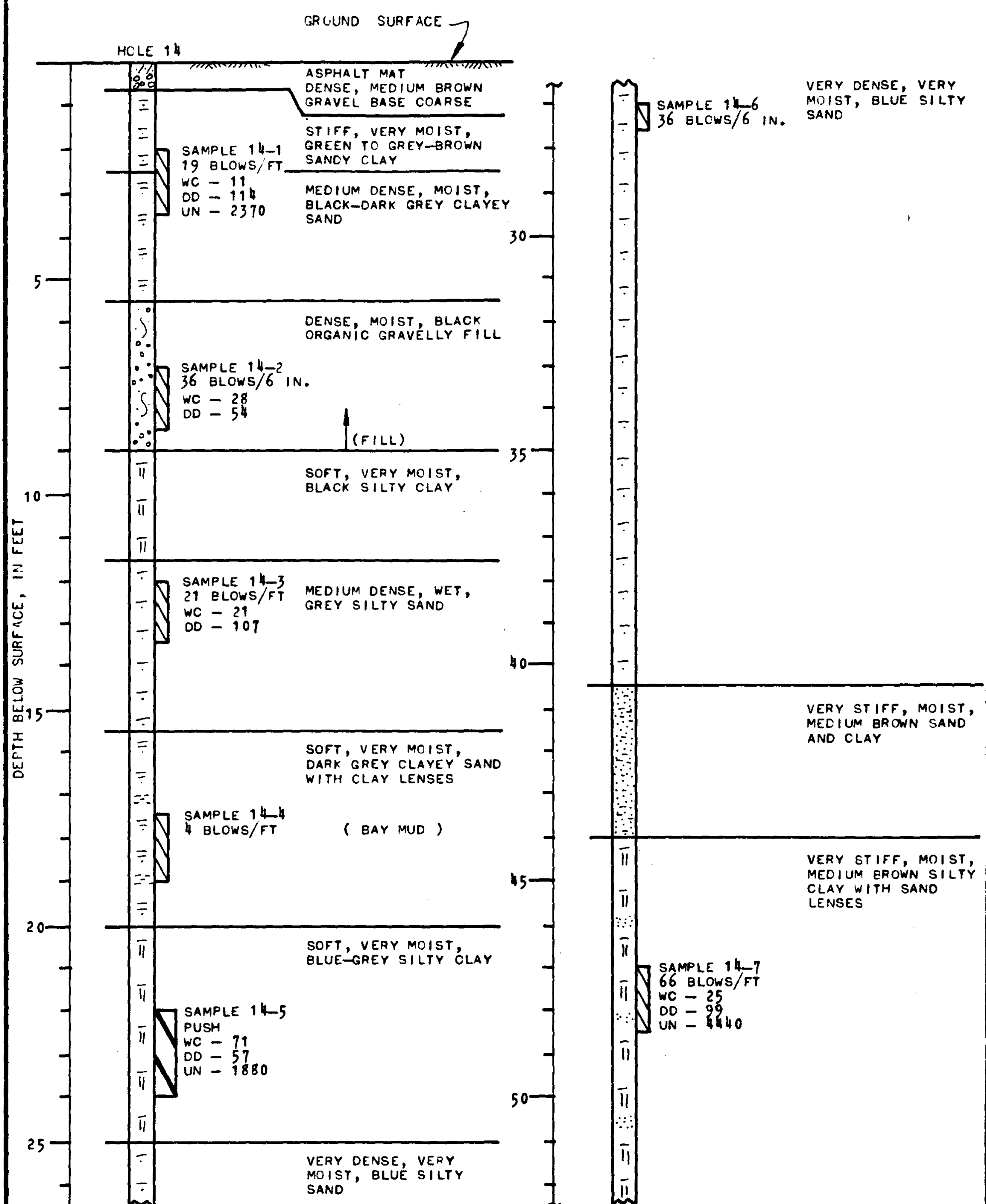


FIG. 21 — LOGS OF BORINGS

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

HOLE 14 (CONT'D)

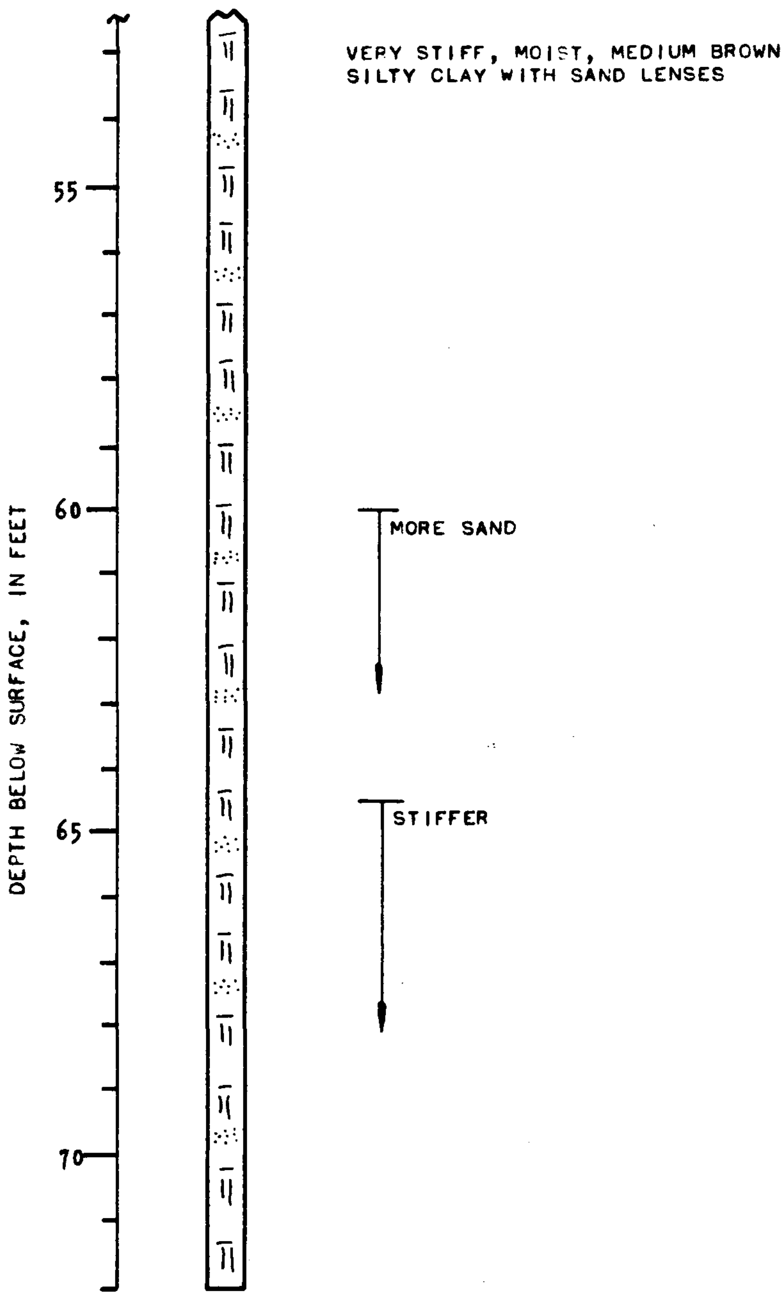
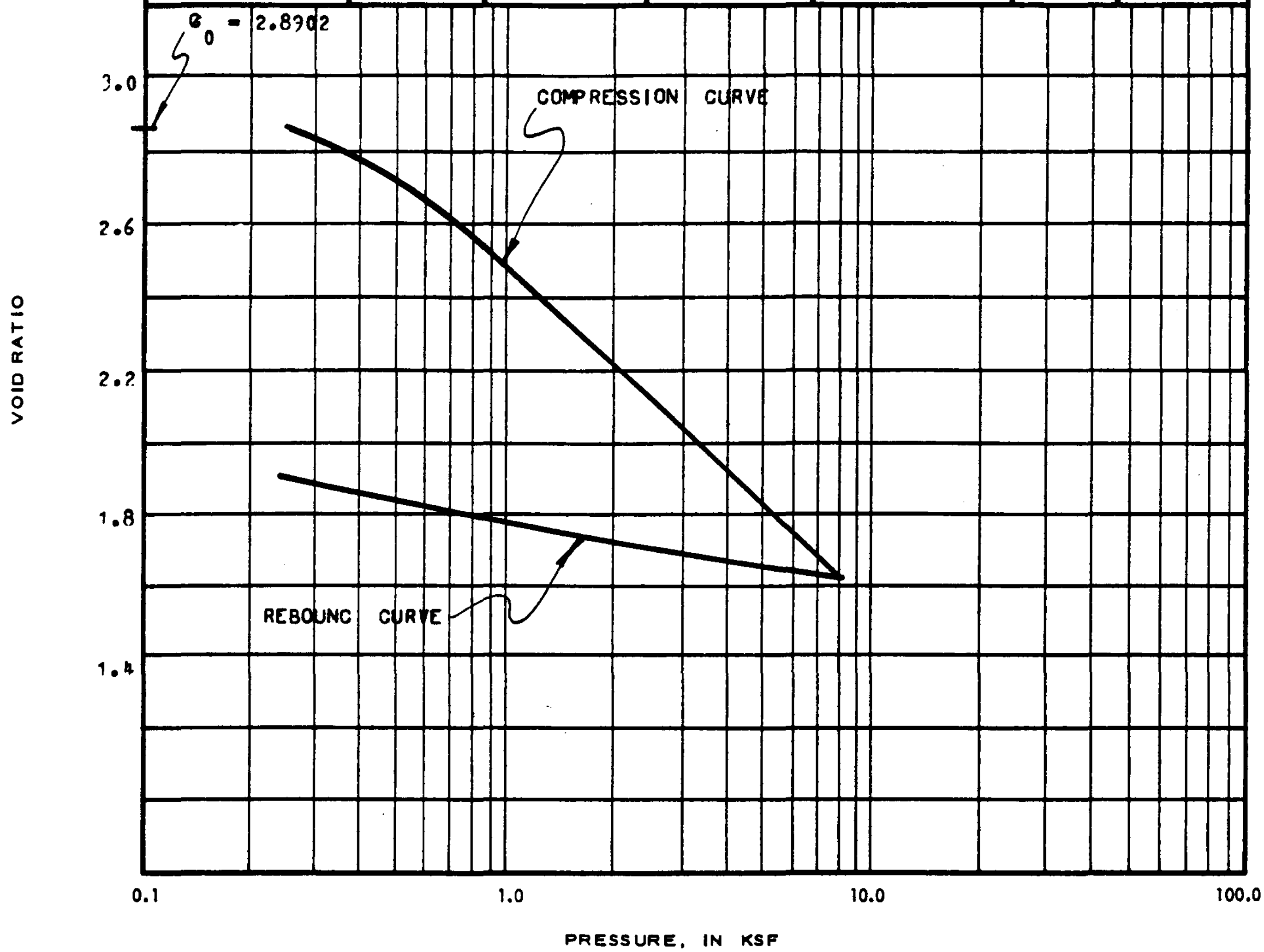
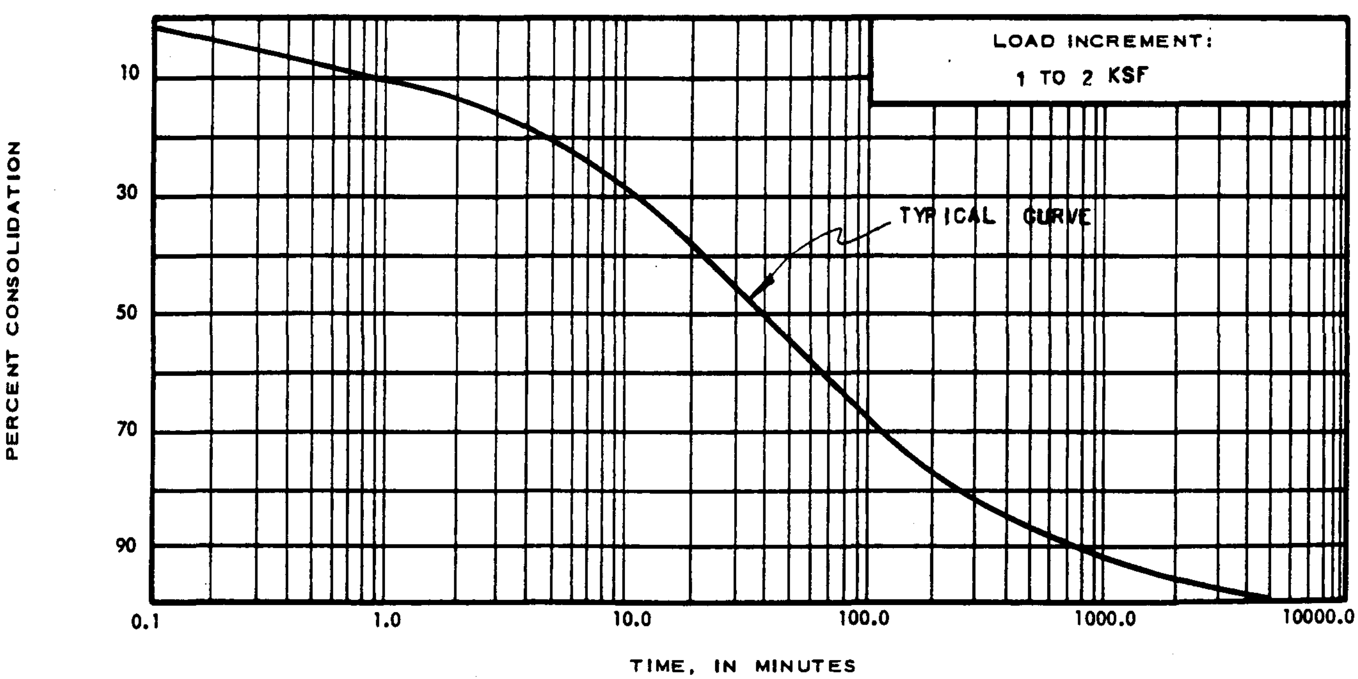


FIG. 22 — LOGS OF BORINGS

SAMPLE NO. 13-3-9	SUMMARY OF TEST RESULTS					
	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	52.17	99.81	.6694	





CONSOLIDATION TEST

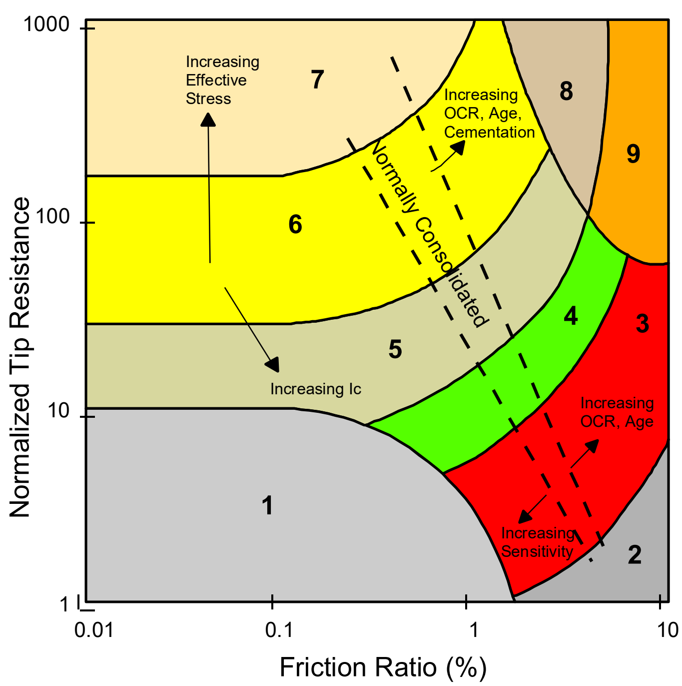
FIG. 26

Appendix 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

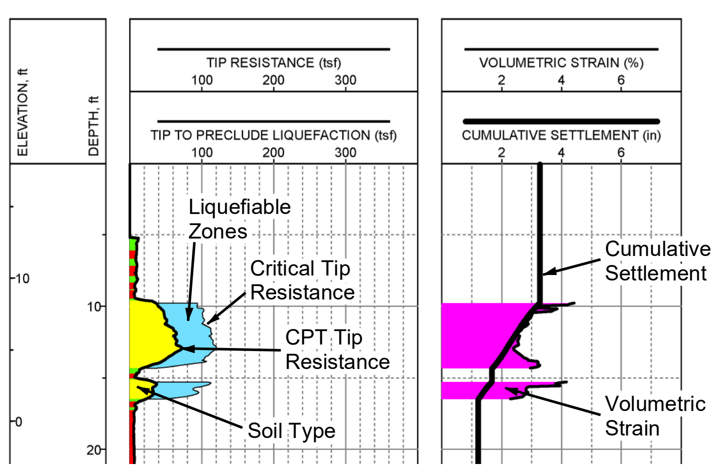
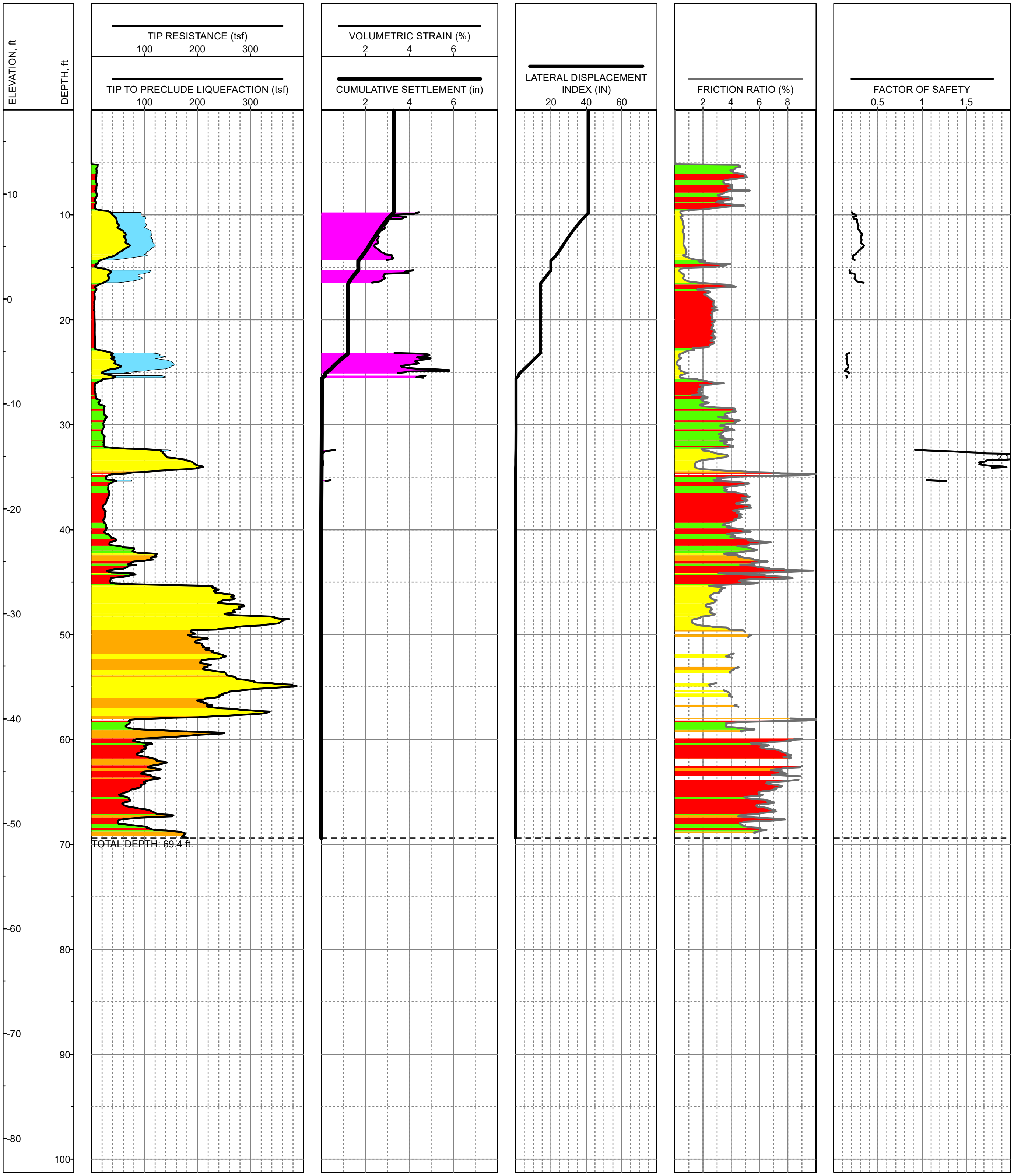


