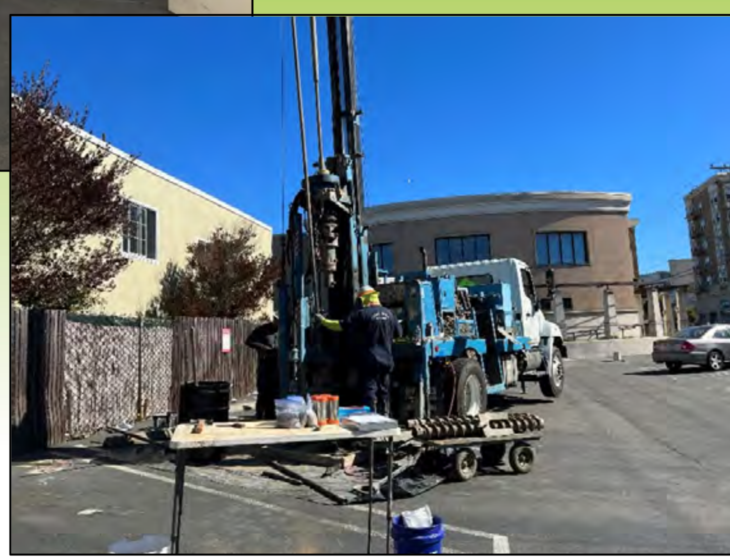
# Appendix G

Geotechnical Investigation Report

# Geotechnical Investigation Report

## 2550 Irving Street
## San Francisco, California

SUBMITTED TO:

Jackson Rabinowitsh
Project Manager
Tenderloin Neighborhood Development Corporation (TNDC)
jrabinowitsh@tndc.org

June 22, 2022

A3GEO

June 22, 2022

Jackson Rabinowitsh
Project Manager
Tenderloin Neighborhood Development Corporation (TNDC)
Email: jrabinowitsh@tndc.org

**RE: Geotechnical Investigation Report**
**2550 Irving Street**
**San Francisco, California**

Dear Mr. Rabinowitsh:

This report presents the results of A3GEO's geotechnical investigation for the project located at 2550 Irving Street in San Francisco, California. We provided our services under our August 17, 2020 Professional Services Agreement with the Tenderloin Neighborhood Development Corporation (TNDC) and the First Amendment to Professional Services Agreement dated November 29, 2021. We previously provided a due-diligence geotechnical evaluation report, dated September 18, 2020. The subsurface investigation and findings from our previous work have been incorporated into this design level geotechnical report.

The findings and conclusions presented in this report were developed in accordance with generally accepted geotechnical principles and practices at the time that the report was prepared. Should you have questions or comments concerning our findings, conclusions, or recommendations, please do not hesitate to call.

Once a design for the project is available, we can provide additional follow-up design services. We look forward to working with you in future project phases.

Sincerely,

**A3GEO, Inc.**

Timothy P. Sneddon, PE, GE
Principal Engineer
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# 1. INTRODUCTION

## 1.01 Overview

This report presents the results of a geotechnical investigation by A3GEO, Inc. (A3GEO) for the 2550 Irving Street project in San Francisco. We provided our services under our August 17, 2020 Professional Services Agreement with the Tenderloin Neighborhood Development Corporation (TNDC) and the First Amendment to Professional Services Agreement dated November 29, 2021. We previously provided a due-diligence geotechnical evaluation report, dated September 18, 2020. The subsurface investigation and findings from our previous work have been incorporated into this design level geotechnical report.

## 1.02 Project Description

The project will include demolishing the existing building at the site and constructing a new building of 7 stories in height. The project is in the early phase of design development. At this time, it is anticipated that the ground floor will consist of residential amenities, offices, and utility/equipment rooms while floor levels 2 through 7 will consist of residential units. A portion of the site will have below grade structures including parking stacker pits and an elevator. Other aspects of the proposed construction will include an at-grade backyard, a roof deck, and new landscaping. The lot size is approximately 19,125 square feet and the proposed footprint of the building is approximately 15,000 square feet.

## 1.03 Purpose and Scope of Services

The primary purpose of our services was to explore and characterize geotechnical, geologic, and seismic conditions at the site and prepare this report presenting data, conclusions, and recommendations for the design development phase of the project. The scope of services included:

- Reviewing reports, literature, maps, photographs, plans and other relevant information.
- Conducting geotechnical site reconnaissance visits.
- Exploring subsurface conditions with three cone penetration tests (CPTs), three exploratory borings, two exploratory hand-auger borings, and geophysical surveys.
- Performing geotechnical laboratory tests on samples of subsurface materials.
- Characterizing geotechnical, geologic, and seismic conditions at the site.
- Developing design ground motions in accordance with the 2019 CBC and ASCE 7-16.
- Conducting geotechnical engineering analyses.
- Developing geotechnical conclusions and recommendations for the design of the project.
- Preparing this geotechnical investigation report.

Please note that our scope was limited to aspects of the project that are geotechnical and/or geologic in nature. The scope of our services did not include an environmental assessment or investigation for the presence of hazardous, toxic, or corrosive materials on, below, or around the site.

## 1.04 Site Overview

The project site is located at 2550 Irving Street in San Francisco, California (Figure 1). The project site is located within San Francisco's Central Sunset District on the block bounded by 26th Avenue on the east, Irving Street on the south, 27th Avenue on the west, and Lincoln Way on the north (Figure 2). The City and County Assessor's Map shows that the site is on Sunset block 647, Lot 38. According to the Assessor's Map, the site measures 240 feet in the east-west direction, 90 feet in the north-south direction on the eastern portion of the site, and 60 feet in the north-south direction on the western portion of the site. The project site is bounded to the north by adjacent residential properties, to the east by 26th Avenue, to the west by 27th Avenue, and to the south by Irving Street (Figure 3). The site is currently occupied by an irregularly shaped building on the eastern portion of the site with an asphalt parking lot on the western portion of the site.

### 1.05 Elevation Datum

Based on information provided by the City of San Francisco Department of Public Works (SFDPW, 2022), the City of San Francisco updated their vertical datum to recover the North American Vertical Datum of 1988 (NAVD88) and establish a new vertical control network known as the San Francisco Vertical Datum of 2013 (SFVD13). The difference between the SFVD13 and the old City Datum varies slightly across the city, but a conversion factor recommended by SFDPW is 11.35 feet (SFVD13 {feet} – 11.35 feet = old City Datum in feet). For this report, elevation estimates are provided relative to the survey point data provided in the ALTA/NSPS Land Title Survey (TNDC, 2021), which we assumed were based on SFVD13 and NAVD88 elevation datum. Maps prepared by the United States Geological Survey (USGS), Google Earth, and others generally use elevation datums relative to NGVD-29, NAVD-88, or WGS84 EGM96. The following elevation conversions were applied for use in comparing various elevation data in our evaluation. The San Francisco Department of Public Works (2022) and other published sources of information indicate that SFVD13 datum is:

1. *11.35 feet above the old City datum*
2. *0 feet above the North American Vertical Datum of 1988 (NAVD 88)*
3. *2.7 feet above the National Geodetic Vertical Datum of 1929 (NGVD 29)*
4. *2.7 feet above the WGS84 EGM96 datum (used by Google Earth)*

# 2. METHODS OF INVESTIGATION

## 2.01 Review of Existing Information

We reviewed a variety of materials containing information relevant to the geologic and seismic setting of the site, including maps and literature published by the United States Geological Survey (USGS) and California Geological Survey (CGS). We obtained information on the site development history by reviewing historical aerial photographs available through Google Earth, USGS, and other sources. A list of selected references is available at the end of this report. We also reviewed existing environmental reports and groundwater studies performed near the site available through California State Water Resources Control Board (SWRCB, 2022).

## 2.02 Site Reconnaissance Visits

Site reconnaissance visits were conducted at various times for our investigations between August 2020 and March 2022. During these visits, we observed the surface conditions at the site, took photographs, checked for obvious geotechnical/geologic concerns, assessed site accessibility, and identified locations for subsurface explorations. Observations made during our site reconnaissance visits are discussed in Section 4, Site Conditions.

## 2.03 Subsurface Investigations

### 2.03.1 Subsurface Investigations by Others

We reviewed several environmental site assessment and subsurface investigation reports performed by environmental consultants in 2019 through 2021 (AllWest Environmental 2019a, 2019b, 2019c, 2020d, 2020e; Path Forward 2020, 2021). These investigations primarily included geoprobe direct push borings, soil vapor probes, and groundwater sampling locations within and in the vicinity of the project site, as shown on Figure 3. Copies of the geo-environmental exploration logs are included in Appendix D. A summary of the Geo-Environmental explorations performed at the site are included in the table below.