PLATE D-0: KEY TO LIQUEFACTION LOGS

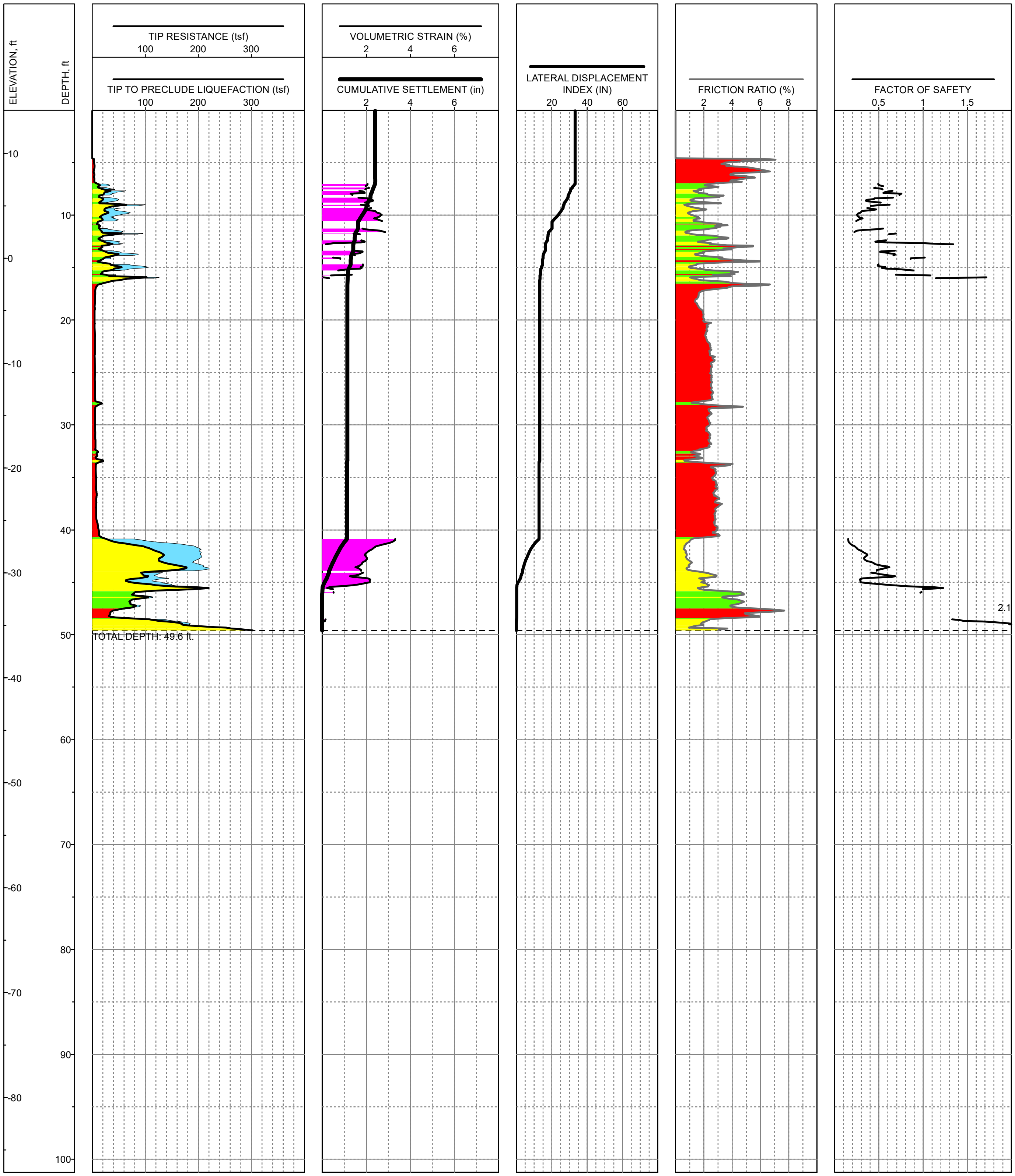


LOCATION: E6,052,365, N2,116,794, NAD83 SP CA Z3 FT
SURFACE EL: 18.02 ft
COMPLETION DEPTH: 69.4 ft
TESTDATE: 3/29/2019

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

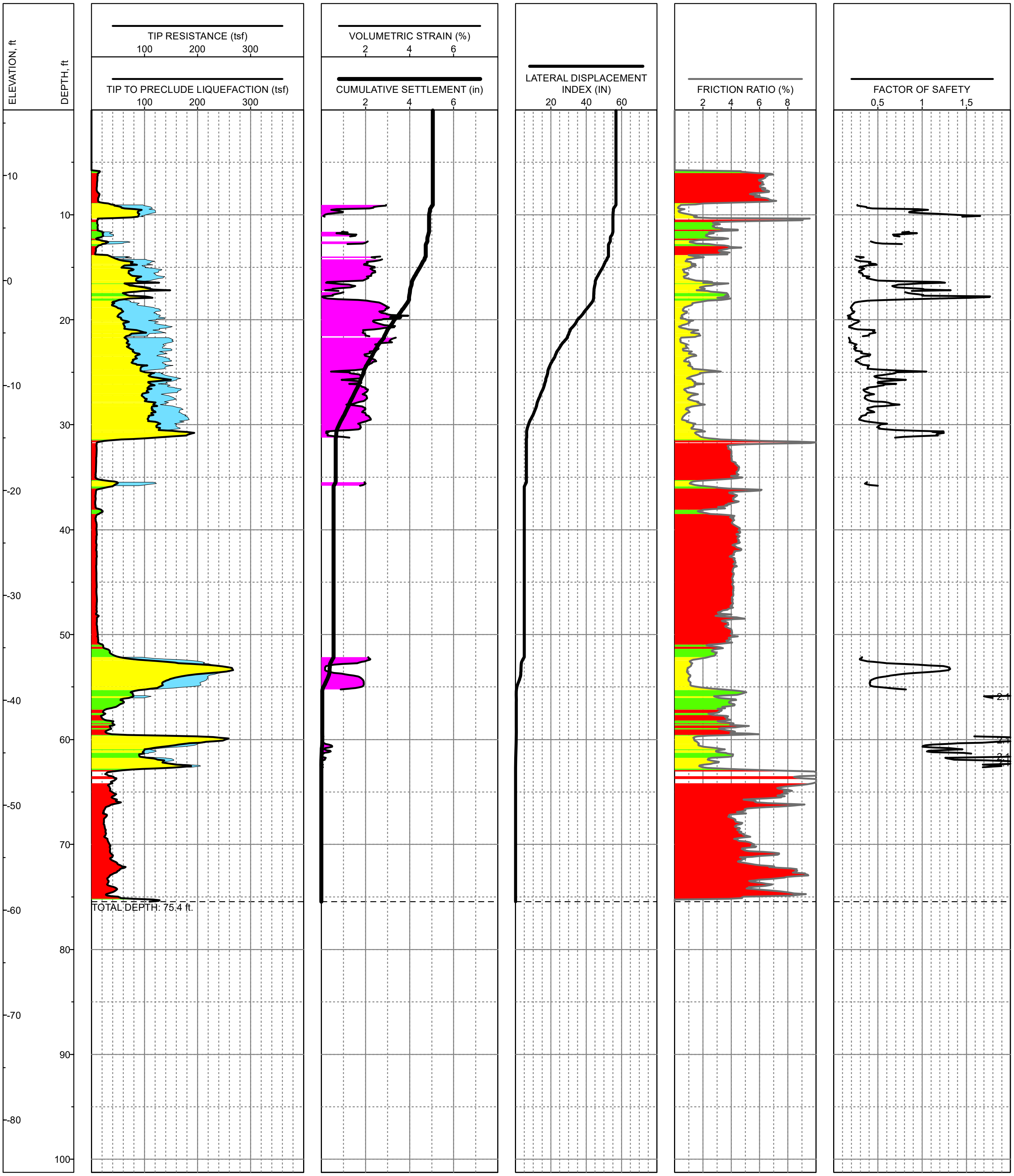
PLATE D-1: LOG OF 2019-CPT-01 – M=7.0, PGA=0.810, N, TL, TR



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

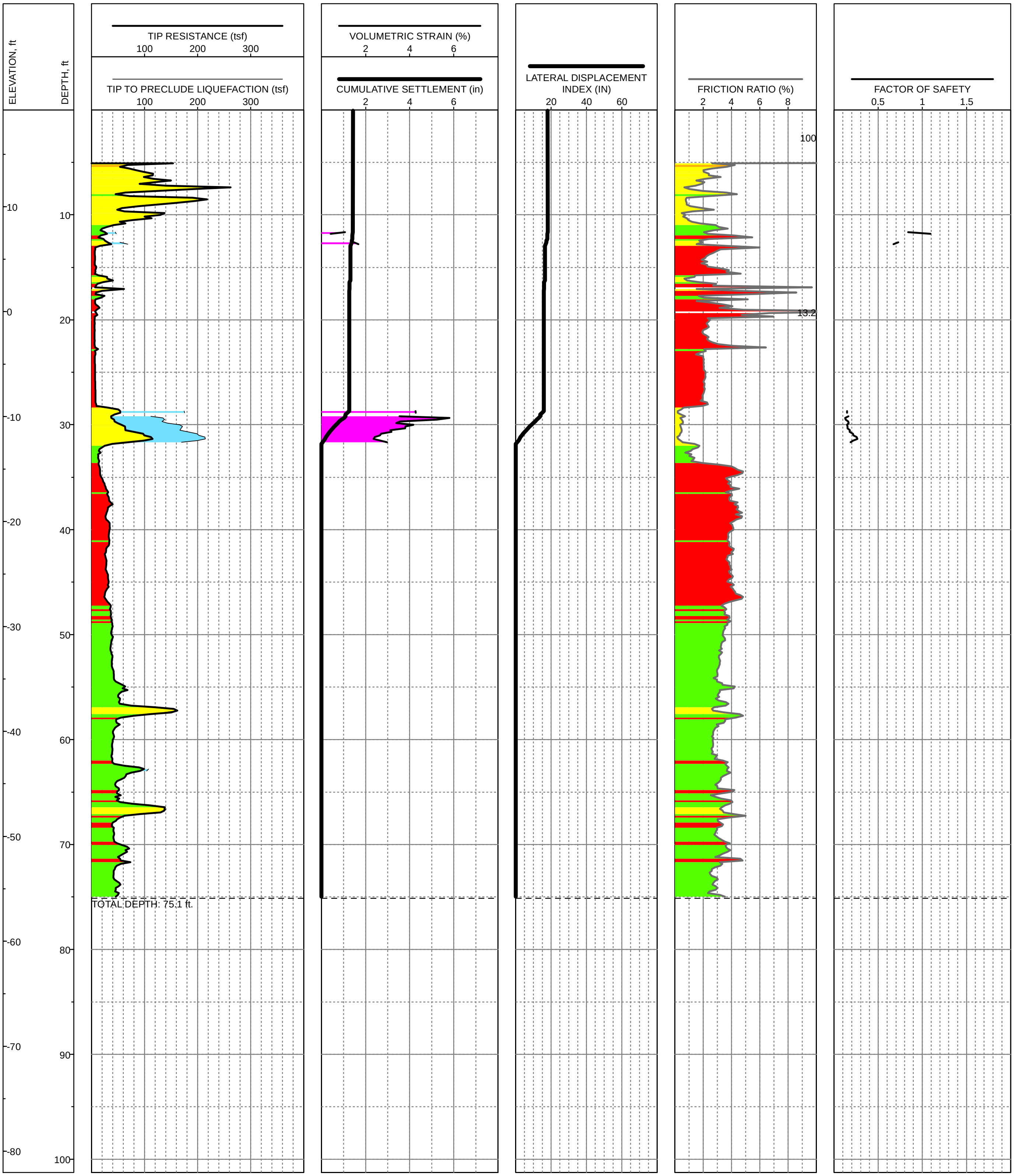


LOCATION: E6,052,570, N2,116,535, NAD83 SP CA Z3 FT
SURFACE EL: 16.26 ft
COMPLETION DEPTH: 75.4 ft
TESTDATE: 3/29/2019

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

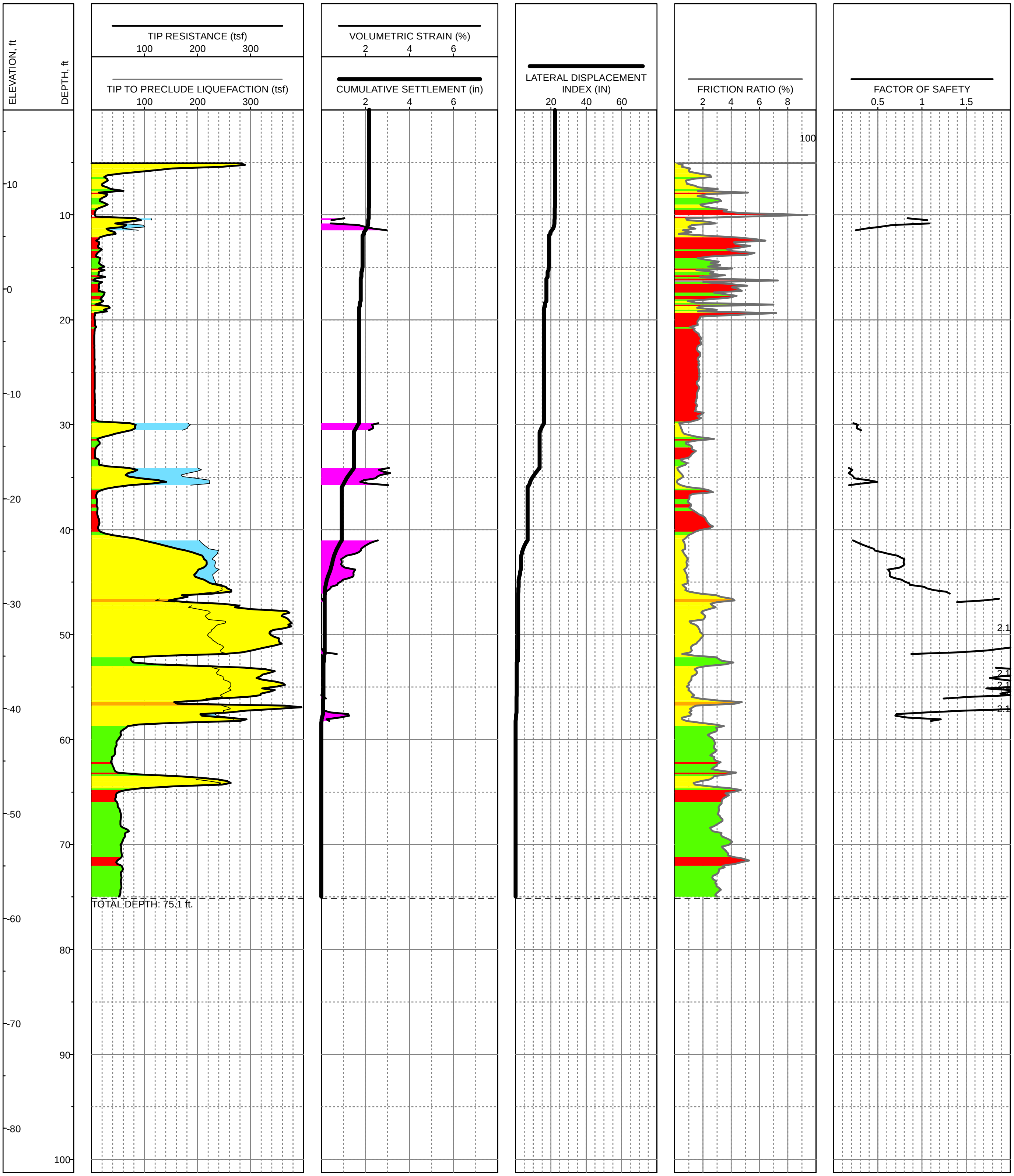
W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\01_Explorations\CPT\2019\Logs\2019_06_18_Logs_M7_0_a0_810_N_TL_TR\MXD\CPT_Logs_M7_0_a0_810_N_TL_TR,6/21/2019,A.Ramirez



LOCATION: E6,052,490, 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

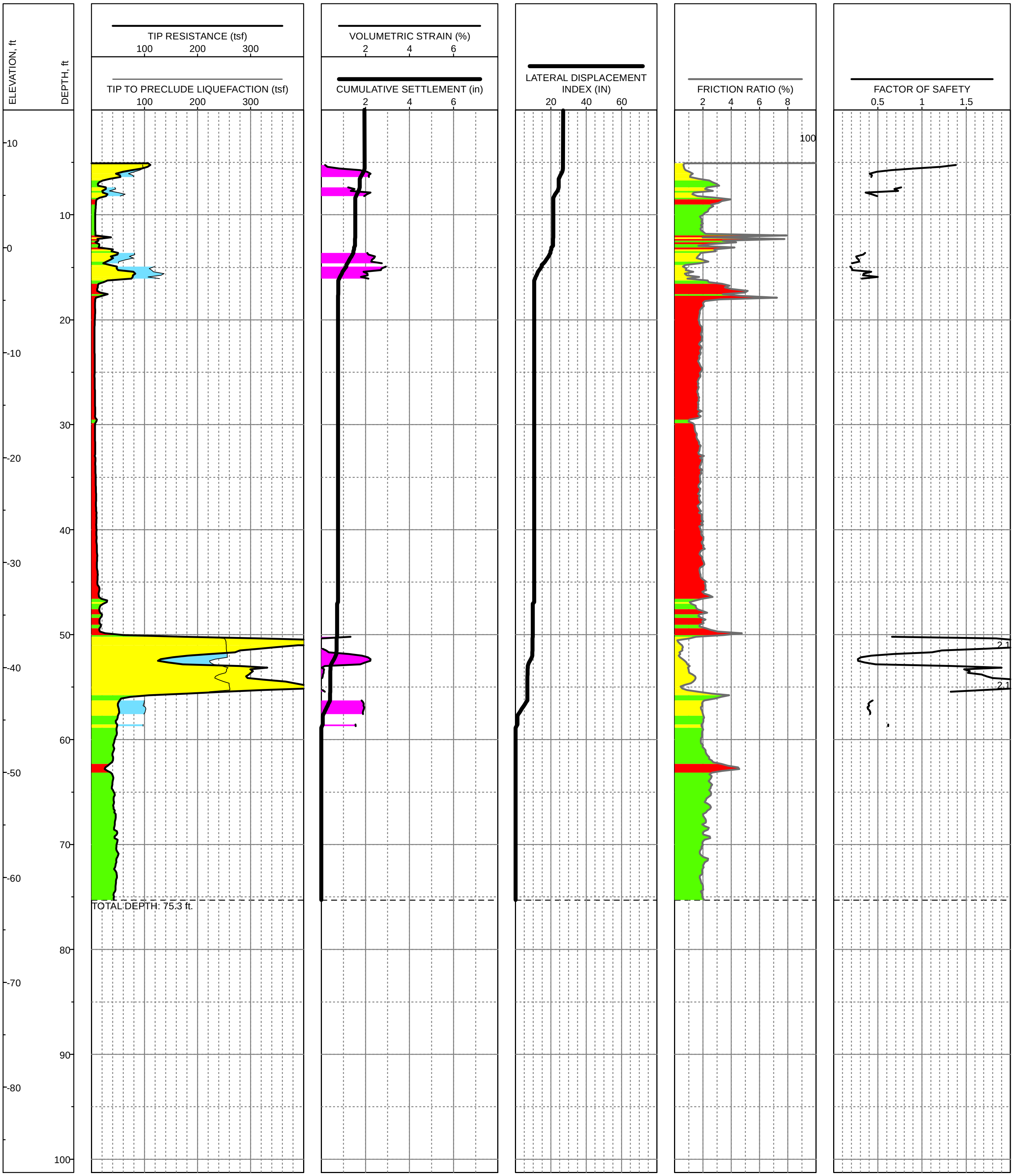
PLATE D-4: LOG OF 2020-CPT-04 – M=7.0, PGA=0.810, N, TL, TR



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

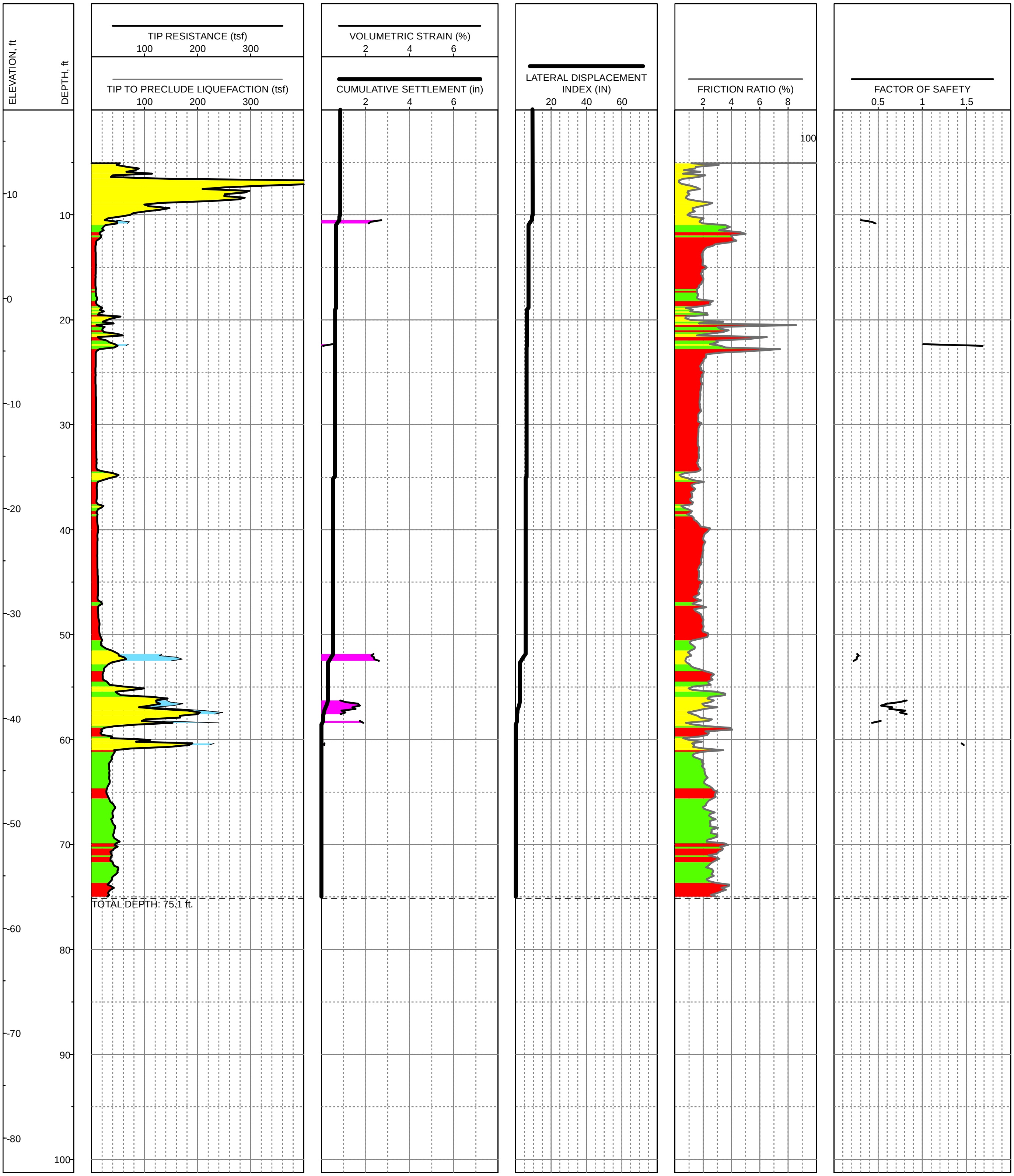
PLATE D-5: LOG OF 2020-CPT-05 – M=7.0, PGA=0.810, N, TL, TR



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

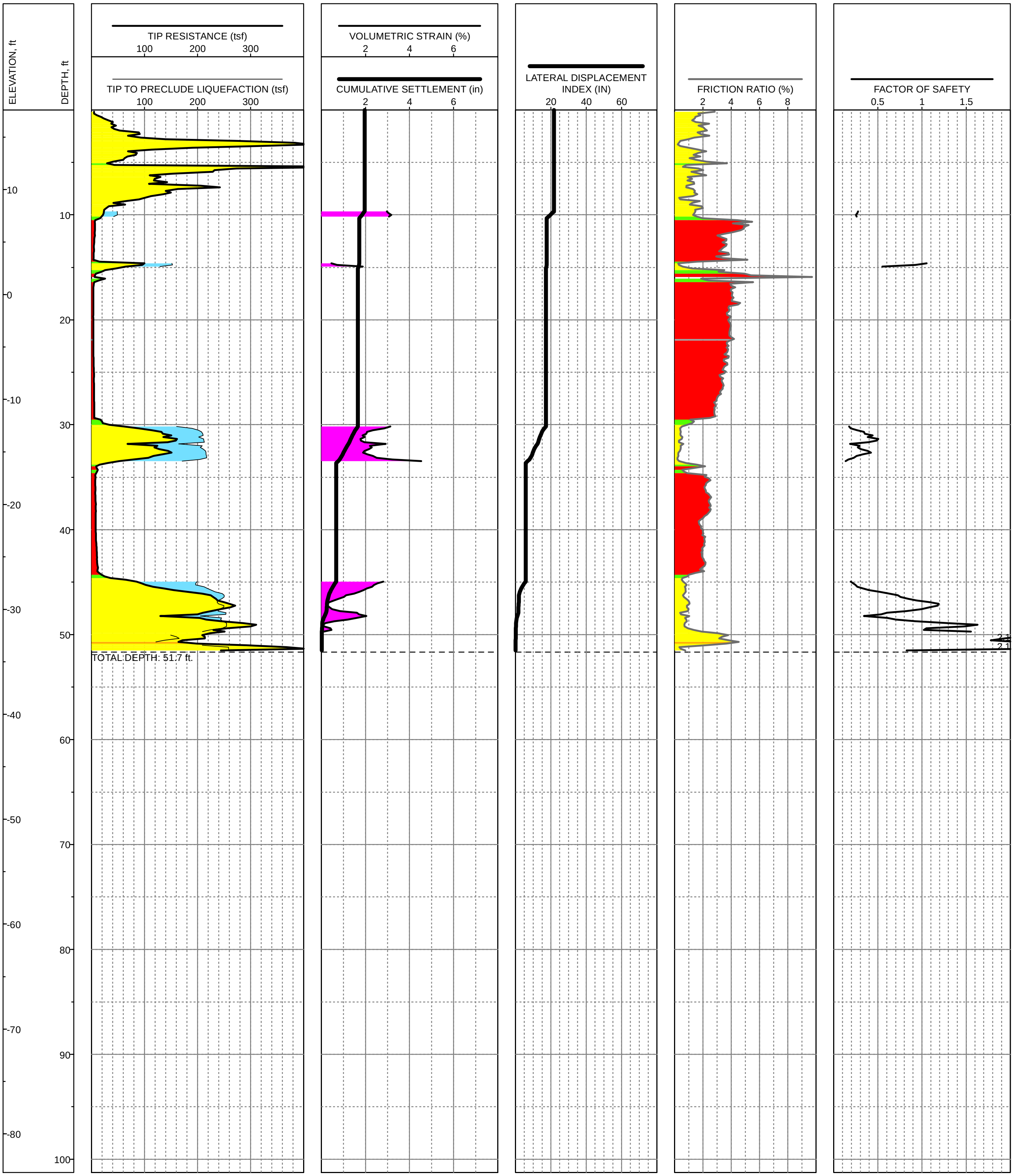
PLATE D-6: LOG OF 2020-CPT-06 – M=7.0, PGA=0.810, N, TL, TR



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

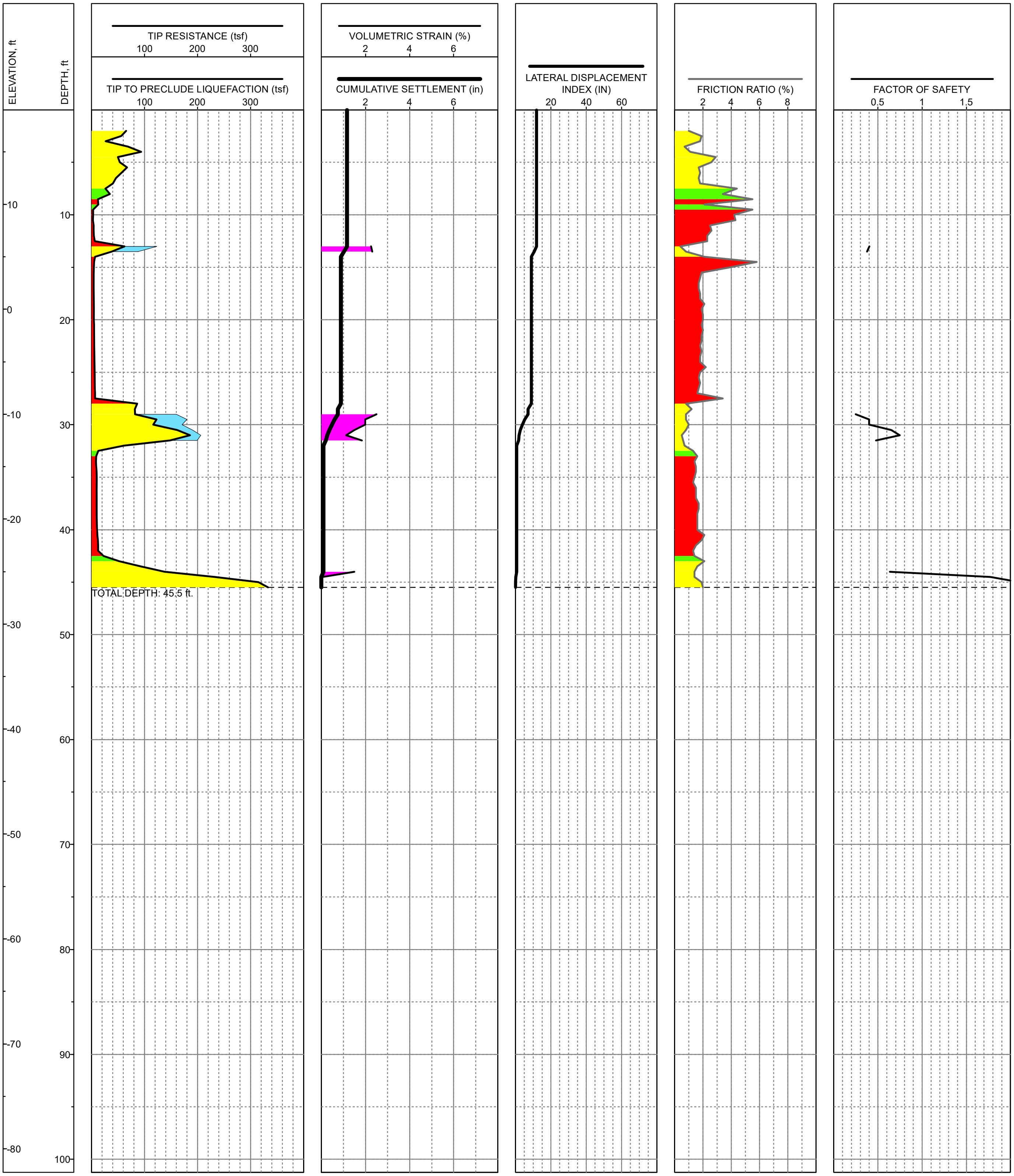
PLATE D-7: LOG OF 2020-CPT-07 – M=7.0, PGA=0.810, N, TL, TR



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

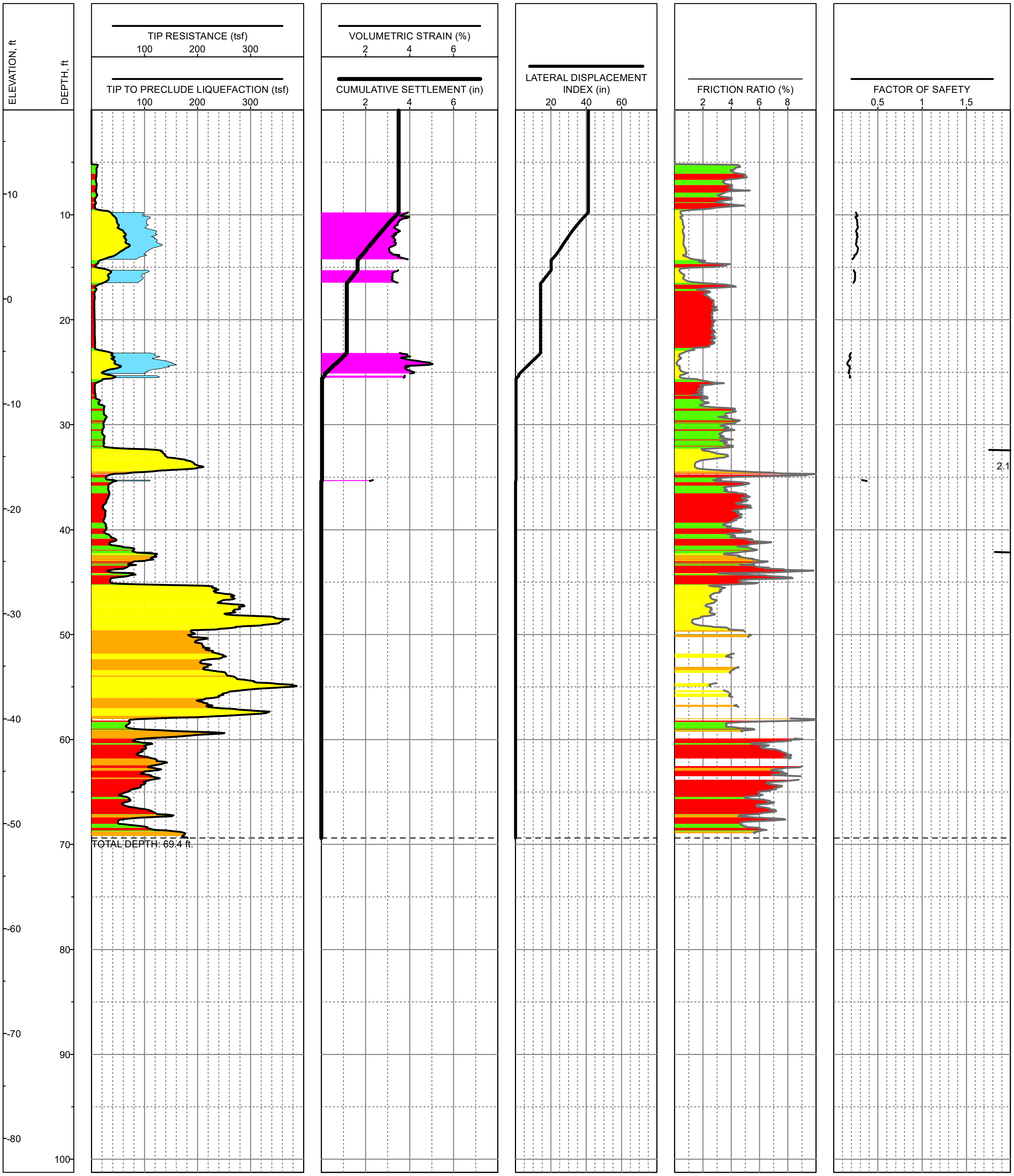


LOCATION: E6,052,497, N2,116,672, NAD83 SP CA Z3 FT
SURFACE EL: 19.00 ft
COMPLETION DEPTH: 45.5 ft
TESTDATE: 2/26/2002

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

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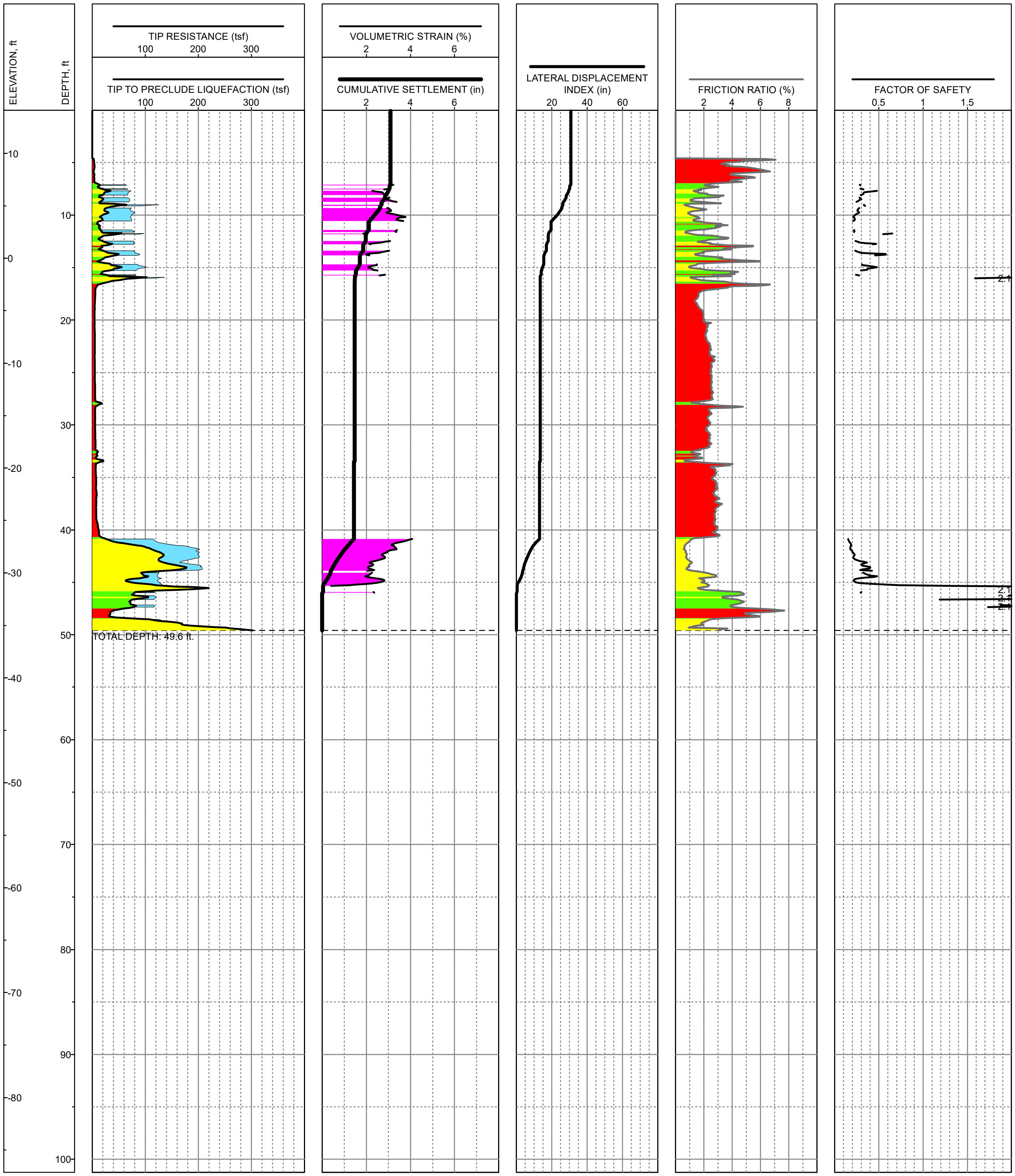
PLATE D-10: LOG OF 2019-CPT-01 – M=7.0, PGA=0.810, B, TL, TR



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

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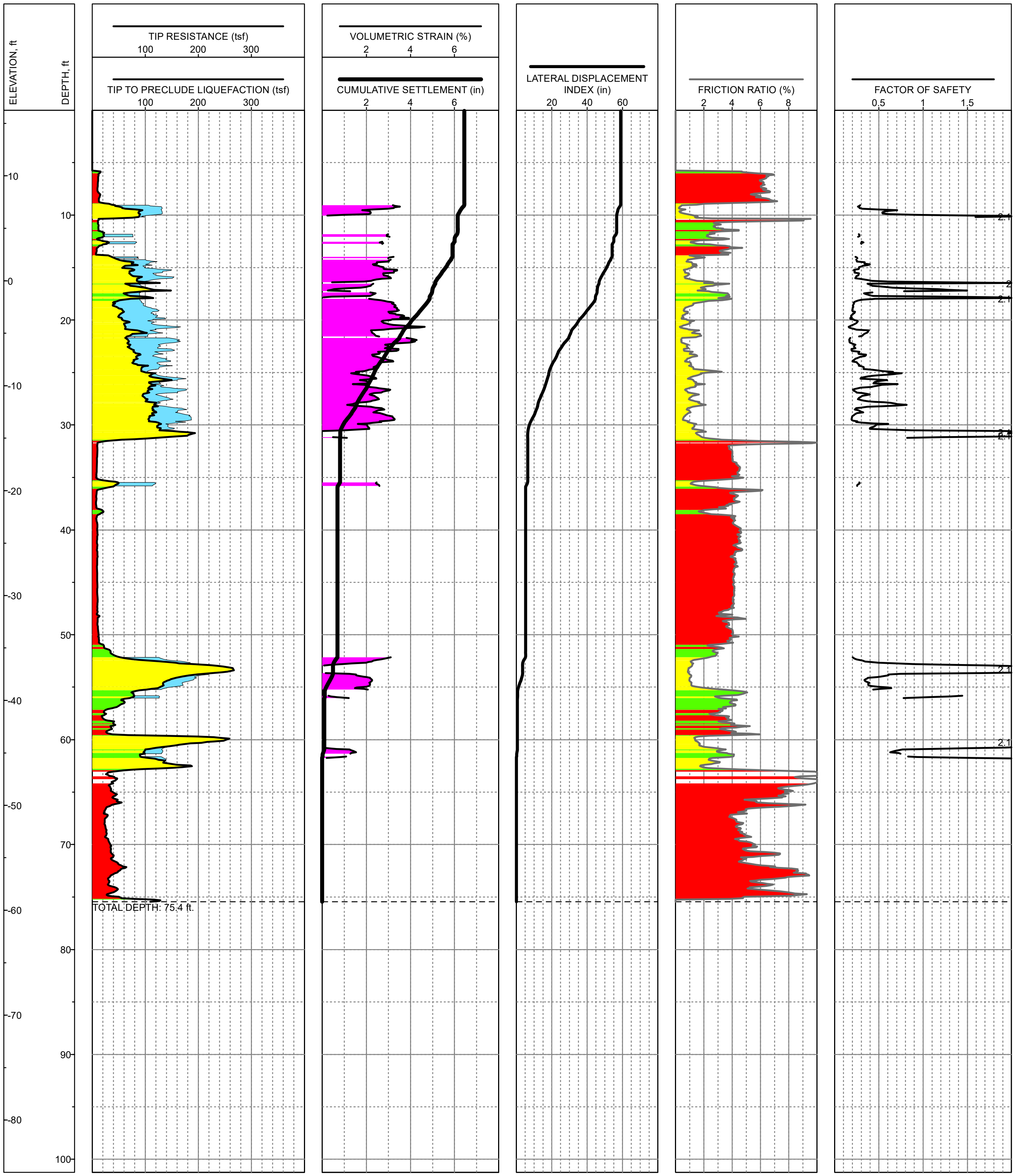
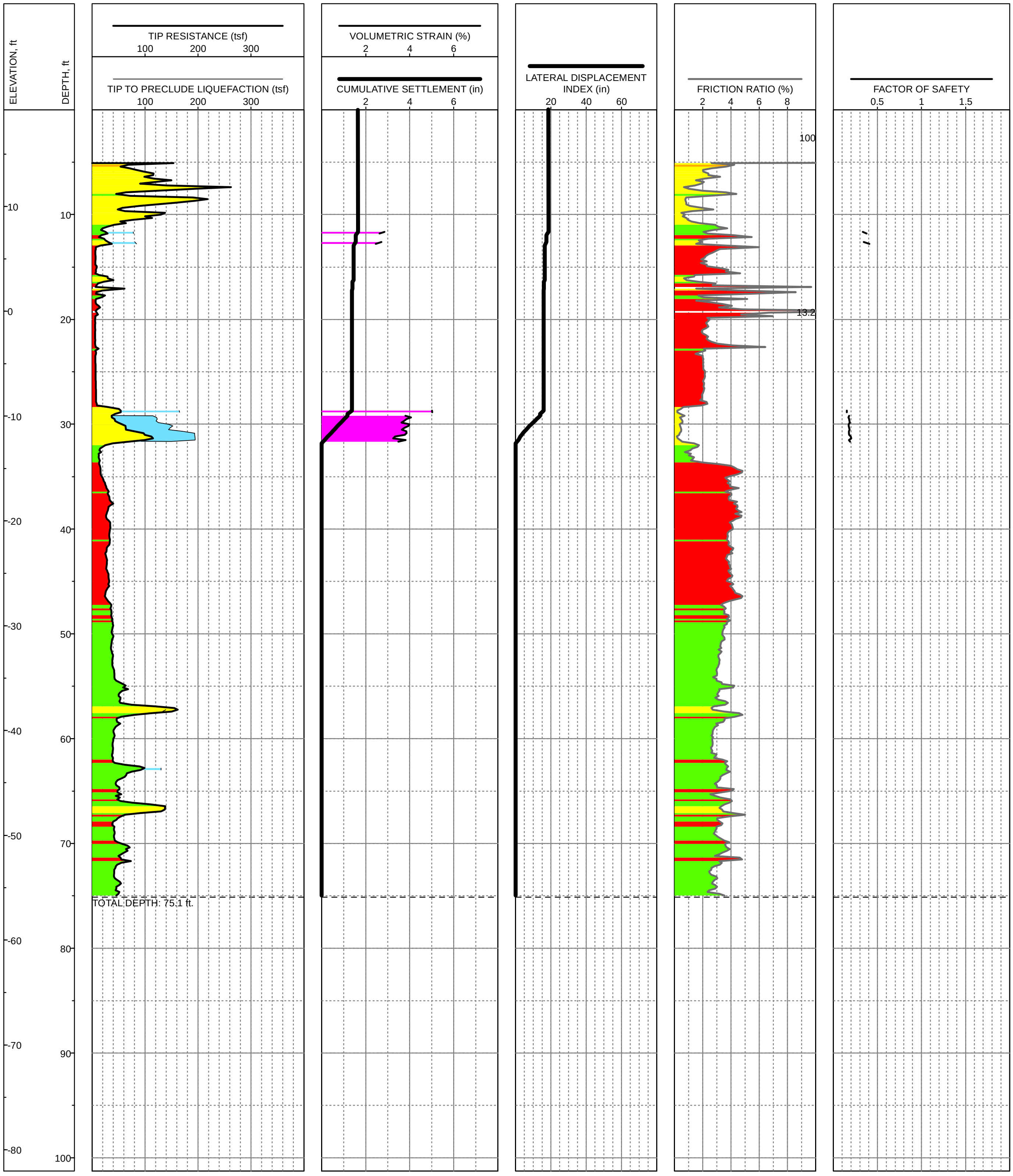


LOCATION: E6,052,570, N2,116,535, NAD83 SP CA Z3 FT
SURFACE EL: 16.26 ft
COMPLETION DEPTH: 75.4 ft
TESTDATE: 3/29/2019

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

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

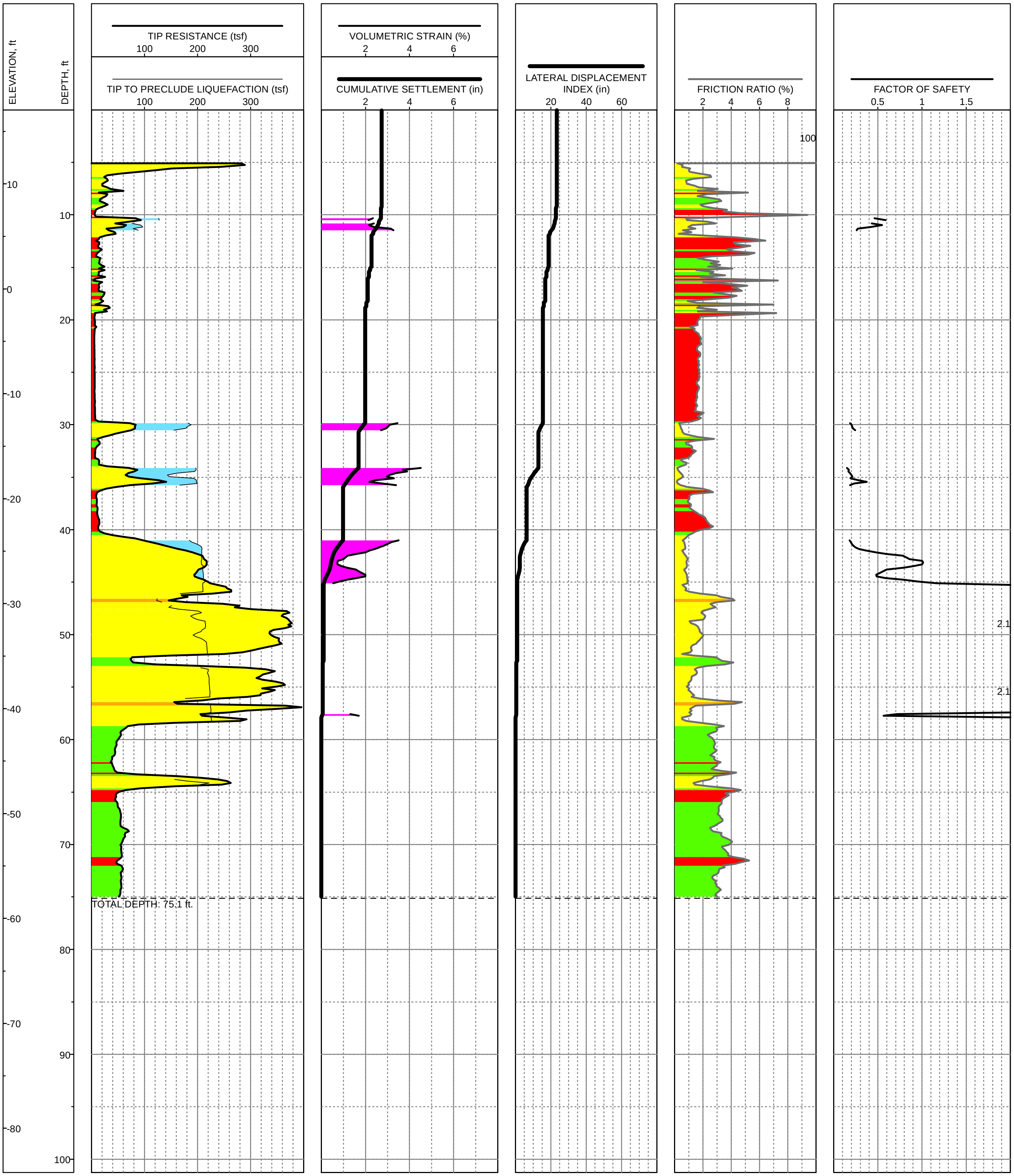
W:\Projects\Location-72\2019\04.72190021 Laney College Library Learning Resource Center\08_GIS\01_Explorations\CPT\2019\Logs\2019_06_18_Logs_M7_0_a0_810_B_IB_TL_TRIMXD\CPT_Logs_M7_0_a0_810_B_IB_TL_TR,6/21/2019,A.Ramirez