**Table 1 – Summary of Geo-Environmental Explorations Performed by Others**

| Consultant (Report #) | Exploration ID | Date Performed | Depth (ft) | Exploration Type | Advancement /Drilling Method | Hole Size (inches) | Groundwater Depth (ft) | Boring Log Availabe |
|---|---|---|---|---|---|---|---|---|
| AllWest (19061.23) | B-1 | 5/21/2019 | 10 | Geoprobe Boring | Direct Push | 2 | Not Encountered | Yes |
| | B-2 | 5/21/2019 | 10 | Geoprobe Boring | Direct Push | 2 | Not Encountered | Yes |
| | B-3 | 5/21/2019 | 10 | Geoprobe Boring | Direct Push | 2 | Not Encountered | Yes |
| | B-4 | 5/21/2019 | 10 | Geoprobe Boring | Direct Push | 2 | Not Encountered | Yes |
| | B-5 | 5/21/2019 | 10 | Geoprobe Boring | Direct Push | 2 | Not Encountered | Yes |
| | VP-1 | 5/21/2019 | < 1 | Sub-Slab Soil Vapor Probe | Electric Drill | 5/8 | Not Encountered | No |
| | VP-2 | 5/21/2019 | < 1 | Sub-Slab Soil Vapor Probe | Electric Drill | 5/8 | Not Encountered | No |
| AllWest (19086.23.1) | B-9 | 7/17/2019 | 52 | Geoprobe Boring | Direct Push | 2 | Not Encountered | Yes |
| | B-10 | 7/18/2019 | 40 | Geoprobe Boring | Direct Push | 2 | Not Encountered | Yes |
| | VP-1A | 7/19/2019 | < 1 | Sub-Slab Soil Vapor Probe | Electric Drill | 5/8 | Not Encountered | No |
| | VP-2A | 7/19/2019 | < 1 | Sub-Slab Soil Vapor Probe | Electric Drill | 5/8 | Not Encountered | No |
| | VP-3 | 7/19/2019 | < 1 | Sub-Slab Soil Vapor Probe | Electric Drill | 5/8 | Not Encountered | No |
| | VP-4 | 7/19/2019 | < 1 | Sub-Slab Soil Vapor Probe | Electric Drill | 5/8 | Not Encountered | No |
| AllWest (19126.23) | B-11 | 9/26/2019 | 80 | Hollow Stem Auger Boring | Hollow Stem Auger | 8 | 78.9 | Yes |
| | B-12 | 9/26/2019 | 90 | Hollow Stem Auger Boring | Hollow Stem Auger | 8 | 77.3 | Yes |
| Path Forward (115-103-106) | B-13 | 12/14/2019 | 15 | Soil Vapor Probe | N/A | N/A | Not Encountered | No |
| | B-14 | 12/14/2019 | 15 | Soil Vapor Probe | N/A | N/A | Not Encountered | No |
| | B-15 | 12/14/2019 | 18 | Soil Vapor Probe | N/A | N/A | Not Encountered | No |
| | B-16 | 12/15/2019 | 4 | Soil Vapor Probe | N/A | N/A | Not Encountered | No |
| | B-17 | 12/15/2019 | 17 | Soil Vapor Probe | N/A | N/A | Not Encountered | No |
| | B-19 | 12/14/2019 | N/A | Groundwater Sampling Location | N/A | N/A | 77.4 | No |
| | B-20 | 12/14/2019 | N/A | Groundwater Sampling Location | N/A | N/A | 79.2 | No |
| AllWest (202006.23) | SVP-3 | 5/28/2020 | 15.5 | Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-4 | 5/28/2020 | 15.5 | Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-5 | 5/28/2020 | 15.5 | Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-6 | 5/28/2020 | 15.5 | Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-7 A/B | 5/26/2020 | 15.5 | Permanent Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-8 A/B | 5/24/2020 | 15.5 | Permanent Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-9 A/B | 5/26/2020 | 15.5 | Permanent Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-10 A/B | 5/23/2020 | 15.5 | Permanent Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-11 A/B | 5/26/2020 | 15.5 | Permanent Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-12 A/B | 5/23/2020 | 15.5 | Permanent Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-13 A/B | 5/23/2020 | 15.5 | Permanent Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-14 A/B | 5/26/2020 | 15.5 | Permanent Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-15 A/B | 5/23/2020 | 15.5 | Permanent Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-16 A/B | 5/26/2020 | 15.5 | Permanent Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-17 A/B | 5/28/2020 | 15.5 | Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-18 A/B | 5/23/2020 | 15.5 | Permanent Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-19 A/B | 5/27/2020 | 15.5 | Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-20 A/B | 5/27/2020 | 15.5 | Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-21 A/B | 5/27/2020 | 15.5 | Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |
| | SVP-22 A/B | 5/27/2020 | 15.5 | Soil Vapor Probe (Geoprobe) | Direct Push | 2 | Not Encountered | Yes |

### 2.03.2 Subsurface Investigations by A3GEO

#### Cone Penetration Tests

On August 24, 2020, as part of our geotechnical due diligence study for the project, we conducted a subsurface investigation at the project site consisting of three (3) CPT soundings. Prior to conducting field activities, we obtained a boring permit from the City of San Francisco, marked exploration locations, contacted Underground Service Alert (USA), and subcontracted with GeoTech Utility Locating of Moraga, California, a private utility locating company, to screen each location for underground utilities. The approximate locations of the CPT soundings are shown on Figure 3 and the results of the soundings are presented in Appendix B. The exploration locations shown on Figure 3 were determined by measuring from existing site features and should be considered approximate. At each CPT location, the upper five feet was excavated with hand-auger equipment to check for potential underground utilities. ConeTec performed the CPT soundings using truck-mounted equipment. ConeTec's CPT logs, including descriptions of the CPT equipment, procedures, data, and interpretive methods, are attached in Appendix B. CPT-1 was advanced to refusal at depth of about 60.1 feet below the ground surface, CPT-2 was advanced to refusal at about 64.6 feet below the ground surface and CPT-3 was advanced to refusal at about 62.3 feet below ground surface.

As indicated in Appendix B, the CPT method involves pushing an instrumented cone and sleeve into the ground using hydraulic pressure. Data is obtained at frequent intervals (0.328 foot), by which soil properties are interpreted. The CPT logs include measured cone tip resistance ($q_t$), sleeve friction ($f_s$) and pore water pressure (u). Also presented on the logs are geotechnical material descriptions interpreted based on the soil behavior type (SBT) as prescribed in the referenced reports. Nine pore water dissipation tests were performed at various depths during the soundings to assess subsurface groundwater levels. All these tests showed no groundwater at any of the CPT locations at the time of our investigation. A3GEO's field engineer also confirmed this result by using a water level meter at all CPT locations right before backfilling the holes with grout. After completing the CPT soundings, all holes were tremie grouted in accordance with permit requirements.

The CPT logs in Appendix B present data and interpretations pertaining to subsurface conditions at the indicated locations at the time the subsurface exploration was performed; the passage of time may result in changes in the subsurface conditions.

<u>Exploratory Borings</u>

On February 10, two geotechnical hand-auger borings were performed (GHA-1 and GHA-2) at the approximate locations shown on the Site Plan, Figure 3. The hand augers were performed by A3GEO staff, who logged the boring and obtained bag samples at frequent intervals. Each of the hand augers were advanced to depths of 5 feet below ground surface.

On February 17, 2022, three geotechnical borings were drilled (GB-1, GB-2, and GB-3) at the approximate locations shown on Figure 3. A3GEO subcontracted with Clear Heart Drilling, Inc. of Santa Rosa, California to perform these borings. Borings GB-1, GB-2, and GB-3 were performed using a truck-mounted drill rig equipped with 6-inch diameter hollow stem augers. Borings GB-1 and GB-3 were advanced to depths of 26.5 feet below ground surface and Boring GB-2 was advanced to a depth of 36.5 feet below ground surface.

Generally, soil samples were collected from each boring at approximately five-foot intervals using a 2-inch outside diameter (O.D.) Standard Penetration Test (SPT) sampler without liners and a 3-inch O.D. Modified California samplers with liners. The samplers were driven using an automatic 140-pound hammer falling approximately 30 inches. The hammer blows required to drive the sampler the final 12 inches of each 18-inch drive are presented on the boring logs. Where a full 12-inch drive could not be achieved, the number of blows and amount of penetration achieved is shown. Sampler blow counts (in blows per foot) obtained using the SPT sampler correspond to SPT N-values. The Modified California sampler blow counts shown on the logs have been adjusted by a factor of 0.63 to account for the larger sampler end area (Adjusted N-Values).

During drilling, an A3GEO engineer visually/manually classified the soil in general accordance with ASTM D2488 classifications, which are based on the Unified Soil Classification System (USCS). Field classifications were subsequently checked and revised, where appropriate, based on laboratory test data. The logs of the borings are attached in Appendix A and are preceded by: 1) a Key to Exploratory Boring Logs that describes the USCS and the symbols used on the logs. Upon completion of drilling, the boreholes were backfilled with grout in accordance with permit requirements.

The boring locations indicated on Figure 3 were determined by taking measurements from existing site features and should be considered approximate. The ground surface elevations at the boring locations were estimated using the ALTA/NSPS Land Site Survey drawing provided by TNDC (TNDC, 2021). The attached boring logs represent our interpretation of the subsurface materials at the boring locations at the time of drilling. The passage of time may result in changes in the subsurface conditions. A summary of our findings from our subsurface exploration can be found in Section 4, Subsurface Conditions.

A summary of the A3GEO geotechnical explorations performed at the project site is presented in the table below.

**Table 2 – Summary of A3GEO Geotechnical Explorations**

| Location ID | Approximate Surface Elevation[1] (feet) | Depth (feet) | Approximate Bottom Elevation (feet) | Date Performed |
|---|---|---|---|---|
| CPT-1 | 203 | 60.1 | 142.9 | 8/24/2020 |
| CPT-2 | 204 | 64.6 | 139.4 | 8/24/2020 |
| CPT-3 | 206 | 62.3 | 143.7 | 8/24/2020 |
| GHA-1 | 204 | 5 | 199 | 2/10/2022 |
| GHA-2 | 205 | 5 | 200 | 2/10/2022 |
| GB-1 | 206 | 26.5 | 179.5 | 2/17/2022 |
| GB-2 | 204 | 36.5 | 167.5 | 2/17/2022 |
| GB-3 | 206 | 26.5 | 179.5 | 2/17/2022 |

## 2.04 Geophysical Surveys

On February 10, 2022, our geophysical subcontractor, NORCAL Geophysical Consultants, Inc. (NorCal) of Cotati, California, performed a surface geophysical survey at the project site. The purpose of the survey was to provide a shear (S) wave velocity profile to a depth of approximately 100 feet, to estimate the average S-wave velocity in the upper 100 feet ($V_{s100ft}$), Seismic measurements were collected using the multi-channel analysis of surface waves (MASW) method. NORCAL's Seismic MASW Survey Report is presented in Appendix C. The NORCAL report includes: 1) additional information about the survey methodology and procedures, and 2) interpreted s-wave velocity profiles for the MASW soundings.

MASW soundings measure shear waves (S-waves) and are used to interpret the physical properties (e.g. density and hardness) of the materials. The survey methods involve placing a continuous line of geophones on the ground and recording the arrival of P- or S-waves, which are induced into the ground by a hammer striking a steel plate. The MASW measurements for this project were made at an array along Irving Street, as shown on the Site Plan, Figure 3.

## 2.05 Geotechnical Laboratory Testing

Samples from the borings were examined to check field classifications, assign laboratory tests, and interpret geologic units. An A3GEO engineer examined the samples, edited the field versions of the logs, and supervised the preparation of the final logs presented in Appendix A.

Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical properties of the soil that underlie the site. The following geotechnical laboratory tests were performed:

[1] Datum: San Francisco Vertical Datum of 2013 (SFVD13)

- Water content per ASTM Test Designation D-2216;
- Dry density per ASTM Test Designation D-2937;
- Particle Size Distribution per ASTM Test Designation D-422; and

Geotechnical laboratory testing was performed by A3GEO. The results of the tests are presented on the boring logs (Appendix A) at the corresponding sample depths. The laboratory test data sheets are included in Appendix E.

### 2.06 Geochemical and Drill Spoils Analytical Laboratory Tests

We screened for naturally occurring corrosive materials by conducting a suite of geochemical laboratory tests on a sample of soil obtained from a depth of 2.5-4 feet in Boring GB-2. The corrosivity test results are included in Appendix E. The geochemical laboratory tests were performed by Cooper Testing Labs, Inc. and included measurements of:

- Resistivity (100% saturated) per Caltrans 643;
- Chloride ion concentration per Caltrans 422 (modified);
- Sulfate ion concentration per Caltrans 417 (modified);
- pH per Caltrans 643; and
- Moisture per ASTM D2216.

To assess drill spoils disposal characteristics, the following analytical tests were performed on a composite soil sample from the soil cuttings from the investigation:

- Concentration of Title 22 Metals by Environmental Protection Agency (EPA) Method 6020
- Volatile organic compounds (VOCs) by EPA Method 8260B
- Total petroleum hydrocarbons (TPH) as gasoline, diesel, and motor oil using EPA Method 8015m

The results of our drill spoils analytical laboratory tests are presented in Appendix F.

## 3. GEOLOGIC, SEISMIC, AND HISTORICAL SETTING

### 3.01 Regional Geology

The San Francisco Bay Region (SFBR) is characterized by hills and valleys that generally trend southeast/northwest. This characteristic topography is partly the result of the SFBR's location at the boundary between the North American and Pacific crustal plates, which are in relative motion with respect to each other. Over geologic time, the topography of the region formed through a complex series of processes that have included deposition, accretion, faulting, folding, uplift, volcanism and changes in sea level. San Francisco Bay and the adjacent flatlands presently occupy a structural depression between the East Bay Hills and the roughly parallel hills of the San Francisco Peninsula and Marin County.

The SFBR includes three "basement" rock complexes; the Great Valley complex, the Franciscan Complex and the Salinian complex. All were formed in the Mesozoic Era (225 to 65 million years ago) and have been brought together by movement occurring along faults. These Mesozoic basement rock complexes are locally overlain by a diverse sequence of Cenozoic Era (younger than 65 million years) sedimentary and volcanic rocks. Since their deposition, the Mesozoic and Cenozoic rocks have been extensively deformed by repeated episodes of folding and faulting. Significantly, the Bay Area experienced several episodes of uplift and faulting during late Tertiary Period (about 25 million to 2 million years ago) that produced the region's characteristic northwest-trending mountain ranges and valleys.

World-wide climate fluctuations during the Pleistocene (about 1.8 million to 11,000 years ago) resulted in several distinct glacial periods. A lowering of sea level accompanied each glacial advance as water became stored in vast ice sheets. Melting of the continental glaciers during warm intervals caused corresponding rises in sea level. High sea levels favored rapid and widespread deposition in the bay and surrounding floodplains. Low sea levels during glacial advances steepened the gradients of streams and rivers draining to the sea thereby encouraging erosional downcutting. The most recent glacial interval ended about 15,000 years ago. Evidence suggests that during the maximum extent of this latest glaciation, sea level was 300 to 400 feet below its present elevation and the valley now occupied by San Francisco Bay drained to the Pacific Ocean more than 30 miles west of the Golden Gate.

Near the beginning of the Holocene age (about 11,000 years ago) the rising sea re-entered the Golden Gate, and sediments accumulated rapidly beneath the rising San Francisco Bay and on the surrounding floodplains. The sediments that now cover the bottom of the bay and blanket much of the adjacent lower flatlands are less than 11,000 years old. The Holocene-age surface deposits are generally less dense, weaker and more compressible than the adjacent/deeper Pleistocene-age soils that predate the last sea level rise.

### 3.02 Regional Active Faults

Within the SFBR, the relative motion of the Pacific and North American crustal plates is presently accommodated by a series of active northwest-trending faults that exist over a width of more than 50 miles (Figure 6). Faults that are defined as active exhibit one or more of the following: (1) evidence of Holocene-age (within about the past 11,000 years) displacement, (2) measurable aseismic fault creep, (3) close proximity to linear concentrations or trends of earthquake epicenters, and (4) prominent tectonic-related aseismic geomorphology. Potentially active faults are defined as those that are not known to be active but have evidence of Quaternary-age displacement (within about the past 2 million years).

The major active faults shown on Figure 6 include the Hayward, Rodgers Creek, San Andreas, San Gregorio, Concord-Green Valley, Calaveras, West Napa and Greenville faults. These major faults are near-vertical and generally exhibit right-lateral strike-slip movement (which means that the movement is predominantly horizontal and when viewed from one side of the fault, the opposite side of the fault is observed as being displaced to the right). Approximate distances and directions from the site to major Bay Area active faults are presented in the table that follows (USGS Fault Map).

**Table 3 – Approximate Distances and Directions to Principal Bay Area Active Faults**

| Fault System | Approximate Distance from Site | Approximate Direction from Site |
|---|---|---|
| San Andreas | 4 miles | West-Southwest |
| San Gregorio | 8 miles | West-Southwest |
| Hayward–Rodgers Creek | 13 miles | East-Northeast |
| Calaveras | 23 miles | East-Northeast |
| Mount Diablo Thrust | 25 miles | East-Northeast |

### 3.03 Regional Seismicity

Since 1836, six earthquakes of magnitude 6.5 or greater have occurred in the region (Bakun, 1999); the dates, magnitudes (M) and epicentral locations of these six large earthquakes (Bakun, 1999; Tuttle and Sykes, 1992) are summarized in the table that follows.

**Table 4 – Magnitude 6.5 or Greater Earthquakes; 1836-1998**

| Date | Magnitude | Epicenter Location |
|---|---|---|
| June 10, 1836 | 6.5 | East of Monterey Bay |
| June 1838 | 6.8 – 7.2 | Peninsula section of the San Andreas fault |
| October 8, 1865 | 6.5 | Southwest of San Jose |
| October 21, 1868 | 6.8 | Southern Hayward fault (Hayward Earthquake) |
| April 18, 1906 | 7.8 | San Andreas fault (San Francisco Earthquake) |
| October 18, 1989 | 6.9 | Santa Cruz Mountains (Loma Prieta Earthquake) |

The Working Group on California Earthquake Probabilities (WGCEP) has developed authoritative estimates of the magnitude, location, and frequency of future earthquakes in California, which are published in Uniform California Earthquake Forecast (UCERF) reports. The most recent forecast (UCERF3) indicates the following likelihoods for one or more earthquake events of the specified magnitude occurring within the SFBR in the next 30 years (starting in 2014):

**Table 5 – San Francisco Region UCERF3 Forecast (WGCEP, 2013)**

| Earthquake Magnitude (greater than or equal to) | 30-year Likelihood of one or more earthquake events |
|---|---|
| ≥ 5.0 | 100% |
| ≥ 6.0 | 98% |
| ≥ 6.7 | 72% |
| ≥ 7.0 | 51% |
| ≥ 7.5 | 20% |
| ≥ 8.0 | 4% |

The WGCEP has also made estimates of the likelihood of earthquakes with magnitude greater than or equal to 6.7 occurring on specific faults (Aagaard et al., 2016). These probabilities are summarized in the table below.

**Table 6 – SFBR UCERF3 Forecast**

| Earthquake Fault | 30-year Likelihood of One or More Earthquake Events with M≥6.7 |
|---|---|
| Hayward - Rodgers Creek | 33% |
| Calaveras - Paicines | 26% |
| San Andreas | 22% |
| Hunting Creek, Berryessa, Green Valley, Concord, Greenville | 16% |
| Maacama | 8% |
| San Gregorio | 6% |

Compared to the previous forecast (UCERF 2; WGCEP, 2008) the likelihood of moderate-sized earthquakes (magnitude 6.5 to 7.5) are generally lower whereas the magnitude of larger earthquakes is higher. While UCERF3 results are generally in line with previous forecasts, UCERF 3 indicates lower probabilities for earthquakes occurring on the most well-known faults of the SFBR (Hayward and San Andreas), while the probabilities for earthquakes on lesser known faults has increased substantially in some case. The probability of an earthquake on the Calaveras fault was estimated at 7% in the UCERF 2 forecast, compared with 26% in the UCERF 3 forecast. This change reflects a better understanding of the regional fault system and the potential for multi-fault ruptures on many faults.

### 3.04 Local Geology

The site is situated on an area that was comprised of sand dunes prior to urban development in the area. The USGS regional geologic map on Figure 7 (Blake et. al, 2000) and other geologic maps depict the near surface soils at the site as dune sand deposits of Quaternary age (map symbol Qd). The Qd unit is described as follows:

> ***Qd: Dune Sand (Quaternary)*** – *Well sorted; loose to soft* (Blake et. al, 2000)
>
> **Qd**: *clean, well-sorted fine to medium sand; yellowish brown to light gray. Maximum thickness approximately 150 feet.* (Schlocker, 1974)

### 3.05 Geologic Hazard Mapping

The CGS Official Seismic Hazard Map presented on Figure 8 (CGS, 2000a) shows the site is not located within a "Zone of Required Investigation" for earthquake-induced soil liquefaction. The site is not within a State-designated hazard zone for surface fault rupture or a zone of required investigation for seismically-induced landsliding (CGS, 2000a). The project site is also above the line of maximum predicted run-up shown on the CGS tsunami hazard map (CGS, 2009).

### 3.06 Site History

The Sunset District underwent major changes in a short time. Prior to development, in the early 1900s most of it was covered by sand dunes. By the mid-1900s, most of the area had been developed with residential and commercial properties. The project site was undeveloped prior to construction of two commercial structures, in the middle of the parcel, around 1927 (AllWest Environmental, 2019a). Another building was constructed on the parcel between the late-1920s and 1932. The earliest photograph that we were able to locate showing the site is from 1938, which shows these three buildings in the middle of the site surrounded by empty (undeveloped) lots on east and west sides. The photograph of the empty lots illustrates the general nature of the ground at that time, which included grassy/brushy vegetation on top of sand dunes.