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

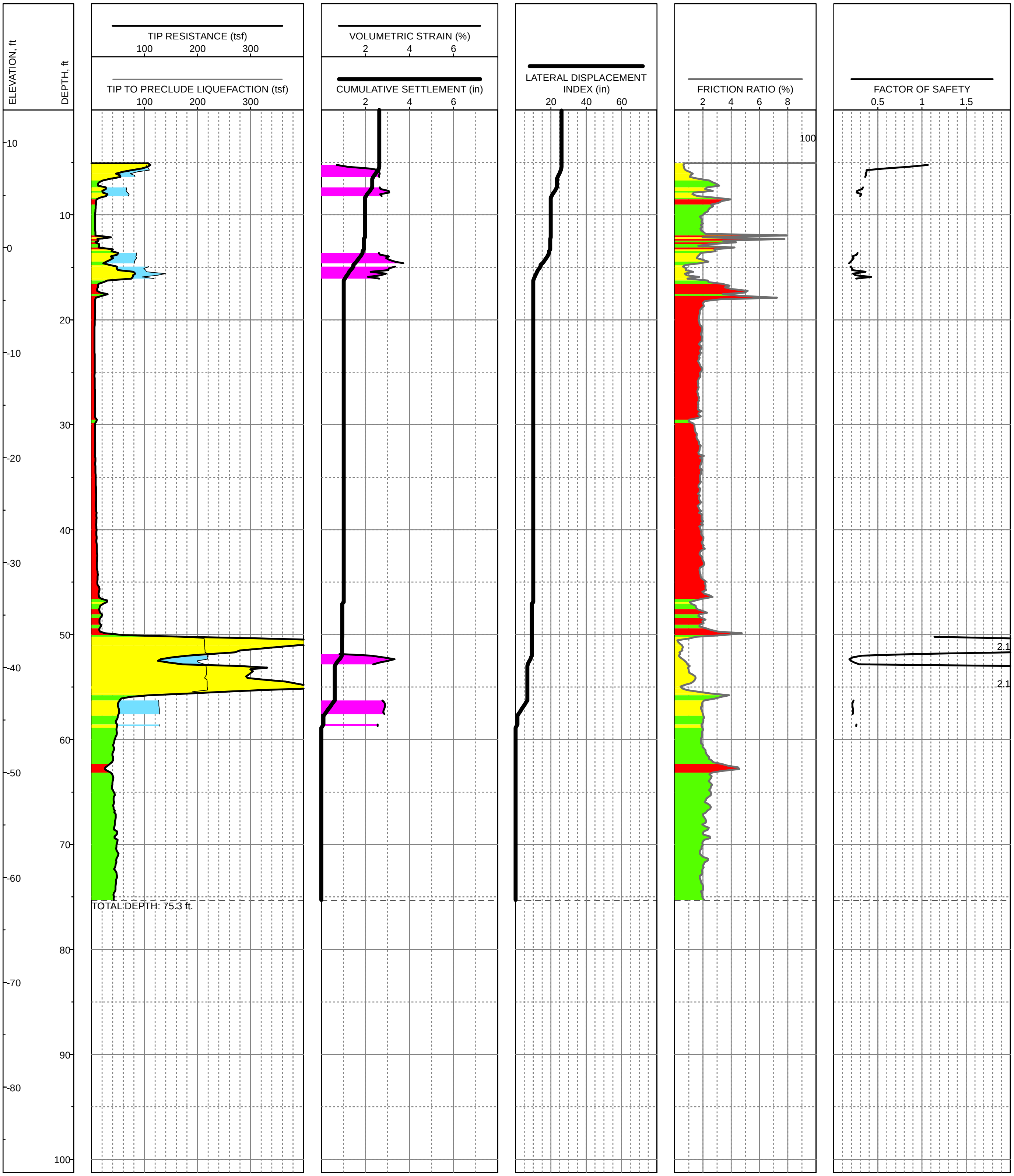
PLATE D-13: LOG OF 2020-CPT-04 – M=7.0, PGA=0.810, B, TL, TR



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

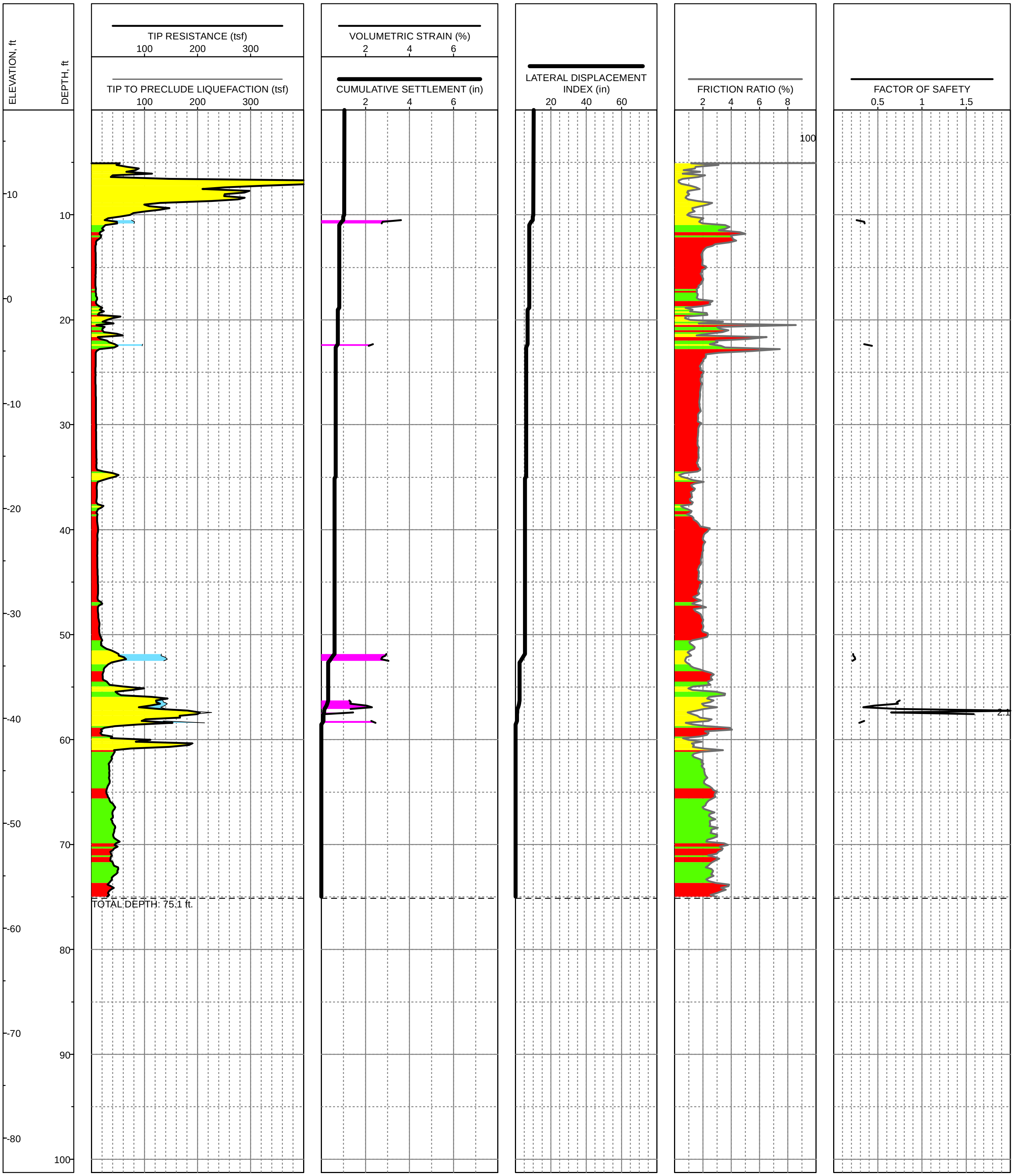
PLATE D-14: LOG OF 2020-CPT-05 – M=7.0, PGA=0.810, B, TL, TR



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

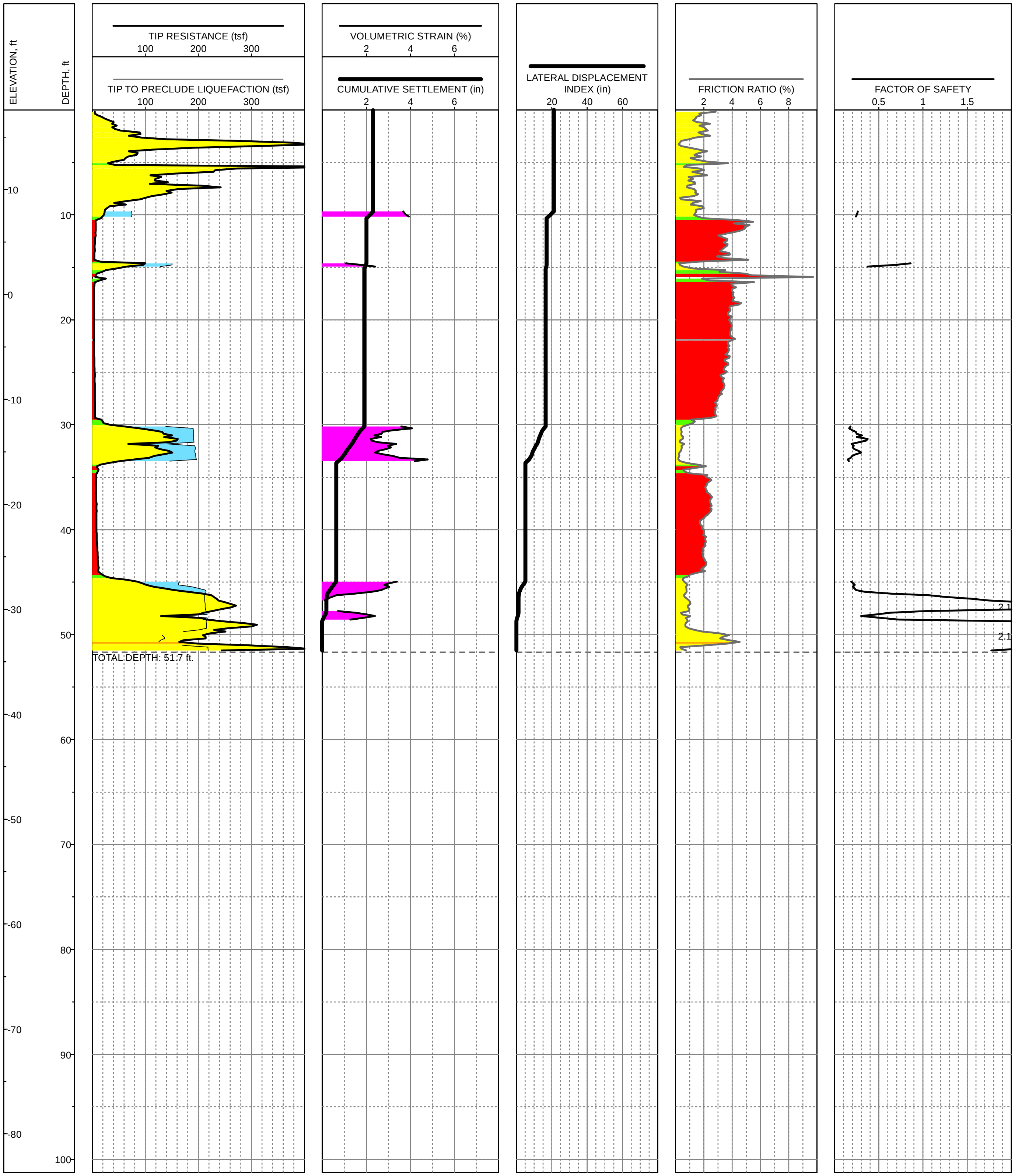
PLATE D-15: LOG OF 2020-CPT-06 – M=7.0, PGA=0.810, B, TL, TR



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

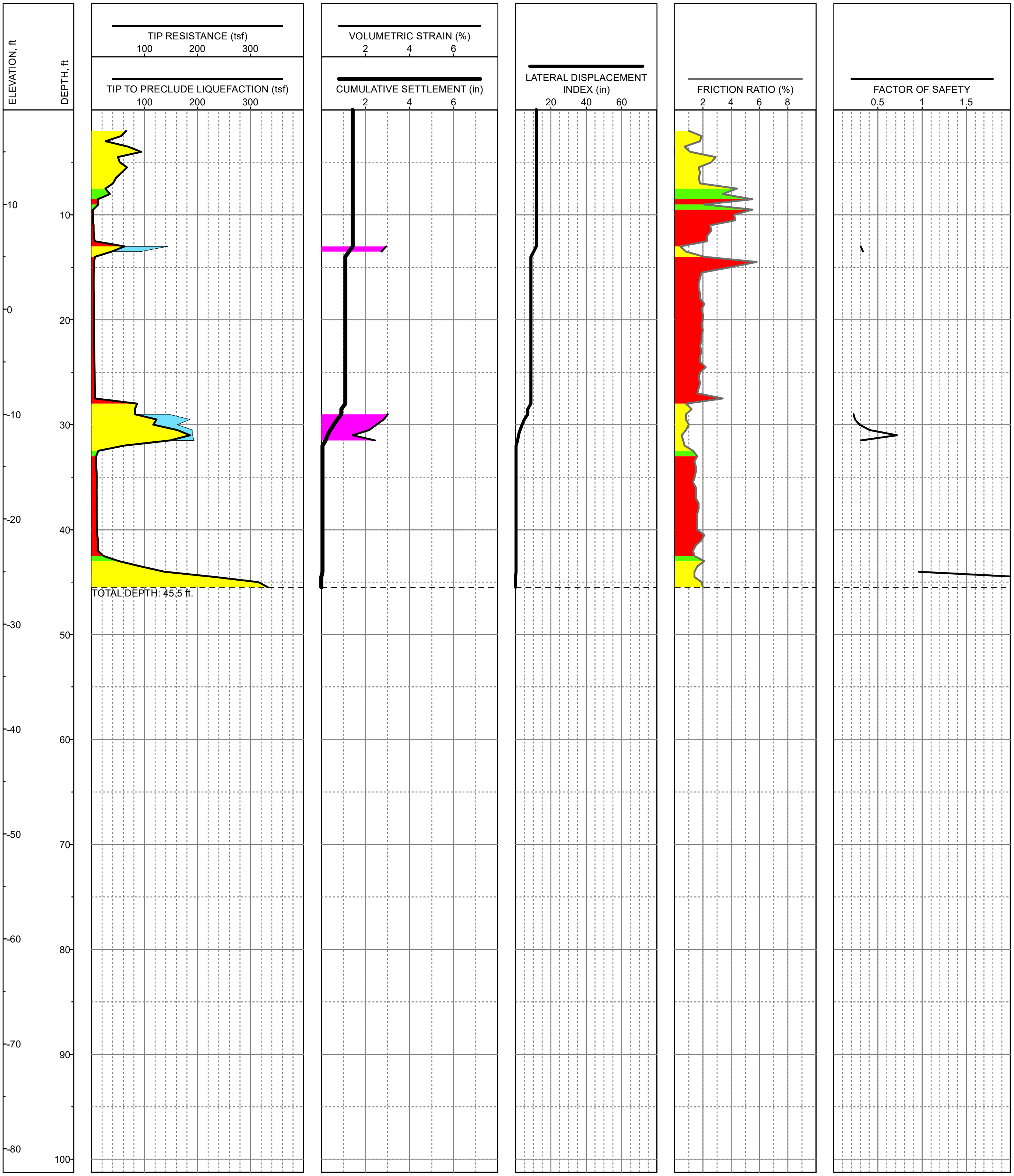
PLATE D-16: LOG OF 2020-CPT-07 – M=7.0, PGA=0.810, B, TL, TR



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

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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	$N_{1,60}$
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	15
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	14
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	26
-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	19
-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	28
-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	116
-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	75

Elevation	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	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	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	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	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	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	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	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	$N_{1,60}$	Assumed FC	ΔN	$N_{1,60,cs}$
ft	ft	m	blow/ft	psf	kPa	psf	kPa		Y/N						%		blow/ft
-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	30	5	11
-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	30	5	25
-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	30	5	106
-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	30	5	51
-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	30	5	85
-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	30	5	40
-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	30	5	44
-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	30	5	48

Elevation	r_d	CSR	MSF	K_σ	$CRR_{M=7.5,1 atm}$	CRR	FS	γlim	Fα	γmax	ε_v	ΔH (ft)	ΔS (in)	ΔLDI (in)
ft											%	ft	in	
-14.7	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	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	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	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	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	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	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	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	Fα	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

Appendix 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

FUGRO

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
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	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	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	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	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	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	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	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	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	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**

FUGRO

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-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				

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	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	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	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	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	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	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	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	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	83.60	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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	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 ΣΔs	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			
Water Table Depth from ground surface	9.1	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	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

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 x $\Sigma\Delta s$	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%

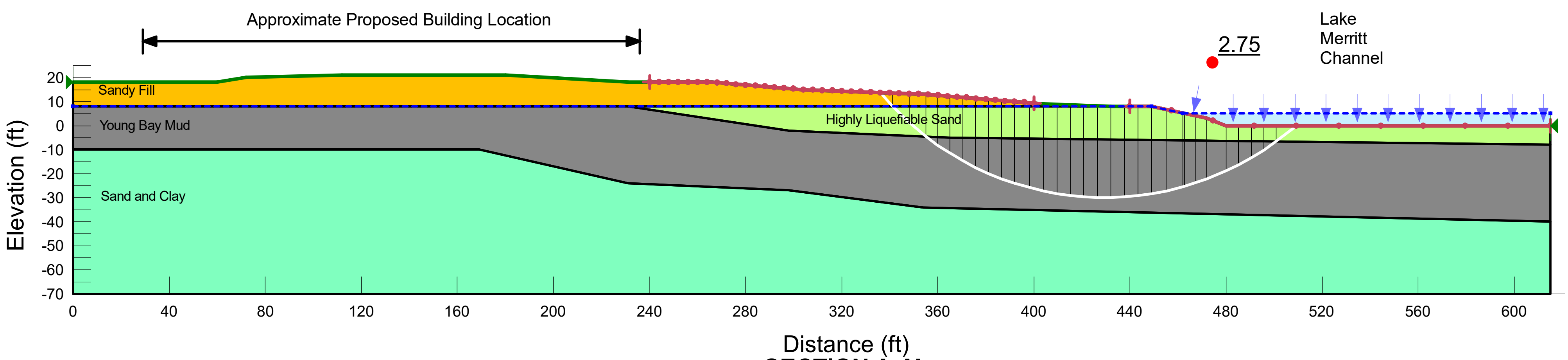
Elevation	Depth	Depth	ΔH (ft)	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N	$N_{1,60}$	σ_m'	σ_m'	$K_{2(max)}$	G_{max}	r_d	$r_{eff}*G_{eff}/G_{max}$	Sand	r_{eff}	r_{eff}	$\varepsilon_{C,N=15}$	$\varepsilon_{C,N}$	ΔS (in)
ft	*ft*	*m*	*ft*	*blow/ft*	*psf*	*kPa*	*psf*	*kPa*		*Y/N*						*psf*	*tsf*		*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

Appendix 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		



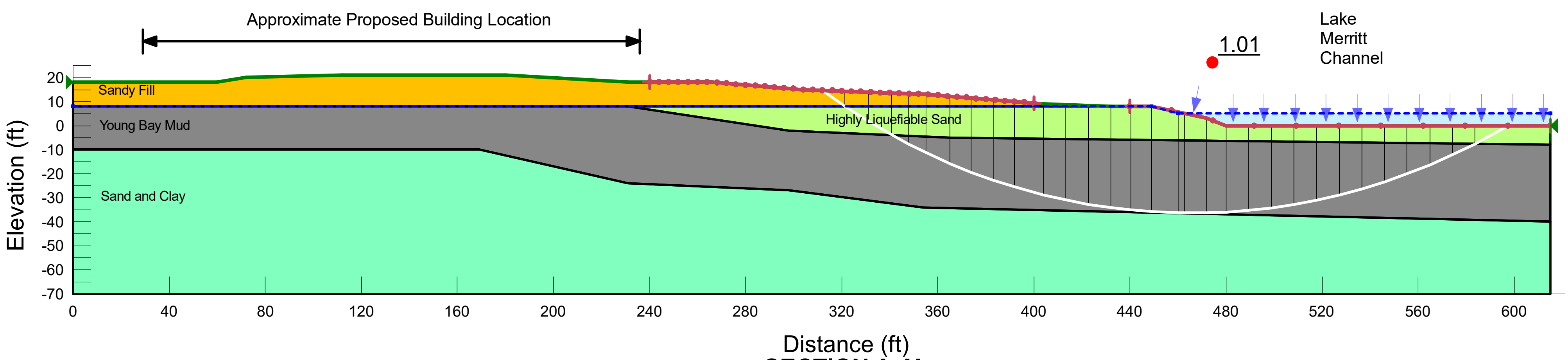
SECTION A-A'

PLATE F-1

FUGRO

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		



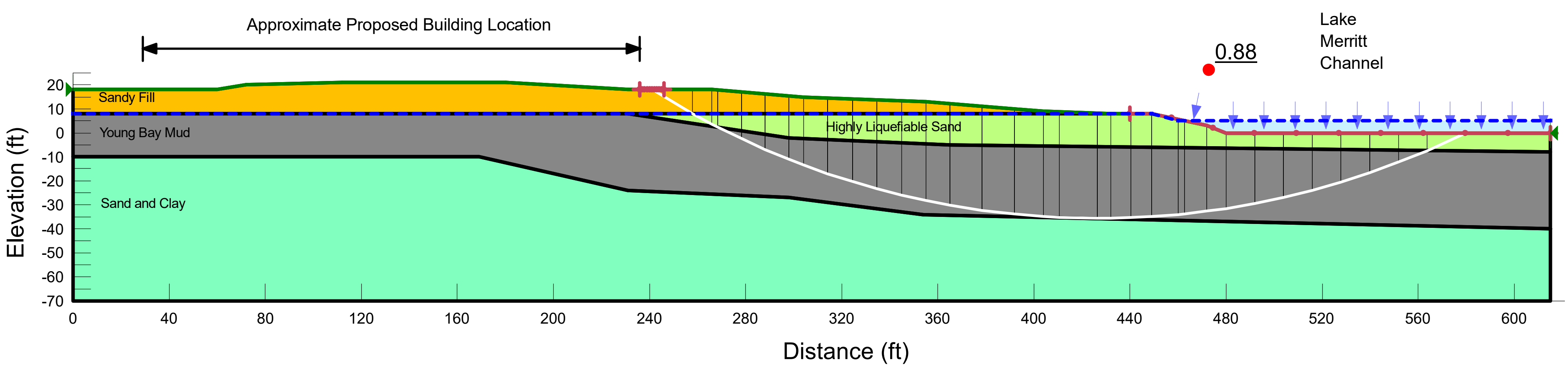
SECTION A-A'

PLATE F-2

FUGRO

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		



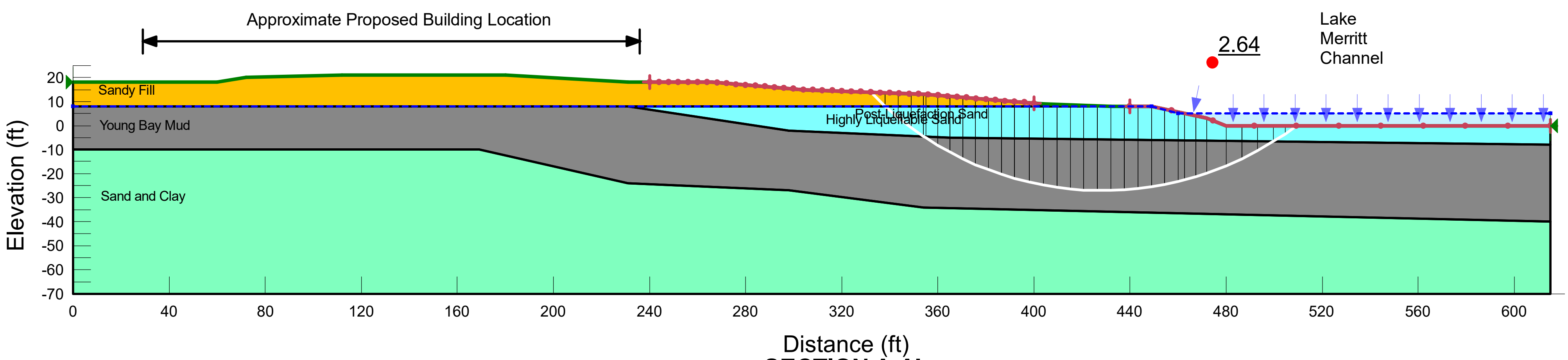
SECTION A-A'