A 1946 photograph of the site shows that the empty lots on east and west sides of the site were developed and the site was covered by five commercial properties. From at least 1940 to the mid-1960s, gas stations operated at the 26th (2500 Irving) and 27th Avenue (2550 Irving) corners of the parcel. In 1965, the original, eastern portion of the existing building was constructed on the parcel, occupied by a mortuary/funeral chapel. By 1968, the building increased in size to the current configuration and the customer parking lot added. The mortuary operated at the parcel through the mid-1980s. In 1988, the San Francisco Police Credit Union (SFPCU) initiated occupancy on the Project's parcel (AllWest Environmental, 2019a).

The approximate locations of the former building structures (as originally presented in the AllWest environmental site assessment reports) is displayed within the Site Plan, Figure 3. The historical aerial photographs we reviewed are also presented in Appendix G, Site Photographs.

# 4. SITE CONDITIONS

## 4.01 Surface Conditions

The project site is presently occupied by a two-story building on the eastern side of the lot with a parking lot in the western side. The property includes a two-story, approximately 18,561-square-foot commercial building on the developed parcel, occupied by the San Francisco Police Credit Union (SFPCU). The building footprint occupies approximately 70 percent of the parcel, with the remaining portions developed with an asphalt paved customer parking areas, driveways, and landscaping.

Ground surface elevations at the site range from approximately 202 to 206 feet above mean sea level (ALTA/NSPS Land Title Survey, 2021; SFVD13). Topographic variation at the site is relatively flat with slope gradients decreasing towards the north-northwest at about 2 percent or less. Along the northern edge of the property, an existing concrete retaining wall is approximately 2 to 4 feet in height, with the adjacent properties at a lower elevation.

During our reconnaissance visits, we observed the general condition of the existing building and parking lots in the direct vicinity of the project site. The paved asphalt parking lot within the site appeared to be in good condition with no obvious signs of cracking or distress. Overall, the exterior of the existing building appeared to be in fair to good condition with no obvious indications of significant settlement or distress. The few minor indications of distress observed included some cracking within the concrete entryway and some cracking at several of the columns on the southside of the building near Irving Street.

Photographs of general existing site conditions observed during our site visits is included in Appendix G, Site Photographs.

## 4.02 Adjacent Structures

The west, south, and east sides of the property are bounded by public sidewalks and roads. Three residential properties are located adjacent to the northern side of the property. The foundation types and lowest level floor elevations for the surrounding structures were unknown at the time of writing this report. Further evaluation of the foundation systems and locations of neighboring structures may be needed. Where excavations bear within a 2:1 zone of influence from existing structures, temporary shoring or similar mitigation measures should be provided. Additional recommendations can be provided after details regarding neighboring structures are known.

## 4.03 Site Soil Conditions

Generally, the soils encountered at the site consisted of a layer of artificial fill over dune sand deposits. The subsurface conditions encountered generally correlate reasonably well to available geologic and historic information and data from previous borings and studies in the vicinity.

Subsurface conditions encountered in the subsurface explorations are described below, in the order of occurrence below ground surface:

> **Pavement Section** – Asphaltic concrete (AC) and concrete was encountered in the exploration locations. The AC pavement sections varied in thickness, but generally consisted of about 2 to 4 inches of asphalt concrete over 4 to 8 inches of aggregate base material. The concrete at Boring GB-3 was found to be 6 inches thick.
>
> **Artificial Fill** – A layer of heterogeneous fill was encountered in the exploration locations from below the pavement section to a depth of about 3 feet. As encountered, the Artificial Fill generally consisted of light brown Poorly Graded Sand (SP). Within the landscaping areas, top-soil was observed to consist of light brown or grayish brown Poorly Graded Sand with Silt (SP-SM).

Grain size analyses performed on samples within the fill indicate the sand content was 99% (0% coarse, 3% medium, 96% fine-grained sand) with a fines content of 1%. Moisture contents were observed to be dry to slightly moist. The apparent density is typically noted as medium dense. Additionally, a few pieces of concrete and steel debris were observed within these depths.

**Dune Sands** – The fill is underlain by dune sand deposits. The full thickness of the dune sand deposits was not encountered in the subsurface explorations. As indicated in the boring logs, the dune sand deposits generally consisted of light brown, slightly moist, Poorly Graded Sand (SP).

Grain size analyses performed on samples within the Dune Sands indicate the sand content ranged from 95 to 100% (0% coarse, 1 – 8% medium, 91 – 97% fine-grained sand) with a fines content ranging from 0 to 5%. One sample within GHA-2 had traces of fine gravel (3%). Blow counts from samples had N-values ranging from 1 to 40, indicating very loose to dense sands. Within Borings GB-1, GB-2, and GB-3, the very loose and loose sands were generally found at depths between 5 to 15 feet below ground surface.

Based on the CPT correlations, the subsurface materials encountered are generally classified as sand and sand mixtures. The CPT based correlations indicate the consistency is loose to dense with friction angles that generally ranged from 32 to 45 degrees.

Logs of the A3GEO borings and detailed soil descriptions are included in Appendix A. The Cone Penetration Test Logs are included in Appendix B. The existing geo-environmental boring logs are included in Appendix D. Within the geo-environmental logs, visual descriptions of the soils were noted, but no geotechnical strength data or blow counts (N-values) were recorded. Soil samples and soil gas/vapor samples were collected primarily for the purpose of analytical testing. Laboratory test results are included in Appendix E. A summary of the geotechnical laboratory test results in presented in the table below:

**Table 7 – Summary of Geotechnical Laboratory Test Results**

| Interpreted Soil Unit | Description | USCS | Boring ID | Sample Depth (ft) | Dry Density (pcf) | Moisture Content (%) | % Gravel | % Sand | % Fines |
|---|---|---|---|---|---|---|---|---|---|
| Artifical Fill | Poorly Graded Sand | SP | GHA-1 | 2.5 | | | 0 | 99 | 1 |
| Dune Sand | Poorly Graded Sand | SP | GB-1 | 3 | 104 | 1 | 0 | 97 | 3 |
| Dune Sand | Poorly Graded Sand | SP | GHA-2 | 4.5 | | | 3 | 95 | 2 |
| Dune Sand | Poorly Graded Sand | SP | GB-2 | 5.5 | 99 | 3 | 0 | 99 | 1 |
| Dune Sand | Poorly Graded Sand | SP | GB-3 | 8 | 101 | 2 | 0 | 99 | 1 |
| Dune Sand | Poorly Graded Sand | SP | GB-1 | 10.5 | 102 | 3 | 0 | 100 | 0 |
| Dune Sand | Poorly Graded Sand | SP | GB-3 | 15.5 | 109 | 2 | 0 | 99 | 1 |
| Dune Sand | Poorly Graded Sand | SP | GB-2 | 20.5 | 107 | 3 | 0 | 99 | 1 |
| Dune Sand | Poorly Graded Sand | SP | GB-3 | 25.5 | 109 | 2 | 0 | 99 | 1 |
| Dune Sand | Poorly Graded Sand | SP | GB-2 | 35 | | | 0 | 95 | 5 |

### 4.04 Groundwater

During the A3GEO subsurface explorations, groundwater was not encountered. At the time of our investigations, it is anticipated that groundwater is deeper than the final depths of CPT's and Geotechnical Borings. Nine pore pressure dissipation tests were performed at various depths at the three CPT locations (Appendix B). The tests showed that the groundwater level at the CPT locations at the time of our investigation was deeper than the final depths of CPTs. A3GEO's field engineer also confirmed this result by using a water level meter at the CPT locations right before backfilling.

Additional groundwater studies were performed by AllWest Environmental Inc. and Path Forward Partners, Inc. During the AllWest investigations, groundwater was encountered at a depth of about 78.9 feet below the ground surface at B-11 and 77.3 feet below ground surface at B-12. During the Path Forward investigations, groundwater was encountered at a depth of about 77.4 feet below ground surface at B-19 and 79.2 feet below the ground surface at B-20.

A summary of groundwater levels observed at each boring is provided in the following table.

**Table 8 – Summary of Groundwater Levels Observed**

| Consultant (Report #) | Location | Approximate Surface Elevation (feet) | Date | Groundwater Levels Observed | |
|---|---|---|---|---|---|
| | | | | Depth Below Ground Surface (feet) | Approximate Elevation (feet) |
| AllWest (19126.23) | B-11 | 206 | 9/26/2019 | 78.9 | 127.1 |
| | B-12 | 203 | 9/26/2019 | 77.3 | 125.1 |
| Path Forward (115-103-106) | B-19 | 206 | 12/14/2019 | 77.4 | 128.6 |
| | B-20 | 205 | 12/14/2019 | 79.2 | 125.8 |

Groundwater levels can fluctuate significantly with seasons, location, precipitation, leakage from utilities, and other factors. The California Geological Survey indicates that the historical high groundwater level near the site is about 40 feet below the ground surface (CGS, 2000b).

### 4.05 Existing Below-Grade Improvements

Undocumented fill was encountered during our subsurface exploration, and it is anticipated that pipes, concrete, and other types of buried features may be present throughout the site. The existing facility that occupies the site likely includes other below-grade improvements that are not visible or apparent at the ground surface.

Drawings of the original building construction in the mid-1960's were unavailable, but we reviewed drawings from 1988 and 2002 for development and renovations of the SFPCU. These drawings indicate the site includes spread footings and grade beams of various sizes across the building footprint. Depths of the existing spread footings are expected to be approximately 5 feet below the existing concrete slab.

A ground penetrating radar study was previously performed to search for potential remnant subsurface features of the gas stations (such as underground storage tanks), which were not found, except for an anomaly in the southwest corner of the property that was considered to be a potential dispenser island slab. (AllWest, 2020b)

We explored near-surface subsurface conditions only to the degree necessary to complete our investigation. We did not review plans for previous structures at the site and have no other information related to existing improvements or remnants of previous improvements below the site.

# 5. EVALUATIONS AND CONCLUSIONS

## 5.01 Geologic Hazard Considerations

### 5.01.1 Earthquake Ground Shaking

Strong earthquake shaking is a hazard shared throughout the region and the direct risks posed to structures by ground shaking are mitigated through the structural design provisions of the California Building Code (CBC). Structures at the site should be designed to resist strong ground shaking in accordance with the applicable building code(s) and local design practice. It is our understanding that the project will be subject to the 2019 CBC. The 2019 CBC includes references ASCE 7-16 for methodology for calculating seismic design parameters.

A site-specific ground motion hazard analysis was performed in accordance with: 1) Section 1613A of the 2019 California Building Code (CBC); and 2) American Society of Civil Engineers (ASCE) 7-16 Standard. A further discussion of the analysis is provided in Section 6.02 and the results of our site-specific ground motion hazard analysis are provided in Appendix H.

### 5.01.2 Earthquake-Induced Soil Liquefaction and Dynamic Settlement

Liquefaction is a phenomenon whereby certain types of soils below groundwater lose strength, density, and gain mobility (i.e. flow) in response to earthquake shaking. Soils that are most likely to experience liquefaction include loose (adjusted blow counts less than 10), clean, course-grained soils (i.e. sands and gravels) that are below groundwater. Fine-grained materials (i.e. silts and clays) below groundwater with very low plasticity can also experience generally similar cyclic degradation in response to earthquake shaking and are considered susceptible to liquefaction-type behavior if certain criteria are not met.

We analyzed liquefaction susceptibility, potential, and effects using the data from the onsite CPT soundings (and geotechnical borings). For the purpose of our liquefaction evaluation, we assumed that soils below a depth of 40 feet could potentially be below groundwater at the time an earthquake occurs. We consider this design groundwater level to be a reasonable upper bound of historical high groundwater based on available groundwater data, coupled with our general understanding of local hydrogeologic conditions.

The results of our liquefaction analyses are attached in Appendix I. In addition to the data obtained from our subsurface exploration, key inputs to the liquefaction analyses include earthquake magnitude (MW), peak ground acceleration (PGA), and groundwater depth. We used the following values in our analyses:

> ***Mw = 8.0:*** the mean characteristic magnitude for the rupture of the San Andreas fault (The Maximum Considered Earthquake, or MCE).
>
> ***PGA = 0.71g***: the geometric mean PGA ($PGA_M$) obtained from our site-specific ground motion hazard analysis.
>
> ***Groundwater Depth = 40 feet:*** the estimated high groundwater level.

We performed an analysis using data from the recent CPT soundings using commercially available liquefaction assessment software (CLiq v. 2.3.1.15 by GeoLogismiki) utilizing the methodology of Boulanger and Idriss (2014). In CPT-based liquefaction analyses, soil behavior (i.e. "sand-like" or "clay-like") is interpreted based on the soil behavior type index, $I_c$. In our CPT-based liquefaction susceptibility evaluation, we considered soils with an $I_c$ less than or equal to 2.6 susceptible to liquefaction. In our analysis of liquefaction settlement, we utilized the CLiq utility that weights calculated volumetric strains linearly down to a depth of 59 feet, as proposed in the paper by Cetin et al. (2009). The purpose of this feature is to decrease the influence of possible liquefaction in deep layers that are unlikely to influence the surface behavior of the site.

Based on the preceding inputs, the CLiq program produced plots showing variations with depth for Cyclic Stress Ratio & Cyclic Resistance Ratio (CSR & CRR), Factor of Safety (FS) against liquefaction, Liquefaction Potential Index (LPI), and vertical settlements. The CLiq liquefaction reports are attached in Appendix I. Based on the observed geology and analysis, liquefaction and related hazards are not design considerations.

The strong vibratory motion associated with earthquakes can dynamically compact loose granular soil leading to surficial settlements. Dynamic settlement is not limited to the near surface environment and may occur in both dry and saturated sand and silt. Cohesive soil is not typically susceptible to dynamic settlement.
We evaluated the potential for dynamic settlement using the CPT data collected during our field investigation and the computer program CLiq with the methodology of Boulanger and Idriss (2014). The results of our analysis, presented in Appendix I, indicate that following the considered seismic event, the free-field total dynamic settlement is estimated to be up to about 4 inches. We estimate the differential dynamic settlement to be up to about 2 inches over a horizontal distance of 30 feet. A majority of the settlement occurs within the upper 22 feet, 9 feet, and 14 feet, respectively, for CPT-1, CPT-2, and CPT-3. Based on the analysis, seismic induced settlement is a design consideration.

5.01.3 Other Geologic Hazards *Not* Present

**Liquefaction Induced Reduction in Bearing Capacity** - The Liquefaction Potential Index (LPI) described by Iwasaki et al. (1978) was computed from the results of our liquefaction analysis with the CPT data to evaluate the potential for surface manifestation of liquefaction such as sand boils. The computed values of the LPI, presented in Appendix I, indicate that the potential for surface manifestation of liquefaction or sand boils is low. Based on the results of our analysis and the depth to groundwater, we do not anticipate that sand boils or resulting ground subsidence will occur following a significant seismic event in the vicinity of the proposed structure.

**Lateral spreading** - Lateral spreading is a phenomenon in which blocks of non-liquefied soil move laterally on top of an underlying continuous (or near-continuous) liquefied layer. Hazards posed by lateral spreading are typically greatest where there is a nearby topographic free face towards which spreading can occur. We consider the overall potential for significant earthquake-induced lateral spreading to occur at the site to be very low.

**Faulting and Ground Surface Rupture** - The site is not within an AP Zone and no active faults are mapped in the direct vicinity of the site. The closest AP Zone surrounds the active San Andreas fault, which is approximately 4 miles from the project site. Based on the foregoing, we consider there to be very low hazard for surface fault rupture at the site.

**Landsliding** – Based on the relatively flat topography of the site and vicinity, we consider there to be essentially no potential for large-scale landsliding to affect the site.

**Tsunami and Seiche Inundation** – The site is located at an elevation of approximately 200 feet above mean sea level and is about 7,000 feet inland from the tsunami zone shown on the CGS Tsunami Inundation Map (CGS, 2009). A flood map by FEMA shows the site outside of areas considered susceptible to significant flooding. We consider there to be a low potential for flooding to affect the project site.

**Expansive Soils** – Expansive materials shrink and swell in response to changes in moisture and have the potential to damage improvements that are supported on them. Expansive soils are generally comprised of clayey soils. Based on the materials encountered during our subsurface exploration, which consisted of sands and sand mixtures, expansive soils are not a design consideration.

## 5.02 Geotechnical Considerations

### 5.02.1 Feasibility

Based on the results of our investigation and our understanding of the project, we conclude that the proposed project is feasible from a geotechnical standpoint. Geotechnical design considerations for the project are discussed in the following sections.

### 5.02.2 Undocumented Fill

Existing fills at the site are considered undocumented, unless records are found that demonstrate that the materials were placed and compacted under appropriate engineering controls. Undocumented fill is considered generally unsuitable for the support of the proposed building.

Based on the site development history, which has previously included multiple buildings and structures across the site, undocumented fill should generally be anticipated to depths of up to about 3 feet below the existing ground surface. Deeper undocumented fill may be present in some areas, particularly at locations of former structures that previously occupied the site. Remediation for undocumented fill typically consists of ground improvement, over-excavation and replacement with new engineered fill, or designing foundations to bear below the depth of undocumented fill. Ground improvement methods could include compaction grouting, cement deep soil mixing, rigid inclusions (drilled displacement columns), or other similar methods to mitigate undocumented fill concerns.

### 5.02.3 Dynamic Settlement

Based on the results of our analysis, the free-field total dynamic settlement is estimated to be up to about 4 inches. We estimate the differential dynamic settlement to be about 2 inches over a horizontal distance of 30 feet. Based on our preliminary evaluation, we anticipate that ground improvement will be needed to mitigate dynamic settlement concerns.

### 5.02.4 Design Considerations Relating to Groundwater

Based on a review of documents and our subsurface exploration, groundwater is located at approximately 75 feet, or more, from the ground surface, Historical high groundwater data indicates it may be located 40 feet below the ground surface. We do not anticipate that groundwater will be encountered for excavations that extend to a depth of 20 feet or less from the existing ground surface.

## 5.03 Corrosion Potential

We screened for the presence of corrosive soils by conducting a suite of geochemical laboratory tests on a sample obtained from Boring GB-2 at a depth of approximately 2.5 feet. California Department of Transportation (Caltrans) defines a corrosive environment as an area where the soil contains chloride concentration of 500 ppm or greater, soluble sulfate concentration of 1,500 ppm or greater, and a pH of 5.5 or less (Caltrans, 2018a). Based on the Caltrans guidelines, the tested samples would not be considered corrosive.

**Table 9 – Corrosion Test Data and Guidelines**

| Geochemical Test | Sample ID and Test Results | Corrosion Threshold for Structural Elements |
|---|---|---|
| | Boring B-2 at 4 feet | |
| Resistivity @ 15.5° C (ohm-cm) | 5720 | see below |
| Chloride (mg/kg or ppm) | 3 | ≥ 500 |
| Sulfate (mg/kg or ppm) | 397 | ≥ 1,500 |
| pH | 7.0 | ≤ 5.5 |

The Caltrans guidelines indicate that a minimum resistivity value for soil of less than 1,100 ohm-cm indicates the presence of high quantities of soluble salts and a higher propensity for corrosion and requires testing for chlorides of such soils. The National Association of Corrosion Engineers (NACE) provides guidelines on soil resistivity and soil corrosion classification which are presented in the following table.

**Table 10 – NACE Corrosion Classifications**

| Soil Resistivity (ohm-cm) | Soil Classification |
|---|---|
| Below 500 | Very Corrosive |
| 500 – 1,000 | Corrosive |
| 1,000 – 2,000 | Moderately Corrosive |
| 2,000 – 10,000 | Mildly Corrosive |
| Above 10,000 | Progressively Less Corrosive |

Based on the NACE criteria, the sample from Boring GB-2 would classify as "Mildly Corrosive". A qualified corrosion engineer should be consulted if additional interpretations or recommendations pertaining to corrosion are desired.