PLATE F-3

FUGRO

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

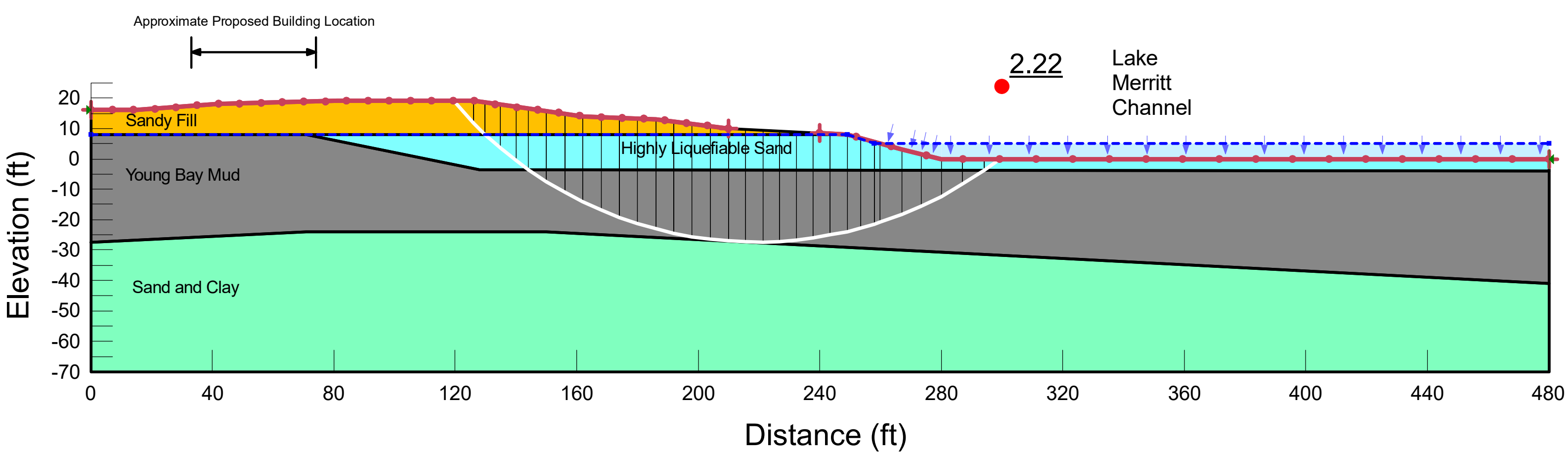


SECTION A-A'

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		

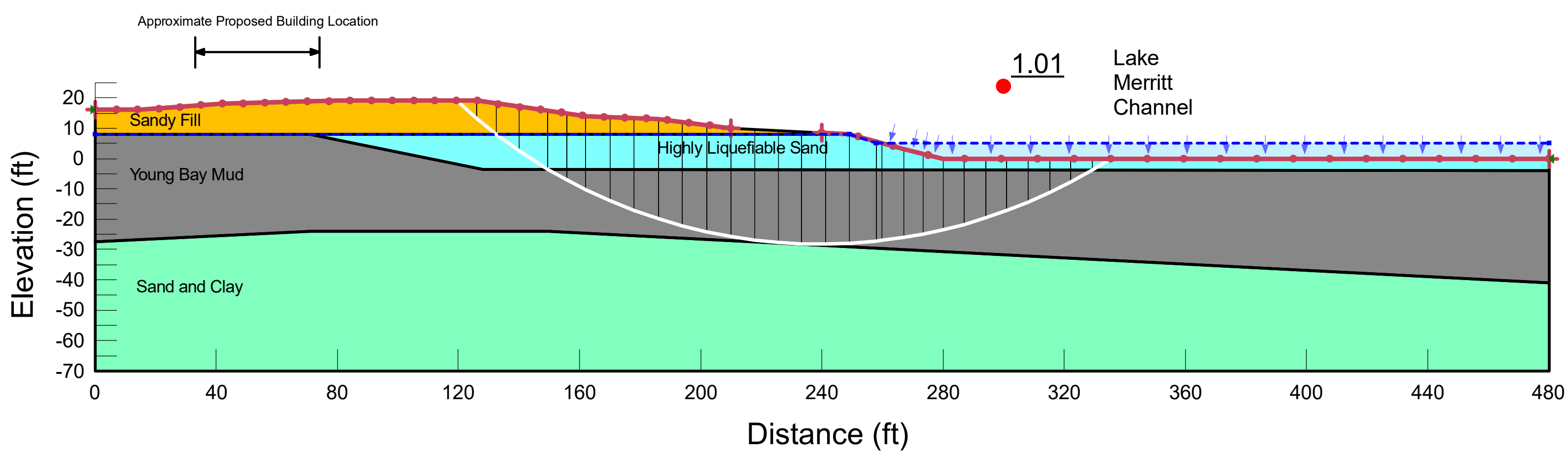


SECTION D-D'

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

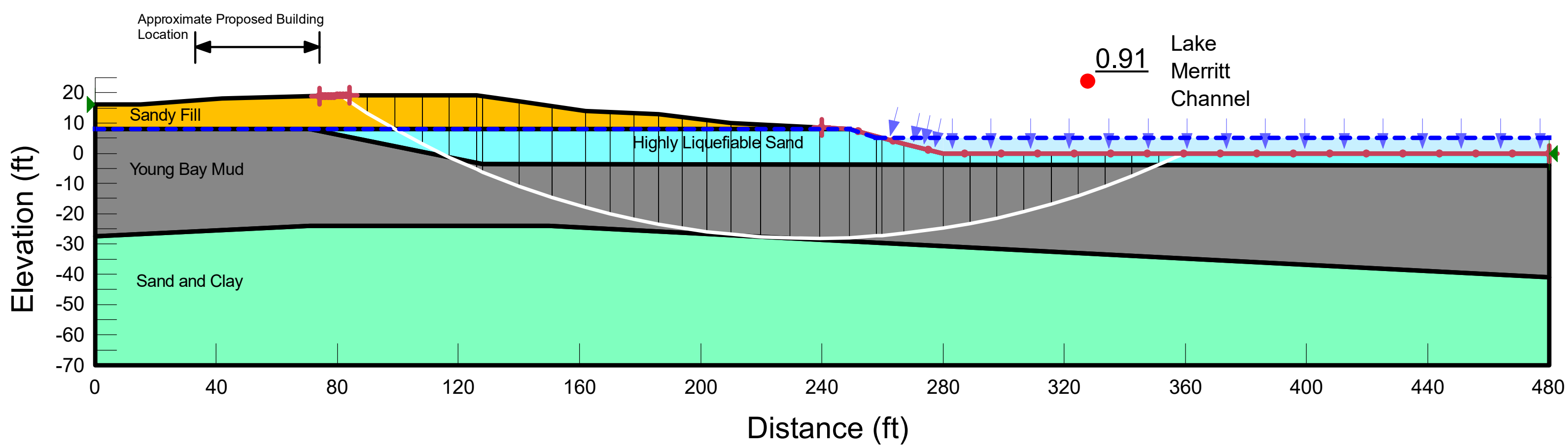
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		



SECTION D-D'

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		

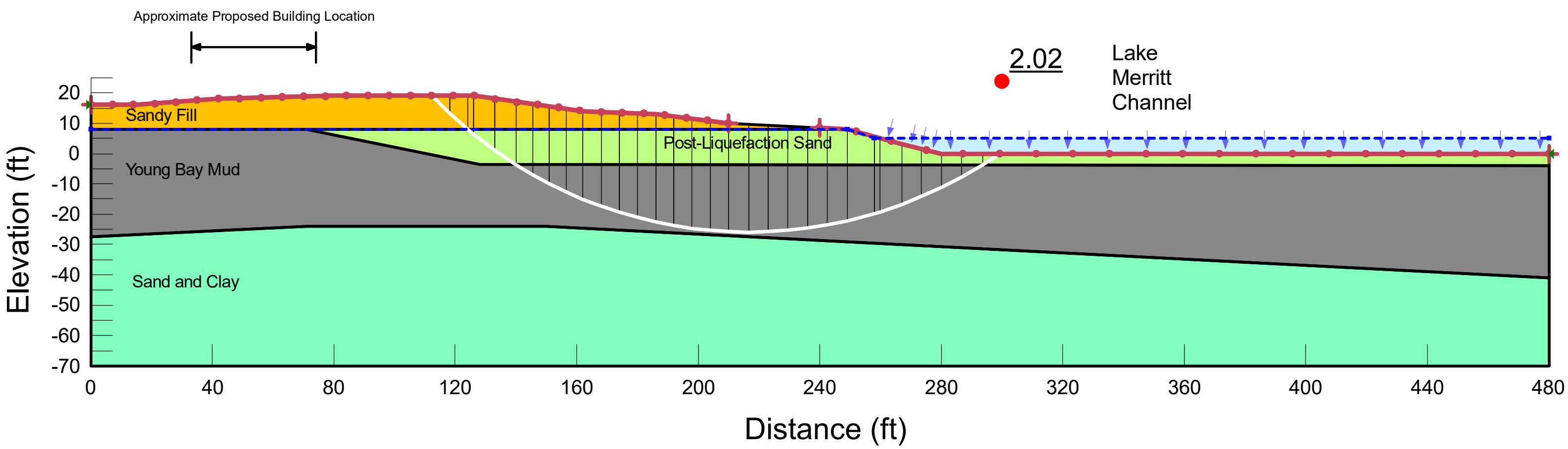


SECTION D-D'

FUGRO

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						

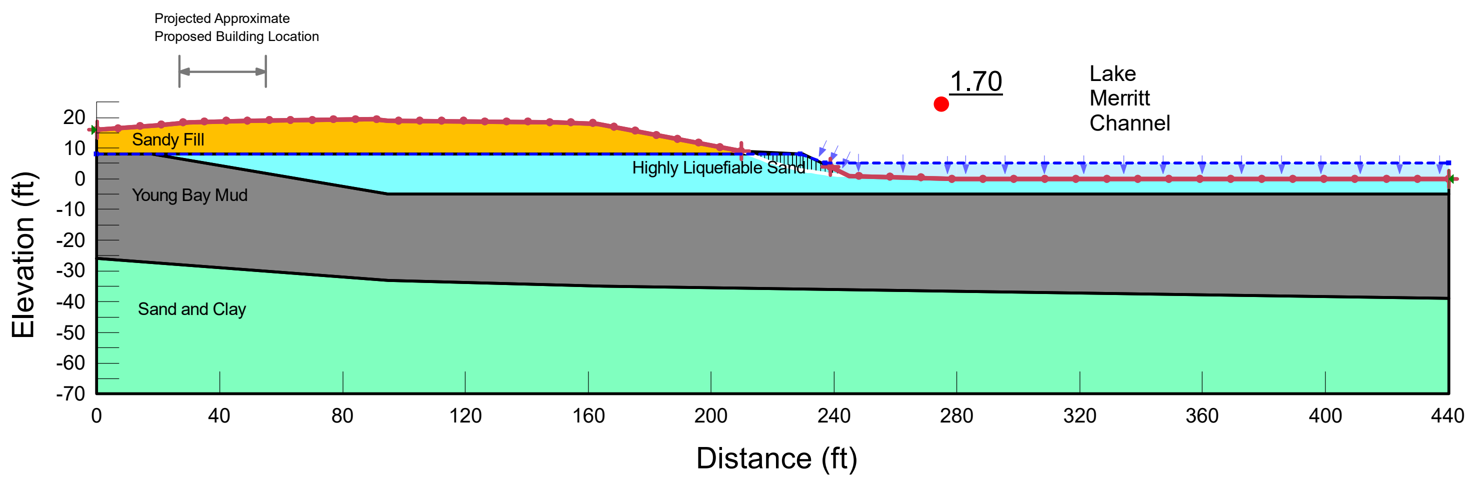


SECTION D-D'

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

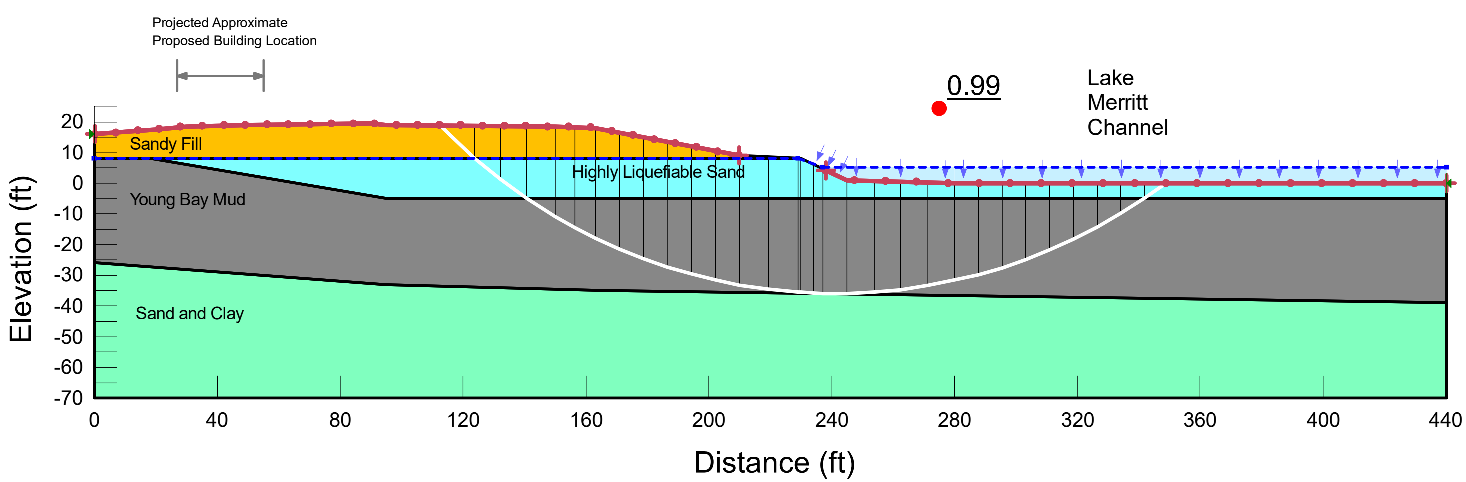
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		



SECTION E-E'

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

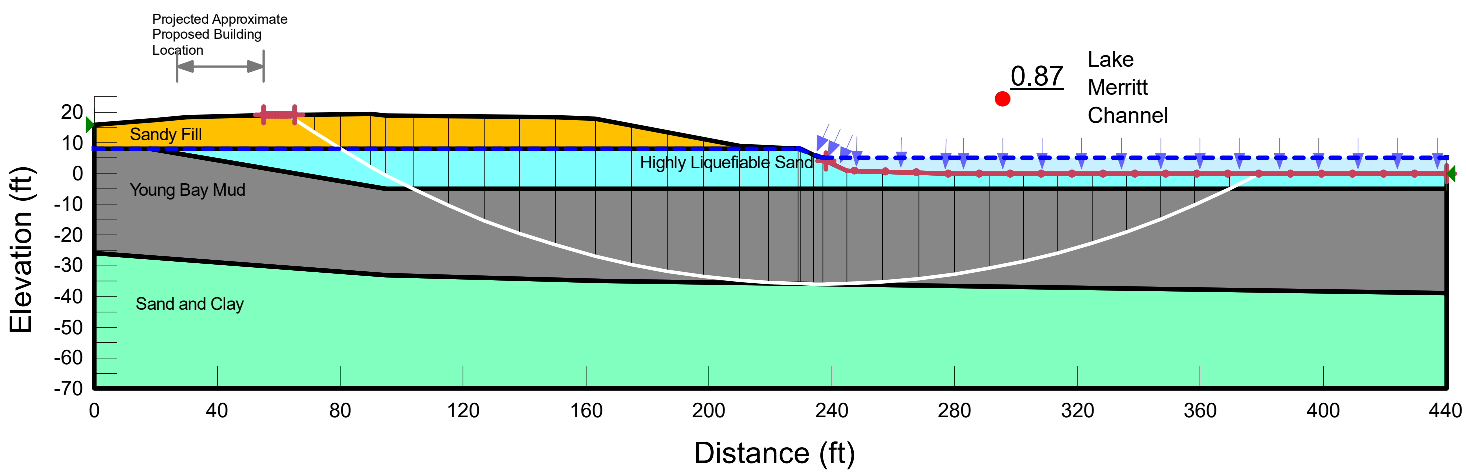
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line	Tau/Sigma Ratio	Minimum Strength (psf)
	Sandy Fill	Mohr-Coulomb	120	0	35	1		
	Young Bay Mud	S=f(overburden)	90			1	0.35	350
	Sand and Clay	Mohr-Coulomb	130	0	40	1		
	Highly Liquefiable Sand	Mohr-Coulomb	110	0	33	1		



SECTION E-E'

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		

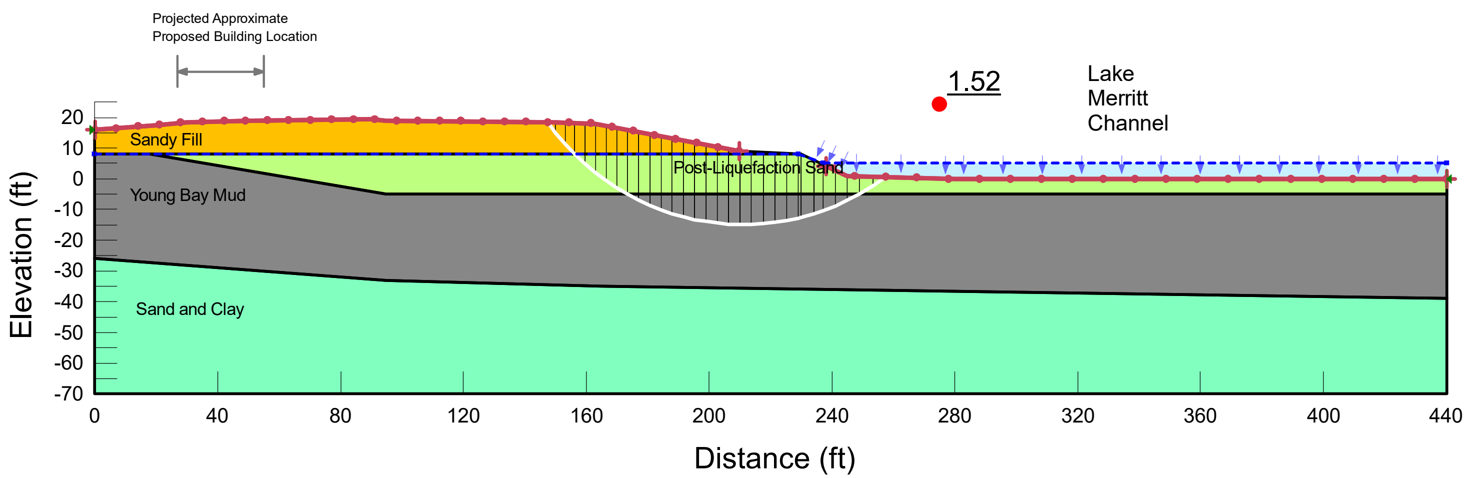


SECTION E-E'

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'

Appendix 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

G.3.2.1 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.

G.3.2.2 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.

G.3.2.3 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).

G.3.2.4 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).

G.3.2.5 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	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	0.945	0.711	0.555	0.782	0.782	0.400	0.521
0.03	0.957	0.970	0.717	0.559	0.789	0.789	0.459	0.526
0.05	1.07	1.08	0.783	0.611	0.861	0.861	0.518	0.574
0.075	1.32	1.34	0.930	0.724	1.02	1.02	0.591	0.680
0.1	1.55	1.57	1.07	0.831	1.17	1.17	0.664	0.781
0.15	1.83	1.86	1.29	1.01	1.42	1.42	0.811	0.946
0.190	2.01	2.04	1.42	1.11	1.56	1.56	0.927	1.04
0.2	2.05	2.08	1.45	1.13	1.60	1.60	0.927	1.06
0.25	2.23	2.32	1.57	1.26	1.77	1.77	0.927	1.18
0.3	2.36	2.49	1.66	1.36	1.91	1.91	0.927	1.28
0.4	2.42	2.62	1.75	1.48	2.08	2.08	0.927	1.39
0.5	2.35	2.61	1.74	1.50	2.11	2.11	0.927	1.41
0.75	1.96	2.25	1.50	1.34	1.89	1.89	0.927	1.26
0.949	1.70	2.00	1.33	1.22	1.73	1.73	0.927	1.15
1	1.65	1.95	1.30	1.20	1.69	1.69	0.880	1.13
1.5	1.19	1.46	0.983	0.942	1.33	1.33	0.587	0.885
2	0.924	1.16	0.783	0.770	1.09	1.09	0.440	0.724
3	0.606	0.789	0.538	0.548	0.773	0.773	0.293	0.515
4	0.429	0.572	0.383	0.400	0.564	0.564	0.220	0.376
5	0.320	0.435	0.283	0.301	0.425	0.425	0.176	0.283
7.5	0.177	0.240	0.140	0.149	0.210	0.210	0.117	0.140
8	0.159	0.216	0.124	0.131	0.185	0.185	0.110	0.124
10	0.110	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

G.6.1.1 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).

G.6.1.2 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)

G.6.1.3 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

G.6.2.1 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).