### 5.04 Infiltration Evaluation

An evaluation of infiltration characteristics was performed based on grain size testing from samples obtained during our field investigation. The City of San Francisco allows for grain size testing as an approved method of assessing infiltration if the soil classifies as a clean sand (5 percent or less fines content) and dune sand with $D_{10}$ between 0.1 and 2.5 millimeters (SFPUC, 2017).

Analysis of soil permeability was conducted using the Hazen Formula and the calculated soil permeability is assumed equal to the infiltration rate, which is then corrected by applying a factor of 0.33 per applicable City of San Francisco guidelines. The following table presents the results of our infiltration evaluation.

**Table 11 – Summary of Infiltration Evaluations**

| USCS | Boring ID | Sample Depth (ft) | % Gravel | % Sand | % Fines | $D_{10}$ (mm) | Calculated Design Infiltration Rate (inches/hour) | Maximum Design Infiltration Rate (per SFPUC) (inches/hour) |
|---|---|---|---|---|---|---|---|---|
| SP | GHA-1 | 2.5 | 0 | 99 | 1 | 0.1558 | 11.4 | 5 |
| SP | GB-1 | 3 | 0 | 97 | 3 | 0.1574 | 11.6 | 5 |
| SP | GHA-2 | 4.5 | 3 | 95 | 2 | 0.1608 | 12.1 | 5 |
| SP | GB-2 | 5.5 | 0 | 99 | 1 | 0.1632 | 12.5 | 5 |
| SP | GB-3 | 8 | 0 | 99 | 1 | 0.1688 | 13.3 | 5 |
| SP | GB-1 | 10.5 | 0 | 100 | 0 | 0.1749 | 14.3 | 5 |
| SP | GB-3 | 15.5 | 0 | 99 | 1 | 0.1576 | 11.6 | 5 |
| SP | GB-2 | 20.5 | 0 | 99 | 1 | 0.1531 | 11.0 | 5 |
| SP | GB-3 | 25.5 | 0 | 99 | 1 | 0.1632 | 12.5 | 5 |
| SP | GB-2 | 35 | 0 | 95 | 5 | 0.1525 | 10.9 | 5 |

All of the samples collected from the investigation showed a fines content of less than 5%, meeting the requirements for using grain size analysis methods to assess infiltration. The calculated design infiltration rates for individual samples ranged from 11.0 to 14.3 inches per hour. However, the SFPUC guidelines state that:

> *"Regardless of test method and results, the Design Infiltration Rate used for sizing infiltration-based BMPs shall not exceed 5 inches per hour."*

As such, we recommend using the maximum design infiltration rate of 5 inches per hour.

### 5.05 Construction Considerations

#### 5.05.1 Excavation and Shoring

We anticipate that soil materials at the site can generally be excavated with conventional earth-moving equipment, however the contractor should anticipate the presence of obstructions within the fill soils, including cobbles, boulders, old concrete slabs and foundation elements, bricks, and blocks, etc. The Contractor should anticipate that equipment capable of cutting steel and/or breaking concrete may be necessary to remove these obstructions within the fill.

Based on the materials encountered in our subsurface exploration, near-vertical temporary cuts in the fill or dune sand deposits should not be considered stable. We anticipate that shoring or other stabilization methods will need to be utilized to prevent sloughing of the materials exposed on excavation sidewalls.

The contractor is responsible for shoring, excavation safety, and the protection of adjacent offsite improvement throughout all phases of construction. All excavations deeper than 4 feet that will be entered by workers must be shored or sloped for safety in accordance with the applicable: 1) California Occupational Safety and Health Administration (Cal-OSHA) standards; and 2) any site-specific health and safety protocols and procedures required by the City of San Francisco. In all cases, the design, installation, monitoring, and abandonment of site shoring systems are the contractor's responsibilities.

#### 5.05.2 Construction Monitoring

The contractor's responsibilities should include: (1) documenting the condition of the adjacent improvements prior to the commencement of site demolition and excavation activities; (2) designing demolition, excavation and construction programs that will keep surface settlements and vibrations within acceptable limits; and (3) coordinating with local agencies, as needed, to assure that adjacent facilities are not adversely affected during the geotechnical aspects of construction. Recommendations for construction monitoring are provided in Section 6.13.

#### 5.05.3 Wet Weather Construction

Although it is possible for excavation and/or construction to proceed during or immediately following the wet winter months, a number of geotechnical problems may occur which may increase costs and cause project delays. The water content of onsite soils may increase during the winter and rise significantly above optimum moisture content for compaction of subgrade or backfill materials. If this occurs, the contractor may be unable to achieve the specified levels of compaction. Dewatering requirements will potentially increase due to rainfall, surface runoff, seepage and rises in groundwater level. The stability of temporary slopes will decrease, potentially increasing the lateral extent of excavation required. If utility or footing trenches are open during winter rains, caving of the trench walls may occur. Subgrade preparation beneath footings, mat foundations, slabs-on-grade, and pavement sections may prove difficult or infeasible. In general, we note that it has also been our experience that increased clean-up costs may be incurred, and greater safety hazards may exist, if the work proceeds during the wet winter months.

#### 5.05.4 Environmental Considerations

This geotechnical and geologic report does not address design or construction issues related to chemically impacted soils and groundwater as environmental services were not included in our scope.

# 6. RECOMMENDATIONS

## 6.01 General

The following presents our geotechnical recommendations for the design and construction of the proposed project. If the project design differs significantly from that discussed previously in this report, we should be consulted regarding the applicability of the conclusions and recommendations presented herein, and be provided the opportunity to provide supplemental recommendations, where appropriate. Contractors responsible for the geotechnical aspects of the project should become familiar with the contents of this report and acknowledge:

- The site conditions, as described in this report and the attached Appendices;
- The construction considerations discussed in Section 5 of this report; and
- Any additional special project requirements (TNDC, City of San Francisco, etc.).

We recommend that these and all other contractor responsibilities be clearly defined in the project plans and specifications.

## 6.02 Seismic Design

A site-specific ground motion hazard analysis was performed in accordance with: 1) Section 1613A of the 2019 California Building Code (CBC); and 2) American Society of Civil Engineers (ASCE) 7-16 Standard.

The site classification was conducted in accordance with ASCE 7-16, Chapter 20. Shear wave velocity obtained as part of our subsurface investigation was used to determine the average shear wave velocity in the upper 100 feet ($v_{s100\ ft}$), in accordance with ASCE 7-16, Section 20.4.1. Based on the results of the geophysical survey (Appendix C), a Site Class of D was assumed for the project. The results of our site-specific ground motion hazard analysis, including the design response spectrum, are provided in Appendix H.

## 6.03 Foundation Alternatives and Design Criteria

### 6.03.1 General

Primary foundation design considerations for the project include: 1) the presence of undocumented fill, and 2) the potential for seismically-induced dynamic settlement. We recommend a foundation system comprised of either: 1) a shallow foundation above ground improvement; or 2) deep foundations (drilled piers or similar system). We recommend that one foundation system be used for the entire building rather than a combination of multiple different foundation systems. Based on preliminary discussions with the project team, we understand that a mat foundation over ground improvement is the preferred foundation system. Specific loading information, such as column loads and locations, was not available at the time this report was prepared.

### 6.03.2 Mat Foundation

If a mat foundation is selected for the structure, we recommend ground improvement be performed beneath the building footprint to an elevation of 182 feet, or deeper, if required by the ground improvement design. Our subsurface exploration encountered variable depths of loose soil; therefore, additional exploration (CPT soundings) could be performed after the existing building is removed to further refine, and possibly reduce in some areas, the depth of recommended ground improvement.

We recommend that the bottom of the mat foundation be located at a distance of 3 feet, or more, below the existing ground surface. Where existing piles or other foundation elements are present, we recommend removing them to a depth of 5 feet below the bottom of the mat foundation. We recommend the mat be 12 inches, or more, in thickness and include at least two layers (top and bottom) of steel reinforcement. The mat foundation should be designed to span an unsupported length of 10 feet.

The new mat slab below the structure should be evaluated using the allowable contact pressures in Table 12 (DL = Dead Loads; LL = Live Loads; Total = DL + LL + wind or seismic). These allowable contact pressures assume that ground improvement will be performed beneath the foundation and represent the total load that can be placed on the soil at foundation subgrade level. The ground improvement may result in higher values than those presented in the table.

**Table 12 – Recommended Design Contact Pressures for Mat Foundation Design**

| Load Case | Bearing Pressure[1,2] (psf) | Minimum Factor of Safety |
|---|---|---|
| DL Allowable | 3,000 | 3.0 |
| DL + LL Allowable | 4,500 | 2.0 |
| Total Allowable | 6,000 | 1.5 |
| Ultimate | 9,000 | 1.0 |

Notes:
[1] Assumes ground improvement performed per recommendations in this report.
[2] In localized areas of the mat, the bearing pressure values can be increased by 10% if needed.

For preliminary foundation analysis, the deflection of the mat due to applied loads may be modeled using a modulus of subgrade reaction. Where a uniform subgrade modulus is used for the entire mat footprint, a value of 20 psi/in (pci) can be used. At isolated column locations and/or to evaluate concentrated loading, a value of 125 psi/in (pci) can be used. These values assume that ground improvement will be performed beneath the foundation. The subgrade modulus is a function of load magnitude, load distribution, soil parameters, and mat stiffness. The subgrade modulus value provided in this report is intended for initial analysis and should be revised once the final building configuration and loading are known.

Resistance to lateral loads can be provided by passive pressures acting on the vertical faces of below-grade structural elements and by friction along the bottom of the mat. Where below-grade structural elements are surrounded by native soils or new engineered fill, passive resistance can be evaluated using an equivalent fluid weight of 300 pcf. This value can be increased by one-third for dynamic loading. The lateral bearing pressure should be neglected to a depth of 1 foot where the ground adjacent to the foundation is not covered by a slab or pavement. A friction coefficient of 0.20 can be used to evaluate frictional resistance between the waterproofing or vapor barrier membrane and the slab. A friction coefficient of 0.35 can be used to evaluate frictional resistance between soil and the slab. The above passive and frictional resistance values include a factor of safety of at least 1.5 and can be fully mobilized with deformations of less than ½ - and ¼ - inch, respectively.

The ground improvement should be designed for a total static settlement of 1 inch, or less, with a differential of ½ inch, or less, over a lateral span of 30 feet for sustained loads, free-field total dynamic settlement of 1 inch, or less, and a differential dynamic settlement of about ½ inch, or less, over a horizontal distance of 30 feet. Additional settlement analysis may be warranted once the final building configuration and loading are known.

### 6.04 Ground Improvement

The primary objective of the envisioned ground improvement program is to allow the new building to be supported on a structural mat. In our opinion, this can be accomplished by improving the soils beneath the building footprint to an elevation of 182 feet, or deeper, if required by the ground improvement design, in order to mitigate static and dynamic total and differential settlement concerns. Our subsurface exploration encountered variable depths of loose soil; therefore, additional exploration could be performed after the existing building is removed to further refine, and possibly reduce in some areas, the depth of recommended ground improvement.

### 6.04.1 General Considerations

Ground improvement should be implemented below the building to reduce the settlement potential of the native soils. Ground improvement methods that are commonly used locally, in urban environments, to mitigate dry dynamic settlement in sands include compaction grouting, cement deep soil mixing, or rigid inclusions. We recommend that ground improvement be performed under the entire building footprint and not isolated locations. Typically ground improvement would not be performed in parking areas, landscaping, or other similar areas where dynamic induced settlement is generally acceptable. Ground improvement methods which generate excessive vibrations at or near the site should not be used.

Ground improvement programs are typically designed and implemented by specialty contractors that utilize proprietary equipment and methods. In these cases, contract documents prepared by the design team require the specialty ground improvement contractor to thoroughly substantiate the basis for their design(s) and confirm specified levels of performance by field and laboratory testing. Because ground improvement design is strongly linked to the contractor's equipment, it is most commonly procured on a design-build basis. The design-build procedure is generally preferred as it: (1) allows the contractor to design an installation that is best suited to their equipment and proprietary procedures; (2) encourages efficiency and innovation in achieving the desired level of ground improvement, and (3) promotes competition between contractors that should result in a lower overall price. Ground improvement methodologies other than those presented in this section may also be acceptable.

Ground improvement design should be performed in general accordance with the Federal Highway Administration (FHWA) Ground Modifications Methods Reference Manual - Volumes I and II (FHWA, 2017). We recommend that the specifications require that the contractor submit along with their bid a design that includes: 1) a list of five or more projects that were completed over the past five years with a description of relevant project details to demonstrate qualifications; 2) a description of the contractor's proposed equipment and approach; 3) a layout drawing (plan) showing the locations and depths of improvements; 4) geotechnical analyses that show static vertical settlement of 1 inch or less in the improved ground after construction of the building and dynamic vertical settlement of 1 inch or less following the design seismic event; and 5) engineering calculations that demonstrate an adequate allowable contact pressures for the values provided in Table 12. The contractor's submittal should be signed and stamped by a California-licensed civil engineer and will be subject to a joint review by A3GEO and the project Structural Engineer. The contractor should be responsible for revising their design, as appropriate, to satisfy the recommendations and requirements of the joint review.

### 6.04.2 Compaction Grouting

Compaction grouting involves the injection of a low-slump, mortar-like grout under high pressure to compact and displace the adjacent soils. The grout is injected at selected target zones in the subsurface through small-diameter, steel grout pipes. The grout is injected in stages at incremental depth intervals to treat the problem soil zone. Typically, a grid pattern is designed to treat the lateral limits of the area of concern. The grout may include a blend of fine aggregate such as sand, silt, clay, and cement to achieve a pumpable, viscous grout with a low slump that remains intact after injection. Grout injection near existing structures should be performed at low rates and carefully monitored. During treatment, the grout pressure, grout flow rate, and volume of grout are monitored to evaluate the grouting process.

The grouting contractor should select the appropriate means and methods, including the grout materials and mix characteristics, grout mixing methods, injection procedures, injection locations, and injection parameters, to form column-shaped inclusions of grout under the new structure to an elevation of 182 feet. The grout should have an unconfined compressive strength of 100 pounds per square inch or more. The injections should be sequenced so that subsequent points are laterally confined by previous injections. The grout injection rate should not exceed 2 cubic feet per minute.

The grouting contractor should furnish equipment to measure the grout pressure and flow rate during injection and the volume of grout injected. Pressures gages should be installed at the grout pump and grout header. The maximum readable pressure for the pressure gages should not be more than 150 percent of the anticipated peak grout pressure and the dial diameter for analog gages should be 3 inches or more. In general, the equipment should be calibrated no more than one month before the start of work and the grout volume measuring system shall be calibrated daily before pumping and when short-stroking or a change in the pumping rate is suspected. The contractor shall locate and take measures to protect utilities or other underground structures near the proposed work. The contractor should check nearby sewer and drain pipes for grout intrusion during grout injection by camera inspection or flushing clean water through the lines to check for traces of grout. The contractor should use manometers or other instruments to check for heave of the ground surface, underground utilities, or adjacent improvements near the work. The monitoring devices should be capable of detecting movements of 0.1 inches or more. These parameters should be continuously monitored during grout injection. Stage grout injection should terminate if structure heave or a rapid decrease in grout injection pressure is observed before the target stage volume is achieved. The contractor shall provide daily samples of the grout for compression testing during the grouting operation.

Verification Cone Penetration Testing (CPT) should be performed to evaluate the degree of ground improvement achieved by the compaction grouting. A pre-production grouting program, consisting of four or more grout columns, should be performed with one verification CPT sounding located between the columns. During production grouting, a minimum of eight verification CPT soundings should be performed to evaluate the compaction grouting improvement. The CPT sounding locations should be selected by A3GEO and generally located at midpoints between grout columns. Dynamic settlement analysis of the CPT sounding should be performed using the computer software CLiq (v. 2.3.1.15 by GeoLogismiki) with the methodology of Boulanger and Idriss (2014) and the same input parameters presented in this report. Where the results of the verification CPT soundings indicate the compaction grouting intent was not achieved (i.e. dynamic settlement greater than allowable criteria), additional grouting or other mitigation measures should be performed.

#### 6.04.3 Cement Deep Soil Mixing

The Cement Deep Soil Mixing (CDSM) method involves the in-situ mixing of soil with cement to create vertical columns or panels that harden into a strong and rigid material. To mitigate settlement, CDSM should be performed in a grid pattern across the building footprint. CDSM design should be performed in accordance with the FHWA design manuals (FHWA, 2013 and FHWA, 2017). If feasible, a field pre-production test program and report should be conducted which incorporates the proposed mix design, mixing equipment, and mixing procedures proposed for use in production. The field pre-production test and production columns should also include coring from the top to bottom of the column to evaluate the thoroughness of mixing and strength testing of the cores to evaluate the strength of the soil-cement mixture.

We recommend that the specifications require that the contractor submit along with their bid a CDSM design that includes engineering calculations for required 28-day soil-cement strength (including replacement ratio and soil strengths for unimproved ground for replacement ratios less than 100 percent).

We recommend that the specifications require that the contractor submit, along with their bid, a proposed CDSM quality assurance and quality control (QA/QC) plan to ensure that material strengths assumed in the design are achieved. The contractor’s QA/QC plan should include testing of soil from within columns/panels as they are mixed as well as cored samples of completed columns/panels after they have hardened. The following recommendations should be considered “minimum” requirements; the contractor is ultimately responsible for the satisfactory post-construction performance of the completed CDSM system.

The quality control CDSM construction should be achieved by continuously measuring the grout flow rates and specific gravity using a mass-flow meter, hydrometer, or mud balance device. The total volume of grout and the total weight of cement used to construct each column should be measured and recorded. Quality should be assured by performing verification tests in the form of wet sampling once per shift, or every 400 cy of mixing, whichever is more frequent, at random locations, and a single depth per location, selected by A3GEO. Samples should be retrieved from a freshly mixed column, cast in molds, cured in a laboratory and tested for unconfined compressive strength by the Owner's Structural and Materials Testing agency. Approximately 6 cylinder samples should be obtained from each column/panel location. Strength test results should achieve the minimum Unconfined Compressive Strength (UCS) indicated in the contractor's approved submittal.

We recommend that the contractor take continuous core samples at a frequency of one core per 3 percent of CDSM elements (where an element is defined as treated soil produced by one setup of either a single or multiple-axis machine). The continuous cores should be performed along the full depth of installed elements for uniformity determination/confirmation. The core samples should be taken in accordance with FHWA design manual recommendations (FHWA, 2013) at locations selected by A3GEO. The testing frequency listed above may be increased if determined necessary by A3GEO Engineer based on the field observations and/or evaluation of test results.

#### 6.04.4 Rigid Inclusions (Drilled Displacement Columns)

Rigid inclusions are commonly installed by specialty contractors under a design-build approach. Rigid inclusions are also known by variety of trade names such as Column-Supported Embankments (CSE), Controlled Modulus Columns (CMCs), Auger Cast Columns (ACCs), Drilled Displacement Columns (DDC), Cast-in-Place Ground Improvement Elements (CGEs) and GeoConcrete Columns (GCCs). Based on site conditions, we anticipate that rigid inclusions would be installed using continuous flight augers (CFAs) or DDC with the use of a steel pipe fitted with a conical tip. CFA and DDC columns most commonly range in size from about 16 to 24 inches in diameter. CFAs and DDC have a hollow stem through which concrete or grout is pumped to cast the pile from the bottom up as the auger is extracted. Spacing of columns is typically designed to achieve an area replacement ratio of between 3 to 10 percent. A load transfer platform, typically about 12 inches thick or more, of aggregate base or clean crushed rock should be placed and compacted at the top of the rigid inclusions. In some cases, geogrid reinforcement is also included in the load transfer platform.

The rigid inclusions should extend to an elevation of 182 feet or until an acceptable refusal criteria is met.

Load testing of columns should be performed to verify design assumptions. Rigid inclusion design should be performed in accordance with the FHWA Ground Modifications Methods Reference Manual - Volumes I and II (FHWA, 2017) for column-supported embankments.