G.6.2.2 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

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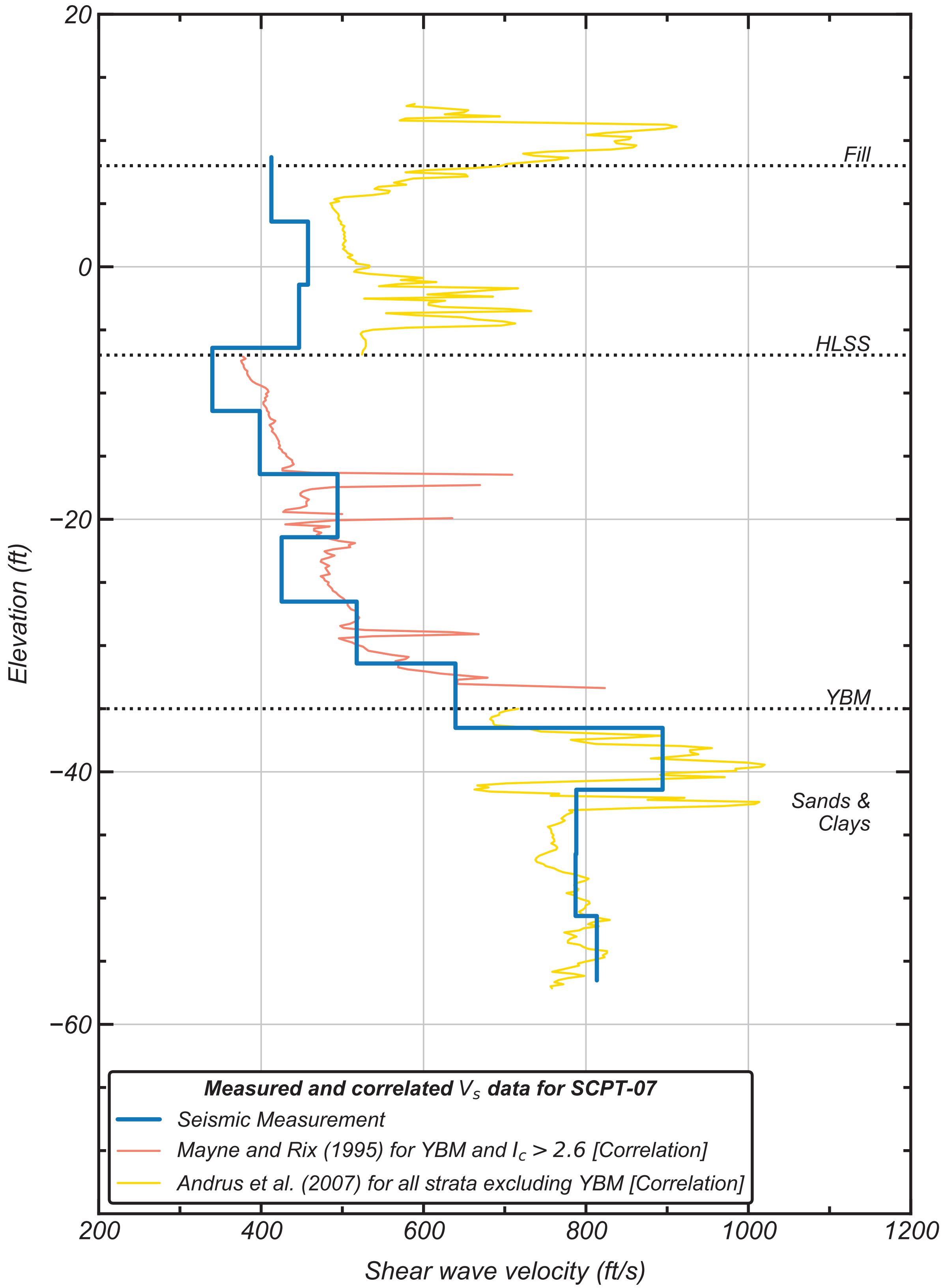

Figure G.2-1: Measured and Correlated Vs Data for SCPT-07

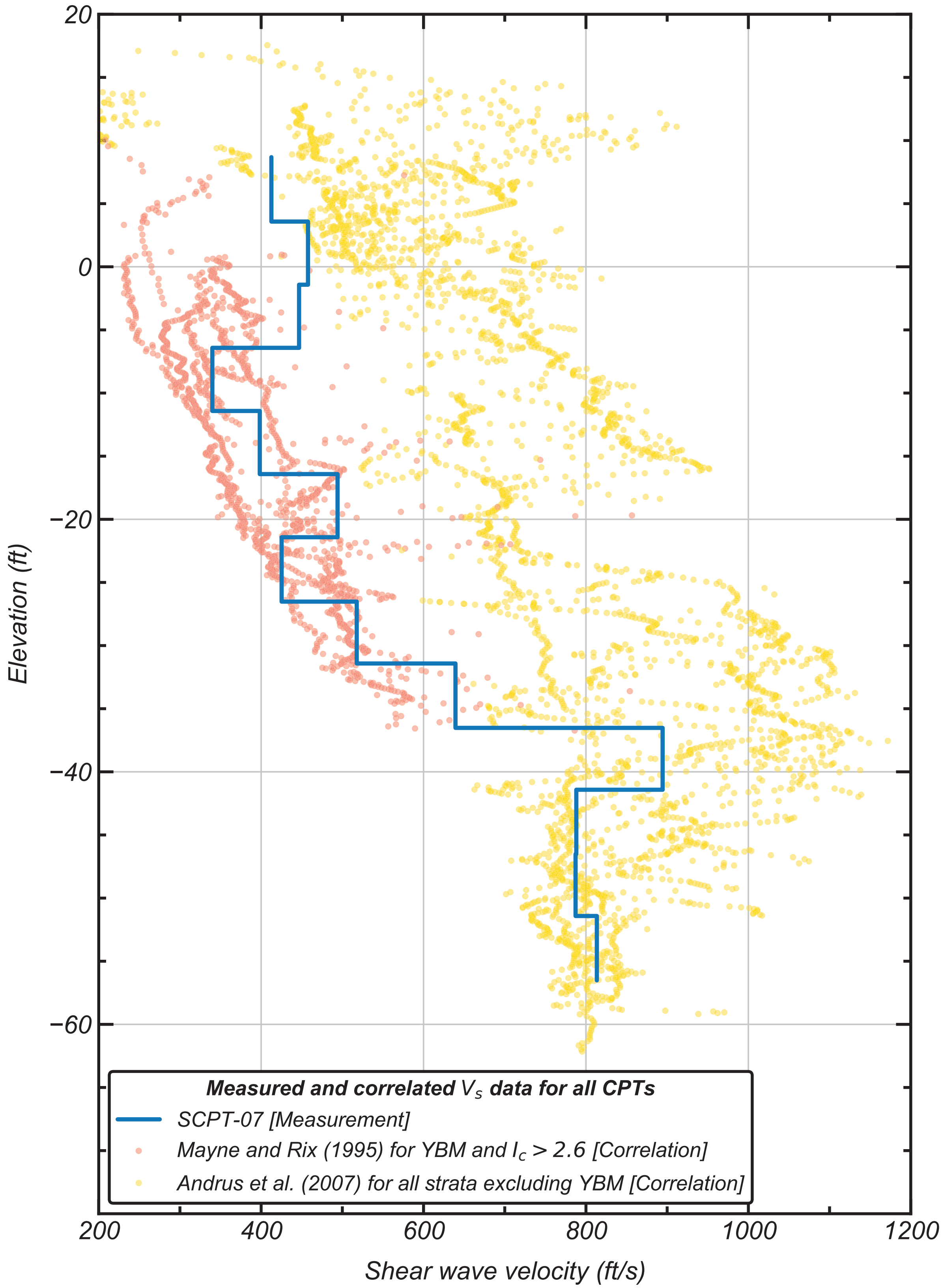

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Figure G.2-2: Measured and Correlated Vs Data for All CPTs

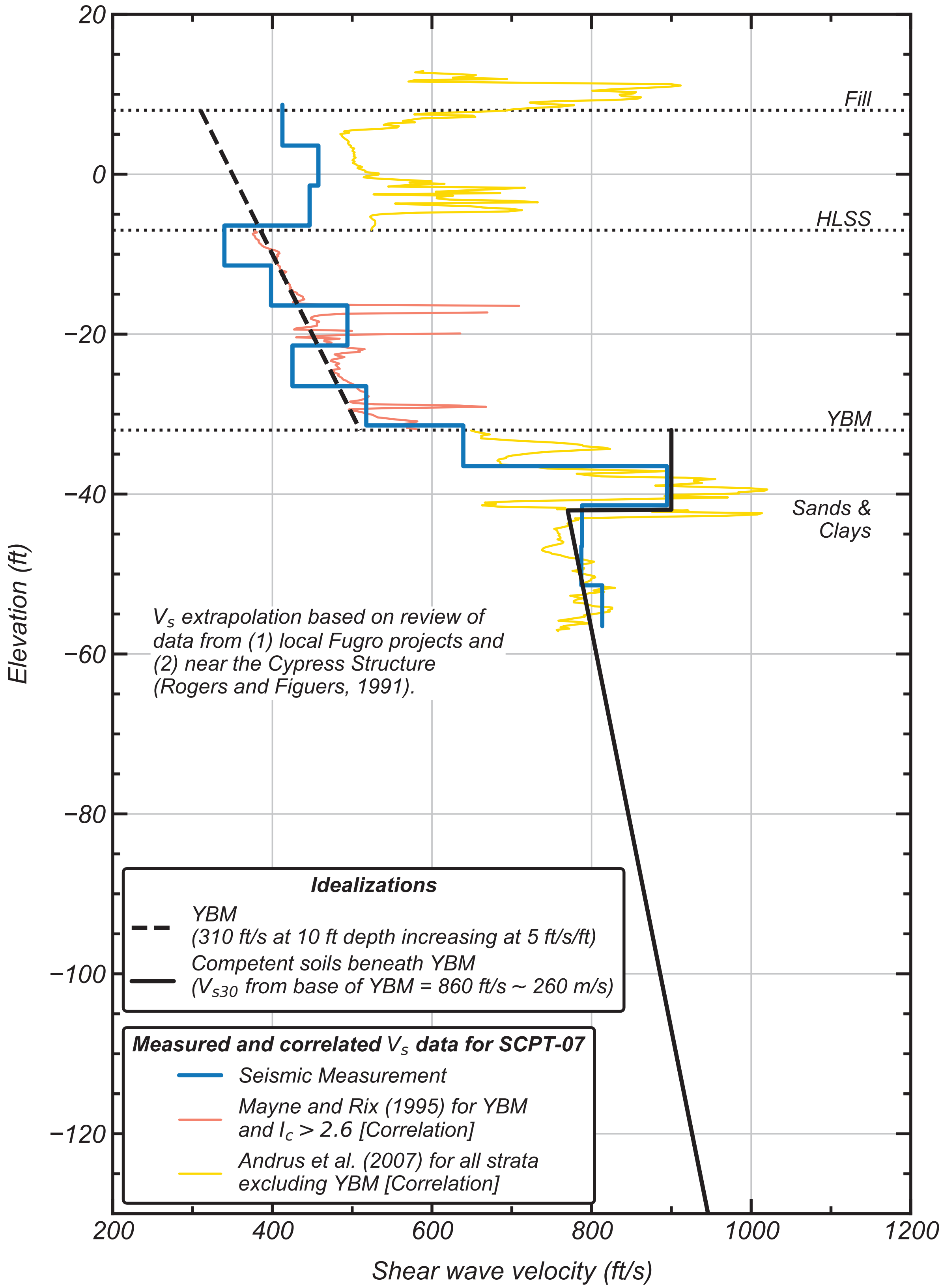

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Figure G.2-3: Shear Wave Velocity Idealizations

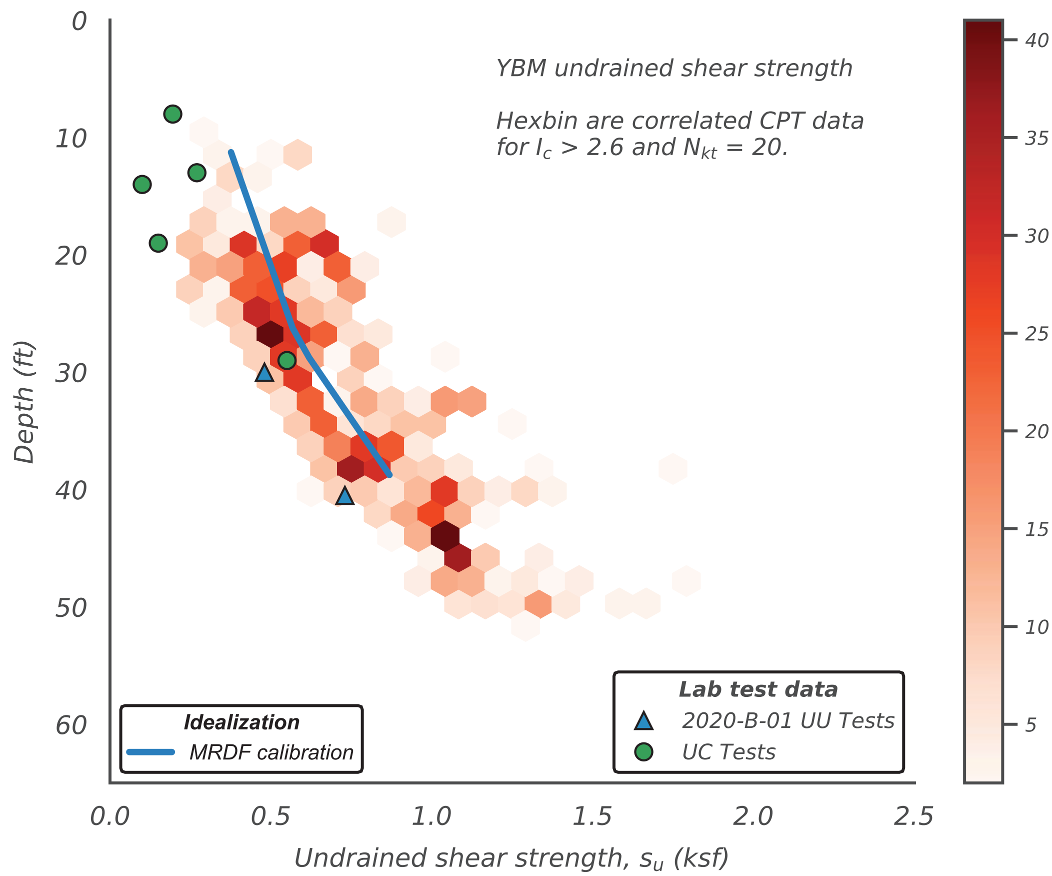


Figure G.2-4: YBM Undrained Shear Strength

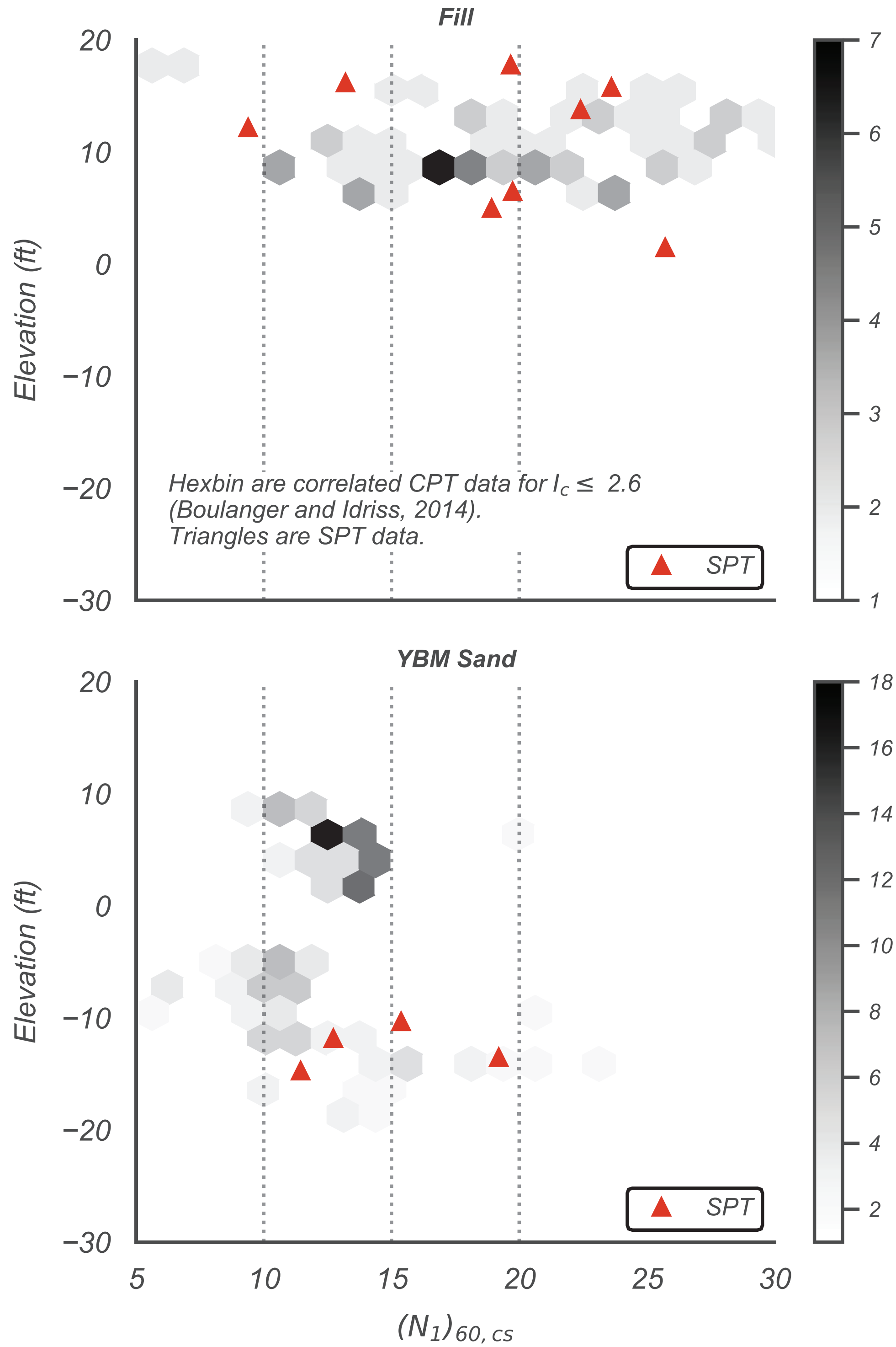


Figure G.2-5: Penetration Resistance (N160cs) vs. Elevation for Fill and YBM Sand

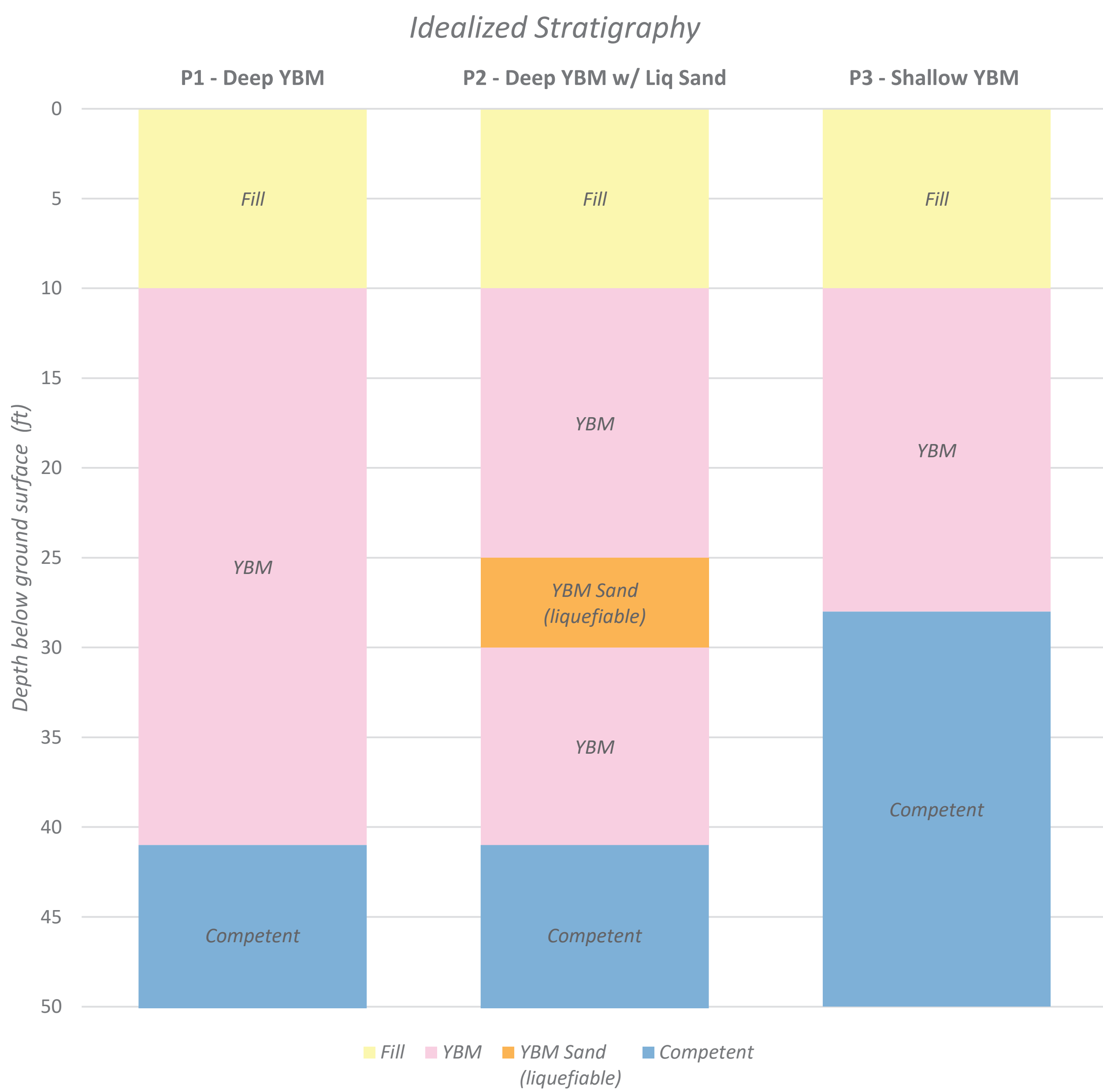


Figure G.2-6: Idealized Stratigraphy for 1D Site Response Analyses

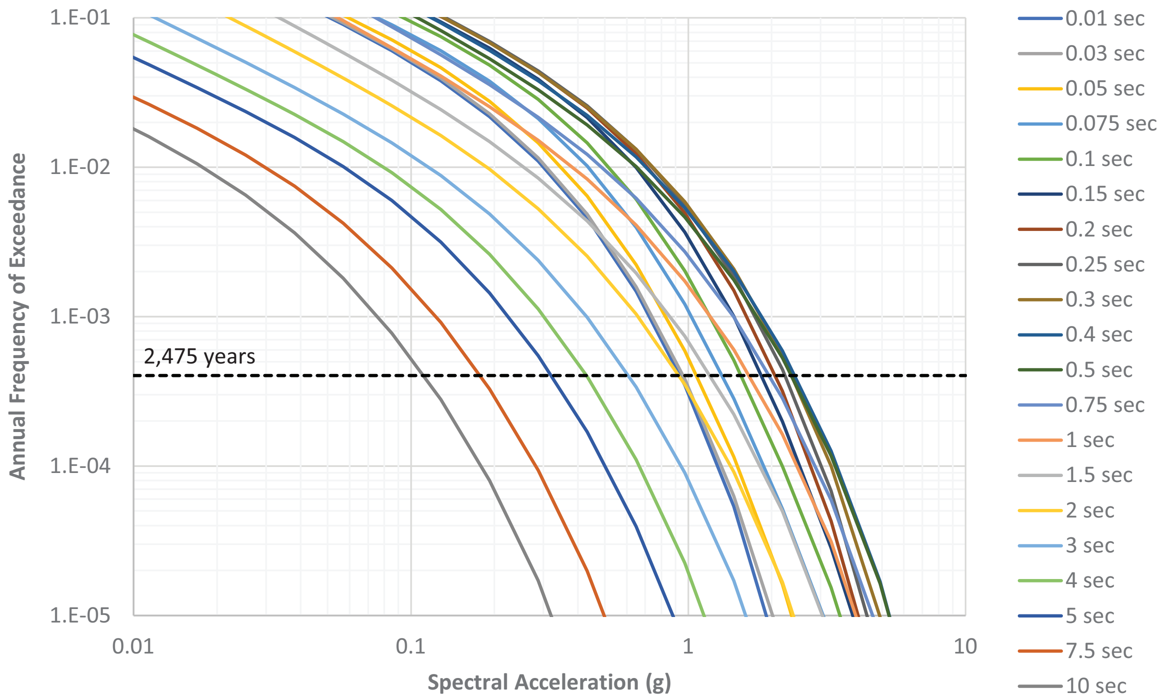


Figure G.3-1: Mean Annual Seismic Hazard Curves for Vs30 of 260 m/s (Base of YBM)

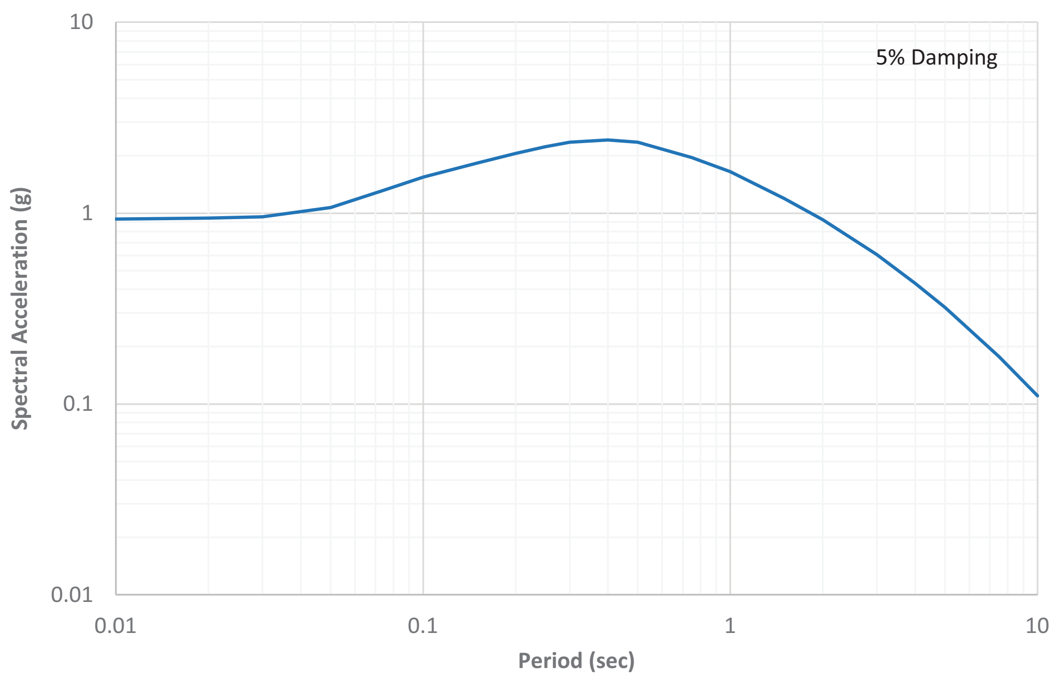
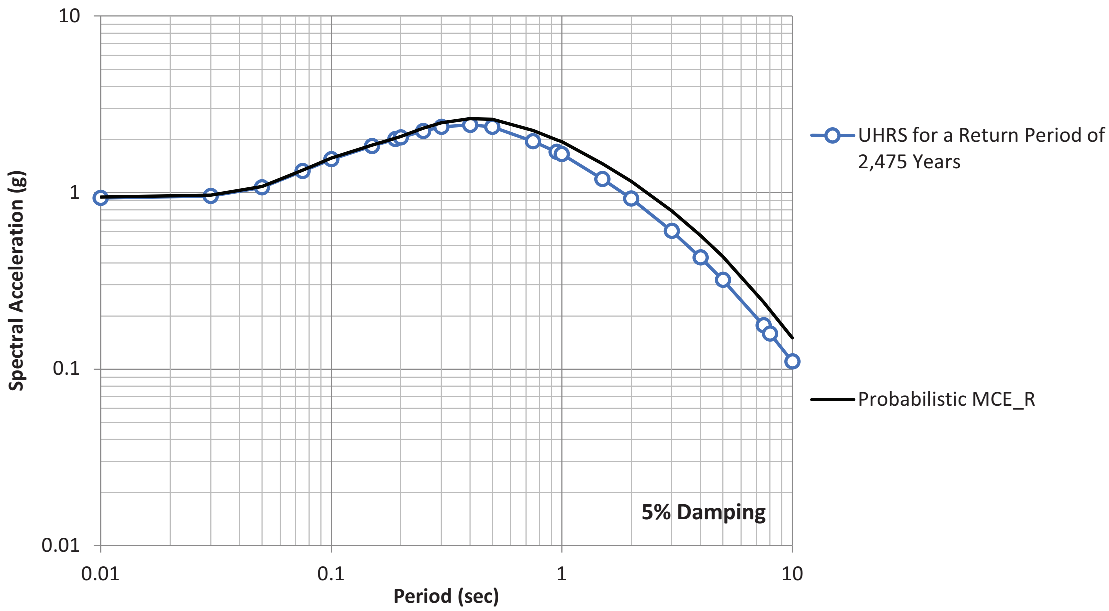


Figure G.3-2: Mean Horizontal Uniform Hazard Response Spectrum for a Return Period of 2,475 Years and Vs30 of 260 m/s (Base of YBM)



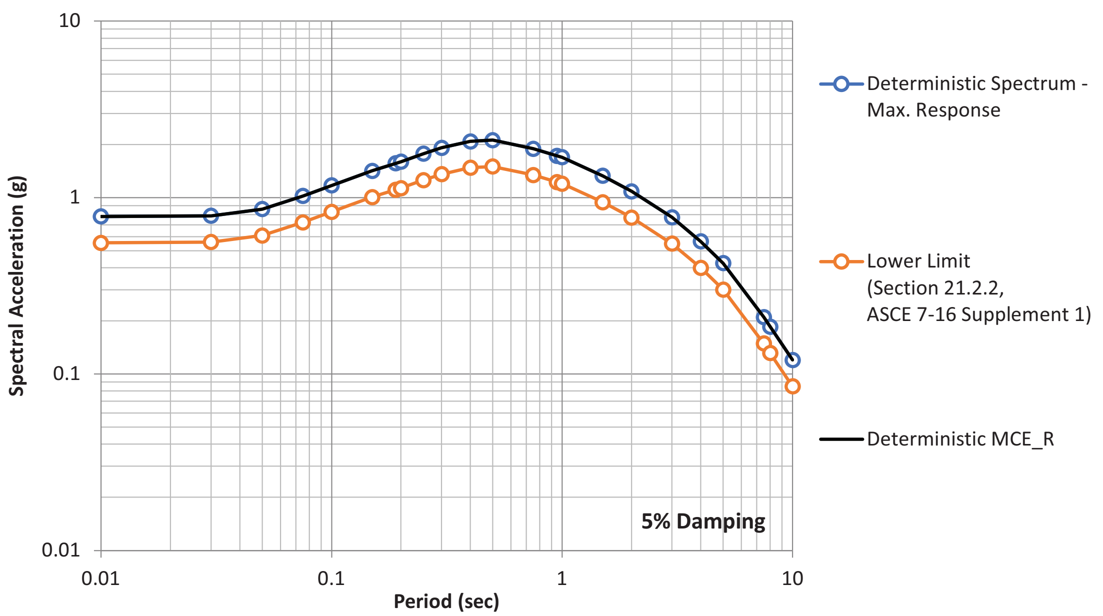
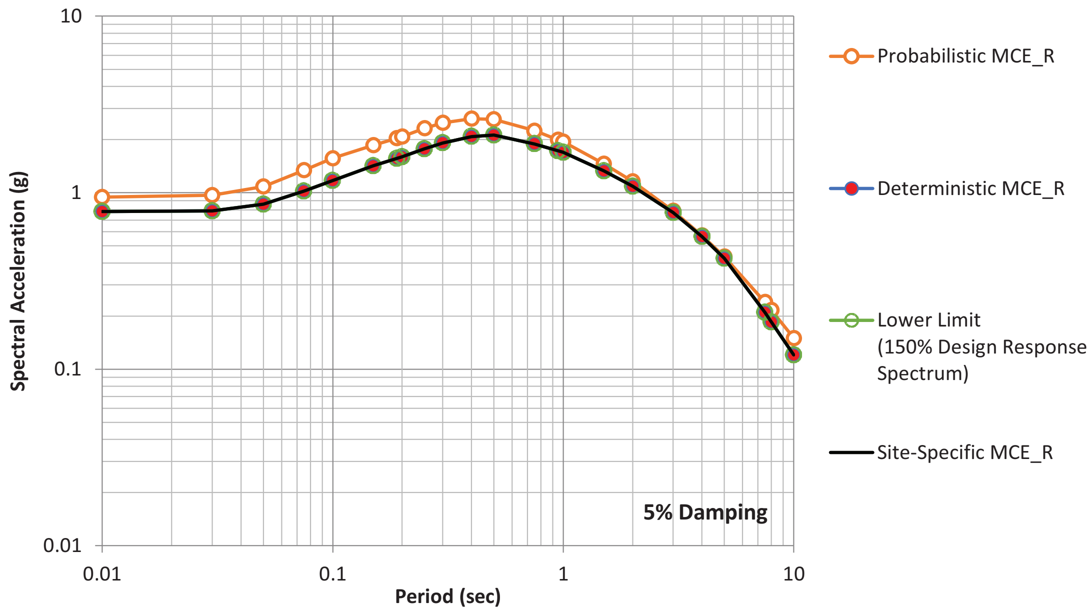


Figure G.4-1: Calculation of the Probabilistic and Deterministic Horizontal MCE_R Response Spectra per ASCE 7-16 for Vs30 of 260 m/s (Base of YBM)



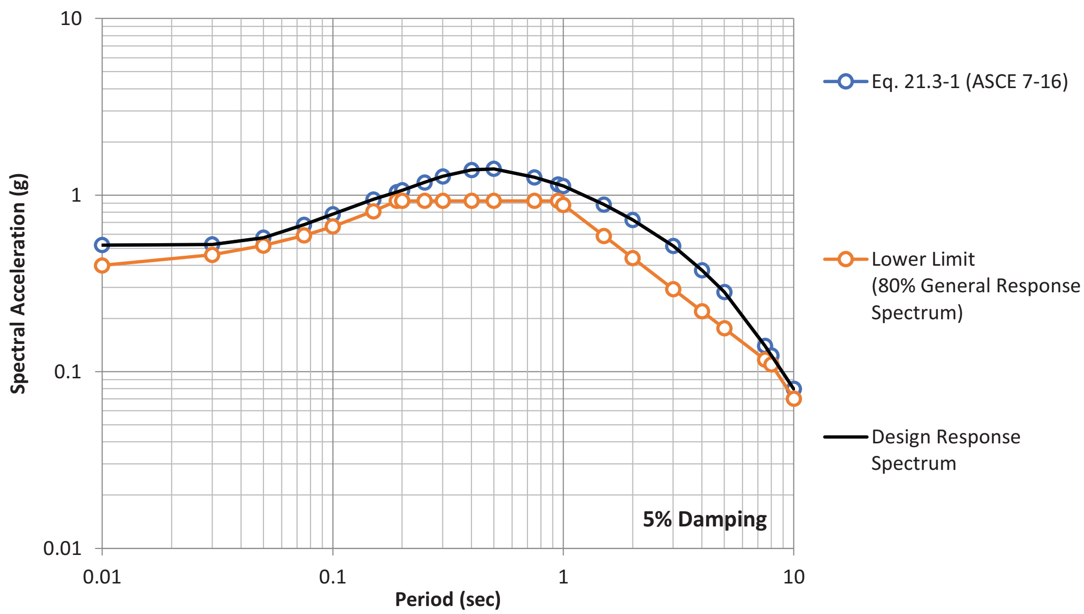


Figure G.4-2: Calculation of the Site-Specific Horizontal MCE_R and Design Response Spectra per ASCE 7-16 for Vs30 of 260 m/s (Base of YBM)

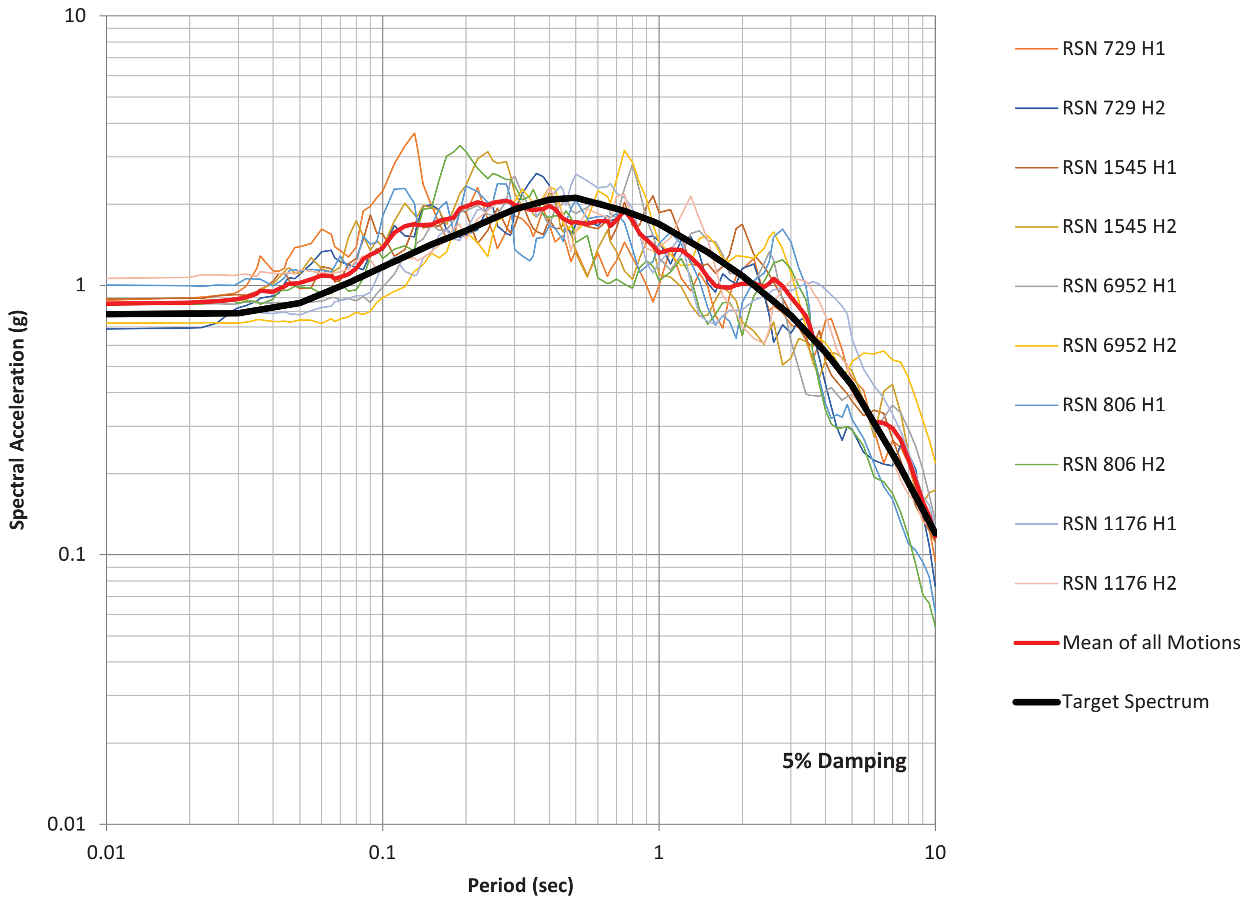


Figure G.5-1: Comparison of Target Response Spectrum (MCE_R), Mean of Scaled Response Spectra and Individual Scaled Ground Motions Response Spectra

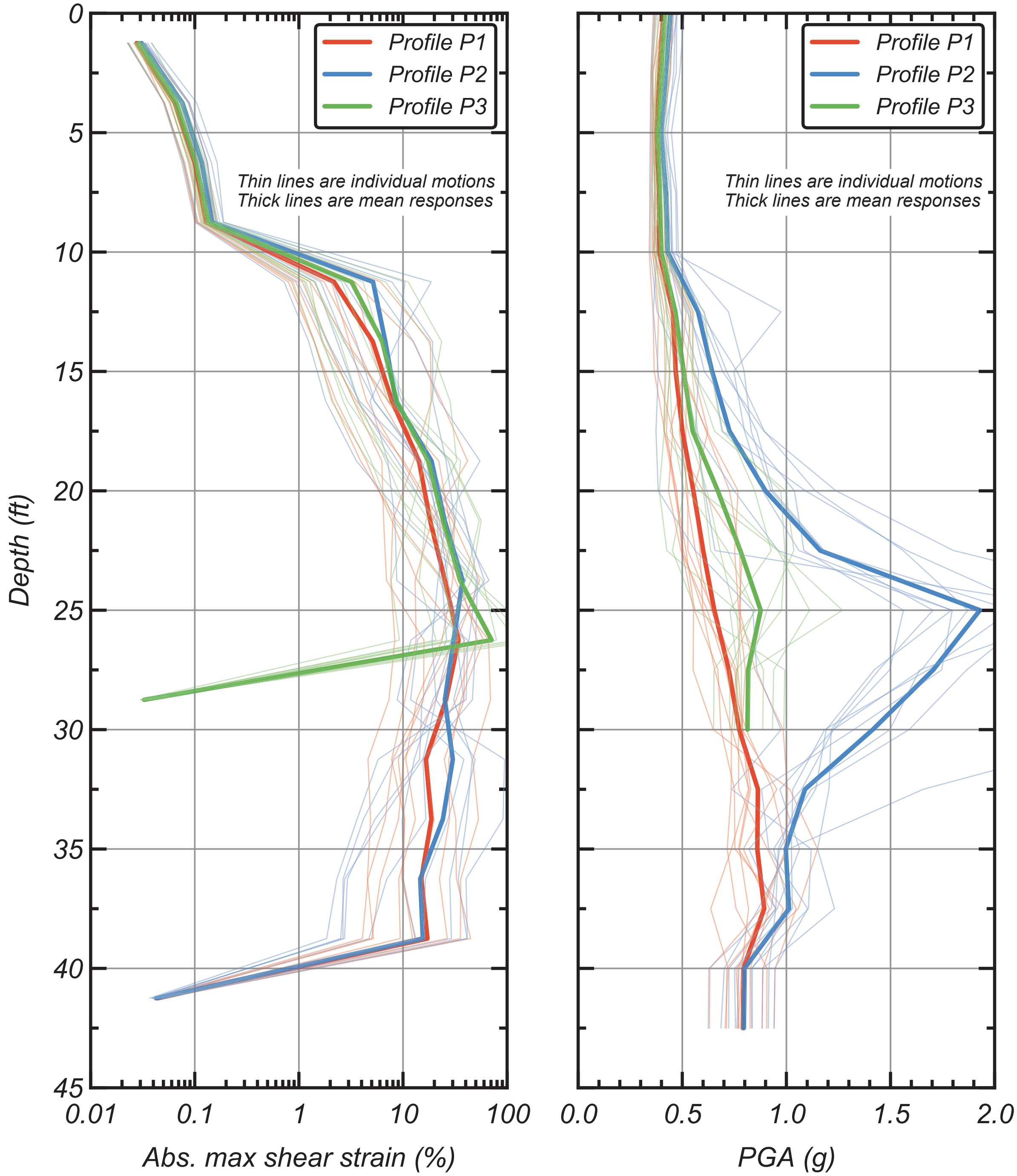


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Figure G.6-1: Profiles of Absolute Maximum Shear Strain and PGA from 1D Site Respone Analyses

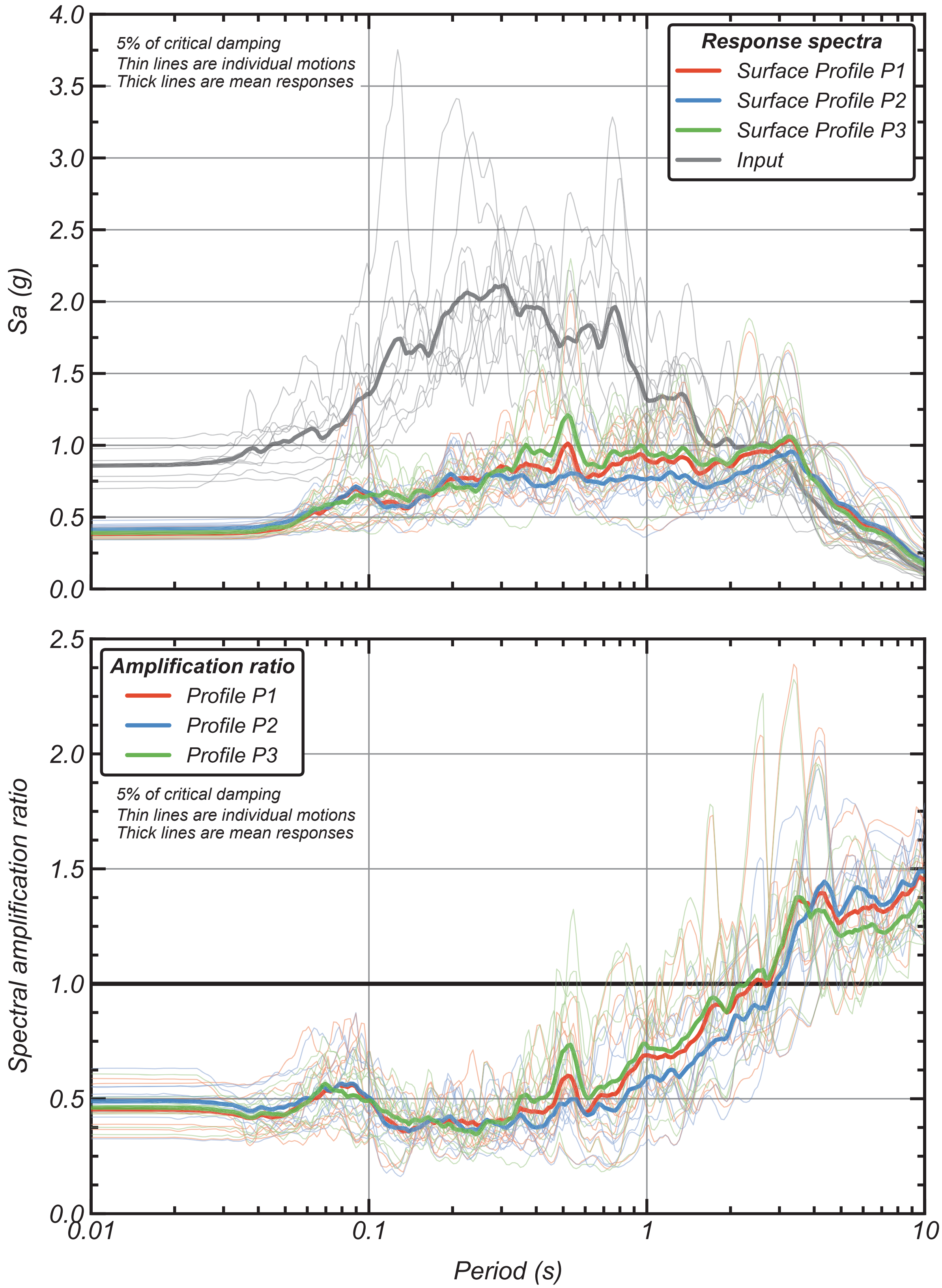


Figure G.6-2: Surface Response Spectra and Amplification Ratios from 1D Site Response Analyses

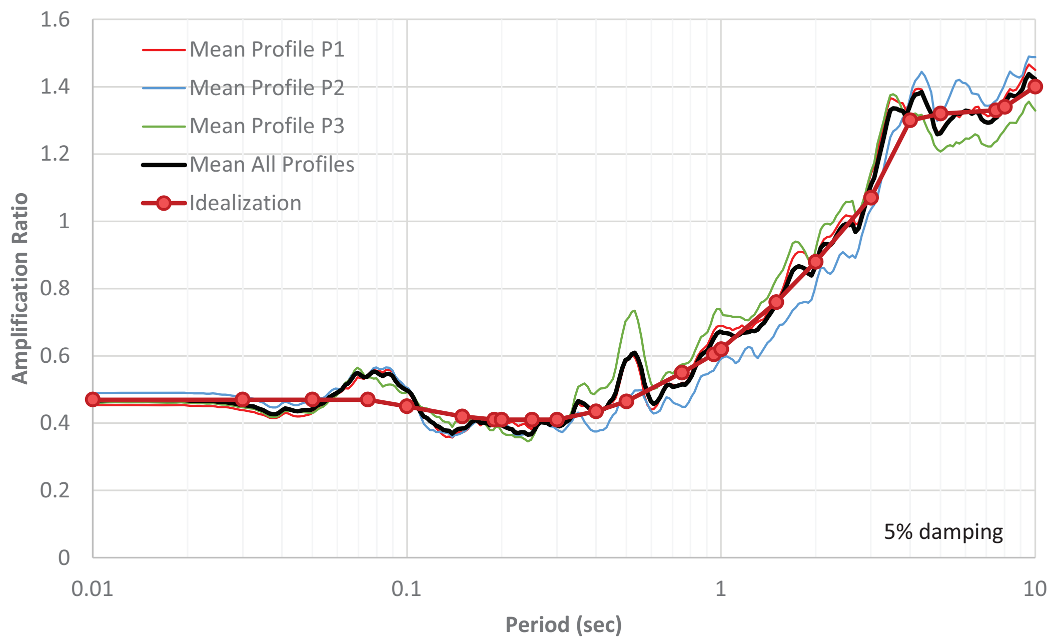
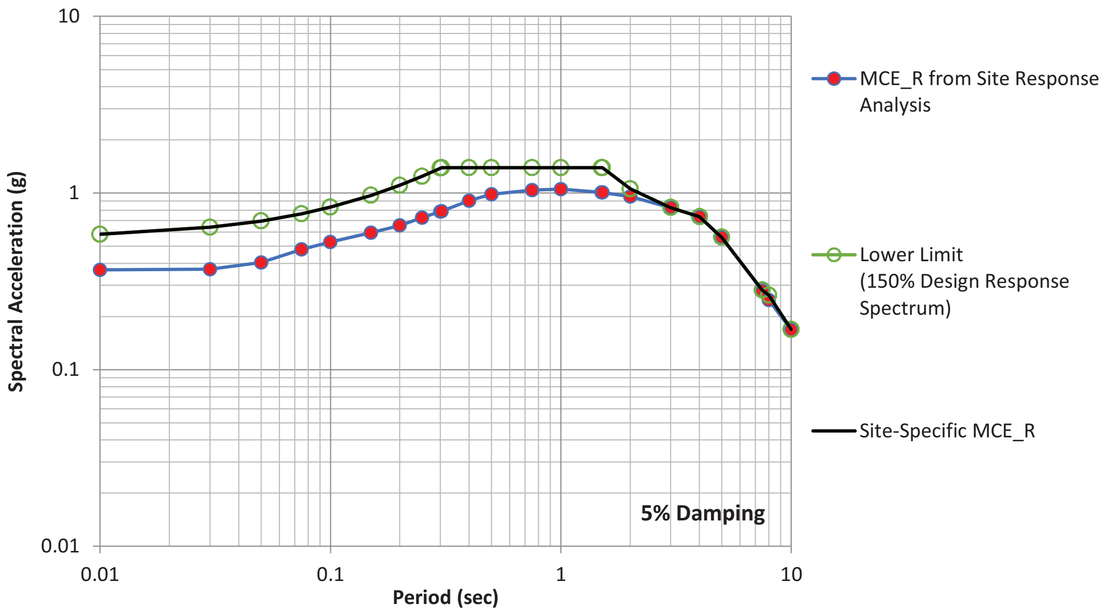


Figure G.6-3: Idealized Amplification Ratios



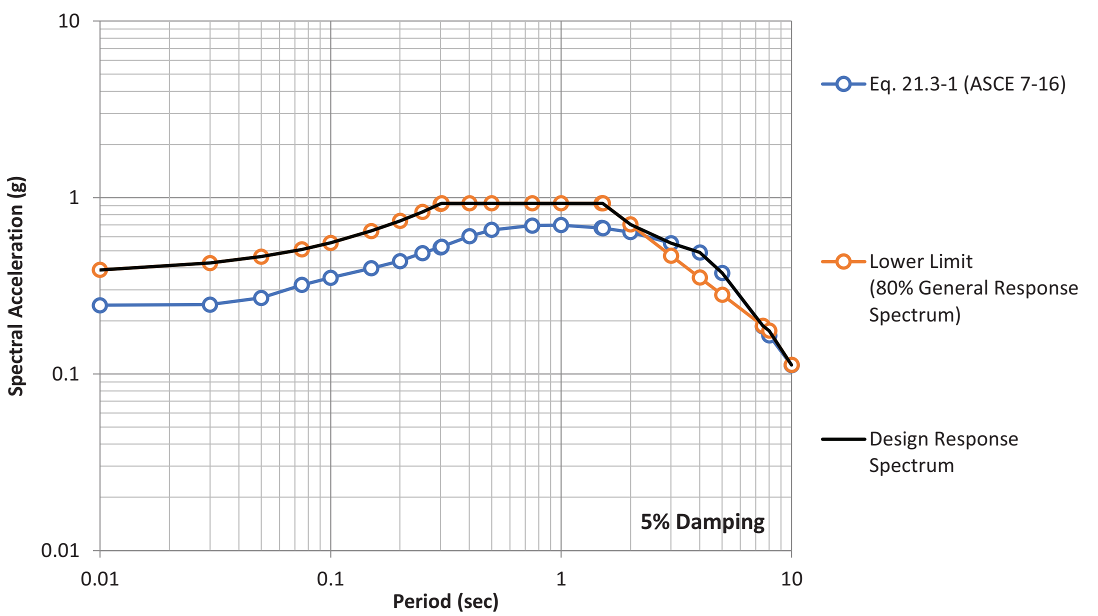


Figure G.7-1: Calculation of the Site-Specific Horizontal MCE_R and Design Response Spectra per ASCE 7-16 for the Ground Surface

Appendix H

LPILE Analyses

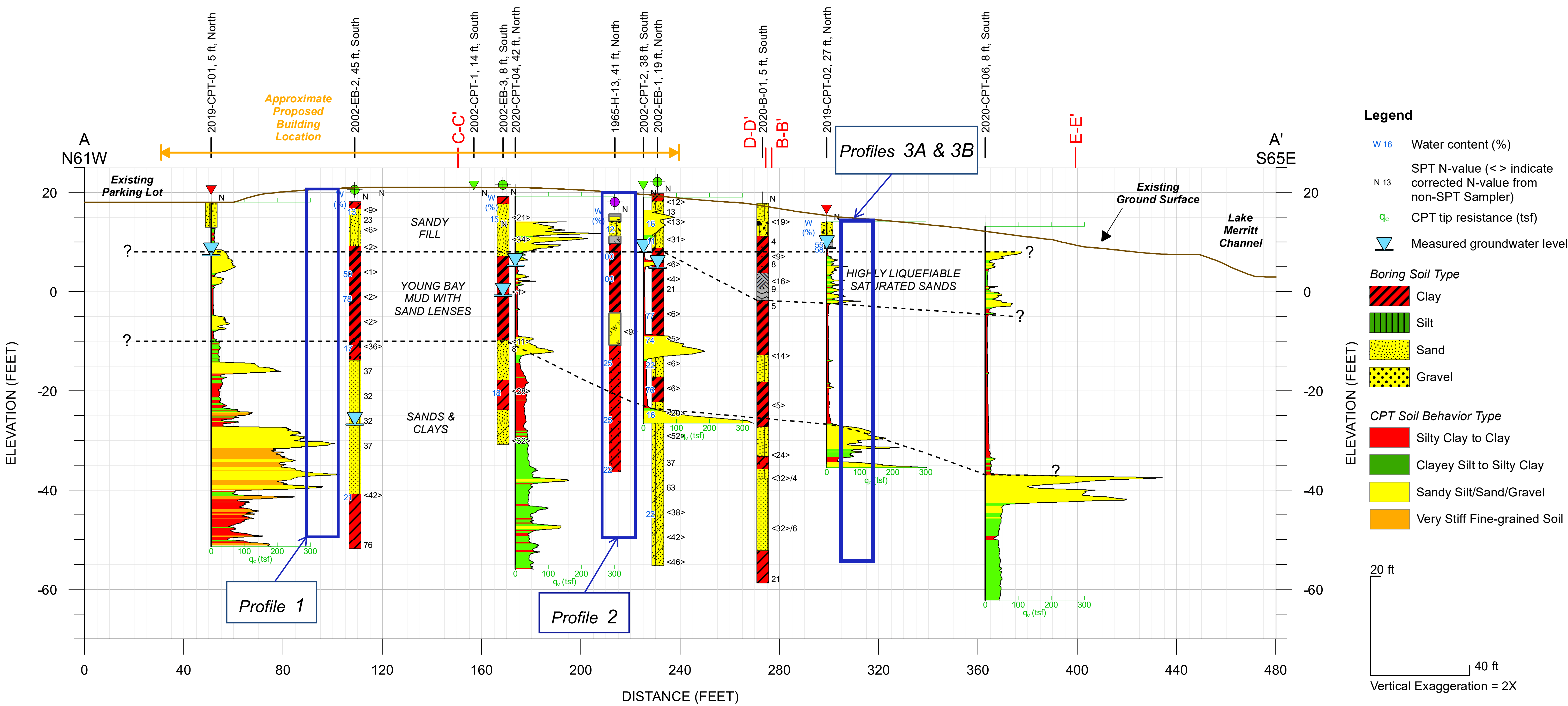


PLATE G-1: Profiles Along Cross Section A-A′

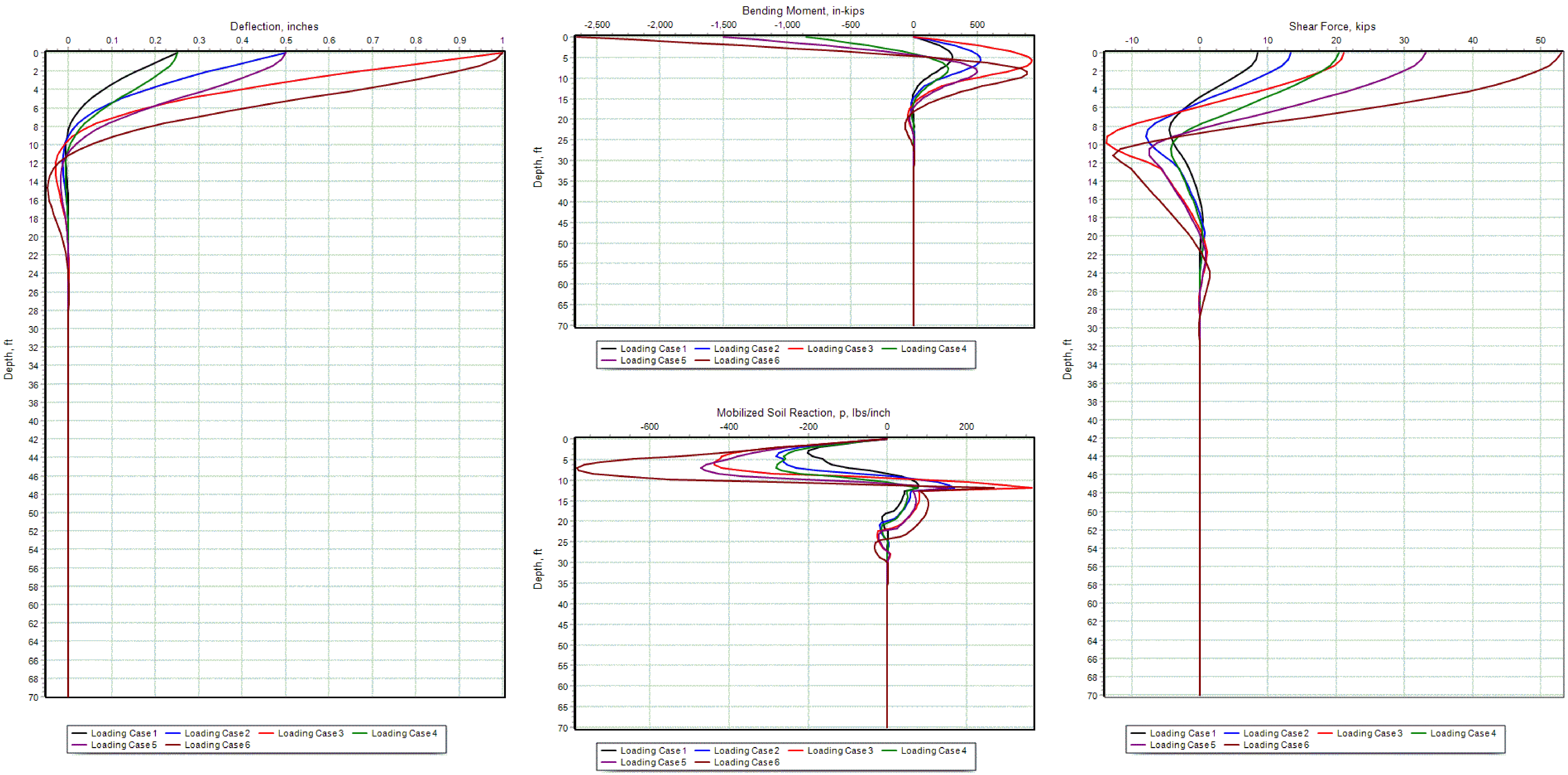


PLATE G-2: LPILE Results for Profile 1

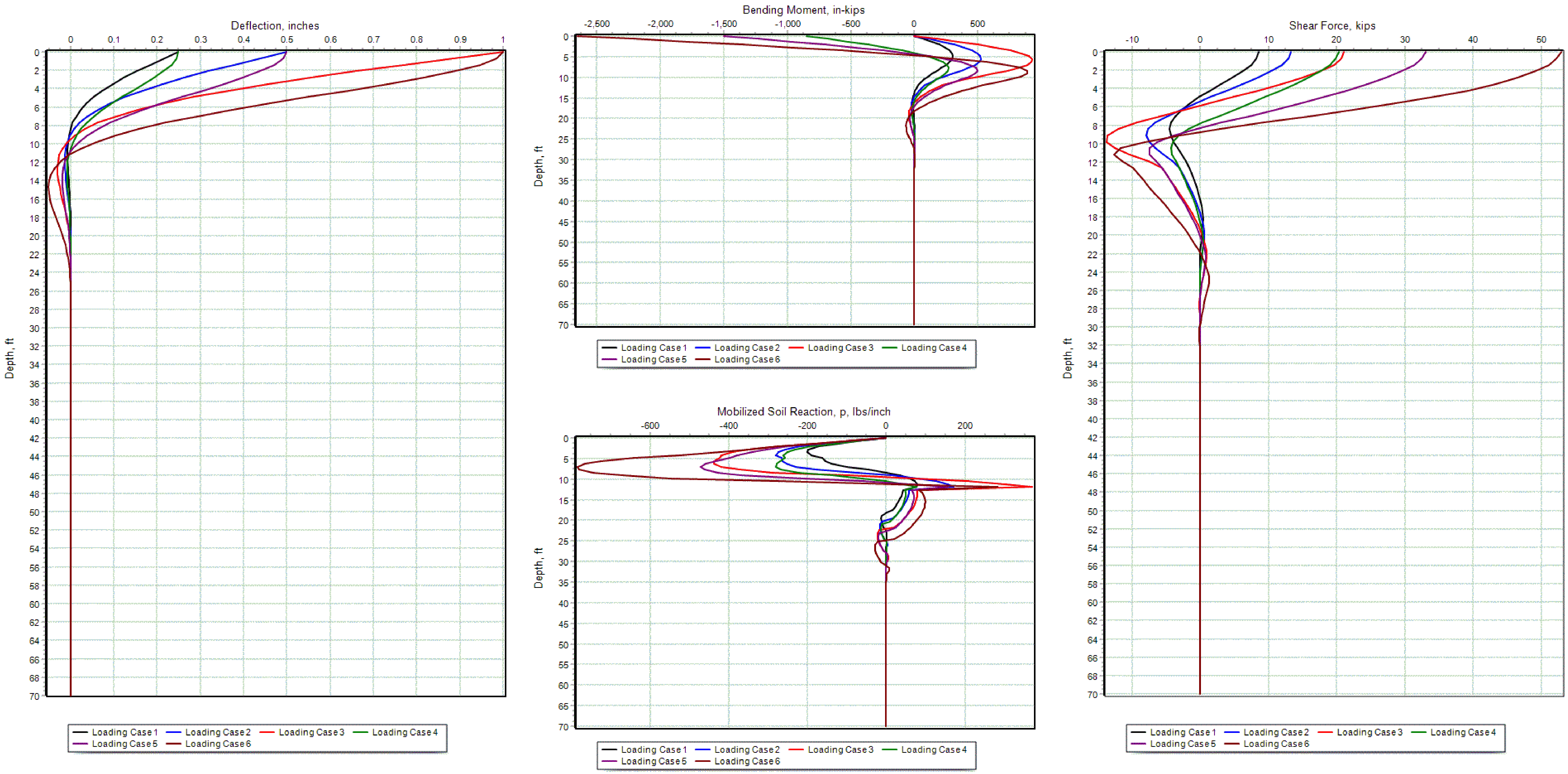


PLATE G-3: LPILE Results for Profile 2

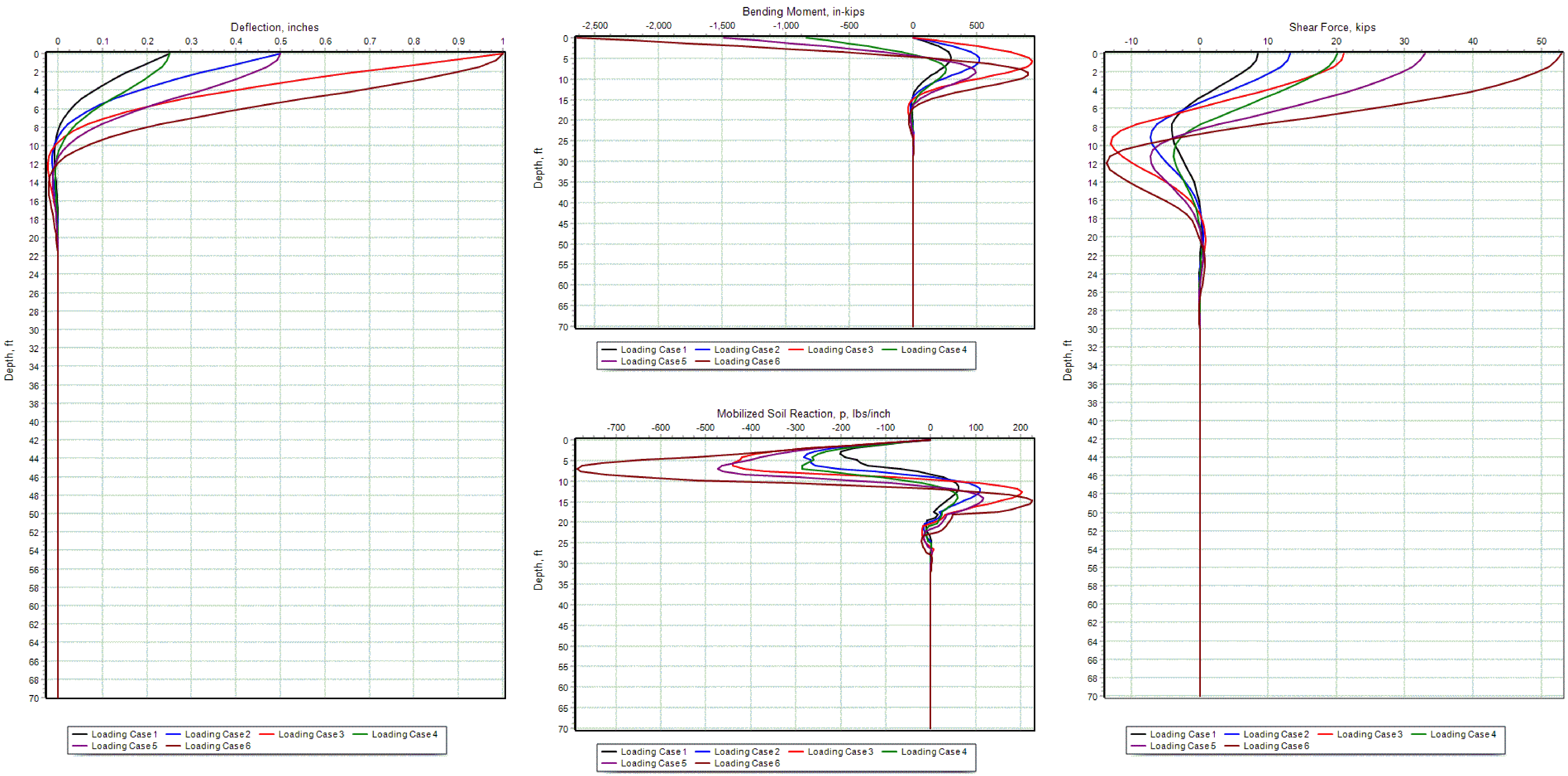


PLATE G-4: LPILE Results for Profile 3A

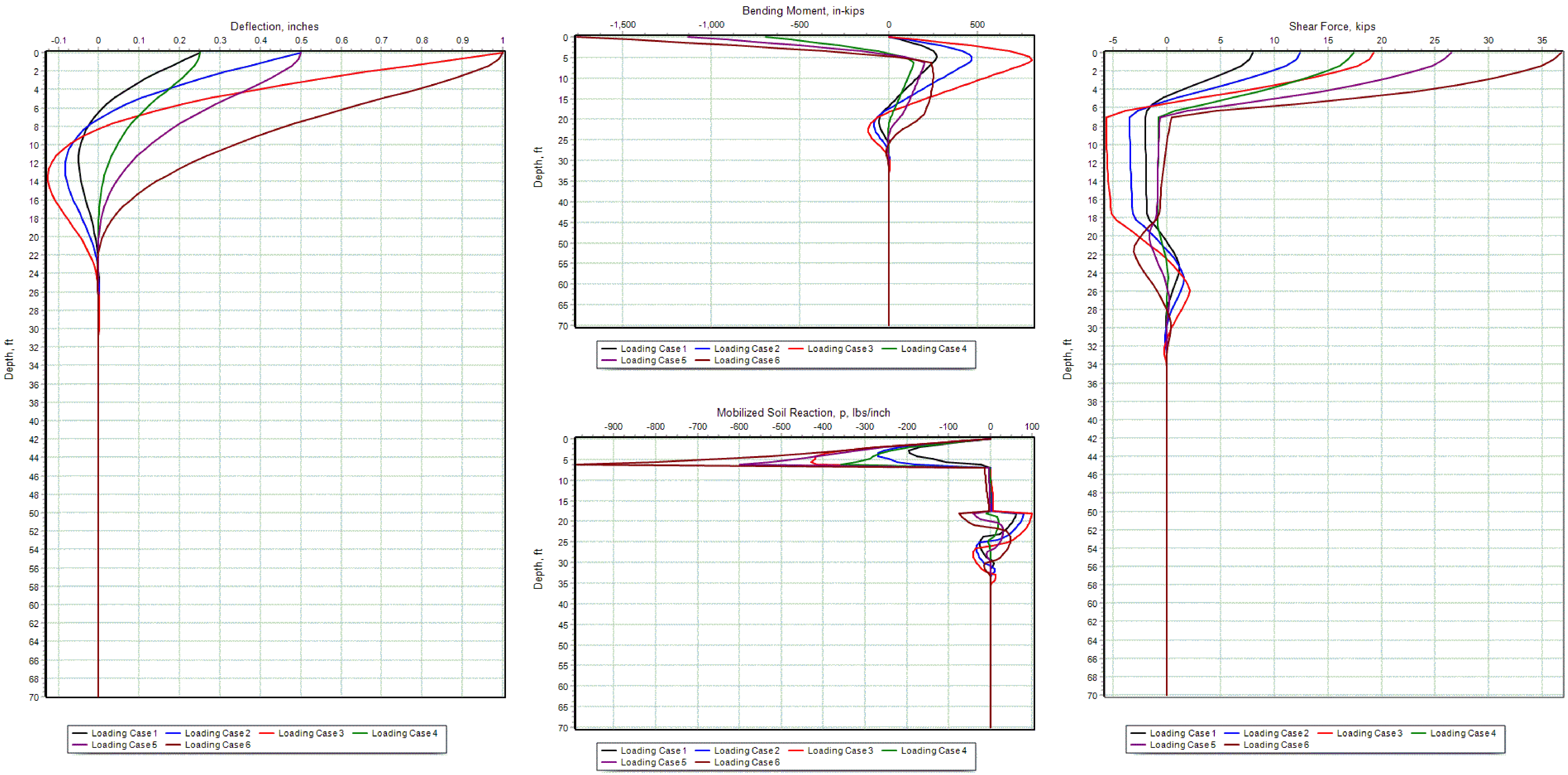


PLATE G-5: LPILE Results for Profile 3B