### 6.05 Retaining Walls

Recommended lateral pressures are provided below for design of retaining walls in the permanent condition. Where possible, we recommend that retaining walls will be fully drained to prevent the build-up of hydrostatic pressure.

#### 6.05.1 Wall Back-drainage

Back-drainage should consist of either: (a) prefabricated drainage material (Miradrain or an approved alternative) installed in accordance with the manufacturer's recommendations, or (b) a drain rock layer at least 12 inches wide. Prefabricated drainage material should drain to a perforated plastic pipe or an approved prefabricated drainage conduit. Back-drainage should drain into a perforated plastic pipe installed (with perforations down) along the base of the walls on a 2-inch-thick bed of drain rock. Plastic pipe should be sloped to drain by gravity to a sump, relief wells, or other suitable discharge and a cleanout should be provided at the pipe's upslope end. Perforated and non-perforated plastic pipe used in the drainage system should consist of 4-inch diameter Schedule 40 PVC or an approved equivalent. Drain rock should conform to Caltrans specifications for Class 2 permeable material. Alternatively, locally available, clean, ½- to ¾-inch maximum size crushed rock or gravel could be used, provided it is encapsulated in a non-woven geotextile filter fabric, such as Mirafi 140N or an approved alternative. The upper 2 feet of retaining wall backfill (above back-drainage) should be comprised of low-permeability soil to limit surface water infiltration into the retaining wall back-drainage system.

6.05.2 Design Earth and Bearing Pressures

Walls that are not free to rotate at their tops (including building walls) should be evaluated using an "at-rest" earth lateral pressure distribution for restrained walls. Retaining walls that are not restrained at the top (i.e., cantilever) can be evaluated using "active" lateral earth pressures. Wall deflection equivalent to about 1 percent of wall height may be needed to fully mobilize active earth pressures. Soil movement at the ground surface behind the wall should be anticipated to mobilize the active pressures. Recommended values for design are presented in Figure 9 for non-restrained walls and Figure 10 for restrained walls.

Continuous footings for retaining walls should be at least 24 inches wide, extend at least 24 inches below lowest adjacent grade and be founded on new engineered fill and/or competent natural dune sand. Footings for retaining walls can be designed using the bearing pressures provided in Table 13.

**Table 13 – Recommended Design Bearing Pressures for Retaining Wall Design**

| Load Case | Bearing Pressure[1,2] (psf) | Minimum Factor of Safety |
|---|---|---|
| DL Allowable | 2,500 | 3.0 |
| DL + LL Allowable | 3,750 | 2.0 |
| Total Allowable | 5,000 | 1.5 |
| Ultimate | 7,500 | 1.0 |

Notes:
[1] Assumes continuous footing with width of 24 inches.
[2] Where retaining walls are located in areas above ground improvement, the bearing pressures will likely be higher than those presented in this table.

Resistance to lateral loads can be provided by passive pressures acting on the vertical faces of below-grade structural elements and by friction along the bottom of the footing. Where below-grade structural elements are surrounded by native soils or new engineered fill, passive resistance can be evaluated using an equivalent fluid weight of 300 pcf. This value can be increased by one-third for dynamic loading. The lateral bearing pressure should be neglected to a depth of 1 foot where the ground adjacent to the foundation is not covered by a slab or pavement. A friction coefficient of 0.35 can be used to evaluate frictional resistance between soil and the footing. The above passive and frictional resistance values include a factor of safety of at least 1.5 and can be fully mobilized with deformations of less than ½ - and ¼ - inch, respectively

### 6.06 General Recommendations for Temporary Shoring and Underpinning

The contractor is responsible for the stability of excavation cuts and for the protection of all existing improvements (including adjacent walls and footings) during construction. Excavations deeper than 4 feet that will be entered by workers must be shored or sloped for safety in accordance with Cal-OSHA requirements. Prior to the start of excavation, the contractor should prepare a submittal detailing their proposed equipment, materials, and means and methods to safely accomplish the work and protect against excavation-related movements and structural damage. The shoring should be designed by a shoring engineer familiar with sandy soil. A3GEO should review and approve the submittal prior to the start of work.

The west, south, and east sides of the property are bounded by public sidewalks and roads. Three residential properties are located adjacent to the northern side of the property. The foundation types and lowest level floor elevations for the surrounding structures were unknown at the time of writing this report. Further evaluation of the foundation systems and locations of neighboring structures may be needed. Where excavations bear within a 2:1 zone of influence from existing structures, temporary shoring or similar mitigation measures should be provided. Shoring design may need to incorporate underpinning of existing foundations. Shoring or underpinning design should provide continuous support of adjacent structures and underlying soils. Slotted excavation techniques or alternating pier construction should be utilized to minimize potential loss of support of adjacent structures during shoring or underpinning construction. Additional recommendations can be provided after details regarding neighboring structures are known.

## 6.07 Earthwork

### 6.07.1 Site Preparation and Excavation

Prior to demolition and site clearing, all active subsurface utilities in and immediately surrounding the site limits should be located, marked and protected or relocated. Areas within the site limits should be cleared of concrete, asphalt concrete, aggregate base, catch basins, storm drains, sewers, utilities and all other near-surface improvements. Cleared materials should be removed from the site unless they are specifically identified as suitable for re-use by the Owner’s environmental consultant and A3GEO. The contractor should document the condition of existing improvements located outside of the site limits and should perform any and all monitoring activities required by the permitting agencies or other adjacent property owners.

Excavation will be required to construct building foundations as well as to remove existing subsurface improvements and debris. The contractor is responsible for the design, implementation and safety of all site excavations; this responsibility includes (but is not necessarily limited to) excavation shoring, temporary cut slopes and construction-phase dewatering.

### 6.07.2 Overexcavations and Removals

From a geotechnical standpoint, it will be necessary to remove all existing footings, slabs, walls, pavements and buried utilities from within and below the footprint of the planned building. Excavations should be backfilled with engineered fill per the recommendations of this report. Where existing deep foundations (e.g. drilled piers or drilled/belled caissons) exist, it will be necessary to remove at lease the upper portions of these elements so that they do not interfere with planned construction or create localized “hard spots” beneath new foundations or slabs-on-grade.

Unsuitable materials include, but may not be limited to dry, loose, soft, wet, expansive, organic, or compressible natural soil, and undocumented or otherwise deleterious fill materials. Based on the site history and materials encountered in our subsurface exploration, undocumented fill should be anticipated to a depth of about 3 feet below the existing ground surface. Excavations should be backfilled with engineered fill or controlled low strength material (CLSM).

If unsuitable materials are encountered during construction, we recommend that all unsuitable soils be removed from within the bearing zone below and surrounding planned foundations. We recommend that the bearing zone be defined by imaginary planes inclined at 1:1 (horizontal to vertical) extending downwards and outwards from the outer edge of the foundations. The minimum vertical extent of overexcavation will depend upon the depth of unsuitable material requiring removal, which A3GEO will determine in the field during overexcavation.

6.07.3 Fill Materials, Placement, and Compaction

Geotechnical requirements for fill materials are provided below:

***General Fill -*** General fill material should have an organic content of less than 3 percent by volume and should not contain environmental contaminants or rocks or lumps larger than 6 inches in greatest dimension. General fill can be used anywhere except where structural fill is required.

***Structural Fill*** - Structural fill should conform to the requirements for General Fill, have a Plasticity Index no greater than 12 and a Liquid Limit no greater than 40.

***Caltrans Class 2 Aggregate Base*** – Aggregate Base (AB) should conform to the requirements of Caltrans Class 2 Aggregate Base, ¾-inch maximum (Caltrans, 2018b). Note that Caltrans Class 2 AB meets the requirements for Structural Fill.

***Imported Fill*** – Imported fill should conform to the requirements for Structural Fill and should be evaluated by our firm and the project environmental consultant prior to its importation to the site.

***Controlled Low Strength Material (CLSM)*** – CLSM should conform to the requirements of Caltrans California Standard Specification Section 19-3.02G (Caltrans, 2018b).

All proposed fill materials should be approved by A3GEO and the project environmental consultant prior to their use. Native/existing materials from the site can be suitable for re-use as fill, from a geotechnical standpoint, if they can be processed (i.e., by crushing or blending) to meet the above requirements.

The subgrades beneath areas to receive fill should be approximately level. Fill should be placed in lifts no greater than 8 inches in uncompacted thickness, moisture conditioned as appropriate, and compacted in uniform and systematic manner. Geotechnical requirements for fill placement and compaction are presented below (per ASTM D-1557 Test Methods):

- General Fill that is predominantly cohesive (greater than 15 percent passing #200 sieve) should be moisture conditioned, as necessary, to between 2 and 5 percent over optimum moisture content and compacted to at least 90 percent relative compaction.
- General Fill that is predominantly granular (equal to or less than 15 percent passing #200 sieve) should be moisture conditioned, as necessary, to near or over optimum moisture content and compacted to at least 95 percent relative compaction.
- Structural Fill should be moisture conditioned, as necessary, to near or over optimum moisture content and compacted to at least 95 percent relative compaction.
- The upper 6 inches of fill beneath concrete slabs-on-grade should be compacted to at least 95 percent relative compaction, per ASTM D-1557.
- The upper 12 inches of fill (excluding aggregate base) beneath areas subject to vehicular loading should be compacted to at least 95 percent relative compaction, per ASTM D-1557.

6.07.4 Subgrade Preparation

Prior to placing fill or foundations, subgrade should be prepared in accordance with the following recommendations:

***Below Fill, Slabs, and Pavement -*** After clearing and grubbing, check for unsuitable materials. If unsuitable material is encountered, remove per Section 6.07.2 and replace with Structural Fill or CLSM. Scarify exposed subgrade to a depth of 8 inches then moisture condition and compact scarified subgrade per Section 6.07.3. Keep in moist condition by sprinkling water.

***Utility Trenches -*** Check for unsuitable materials. If unsuitable material is encountered, remove per Section 6.07.2 and replace with Structural Fill or CLSM. Remove loose or soft material or compact per Section 6.07.3.

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

6.07.5 Utility Trenches

We recommend the contractor carefully evaluate the stability of all trenches and use temporary shoring, where appropriate. The design and installation of the temporary shoring should be wholly the responsibility of the contractor. In addition, all state and local regulations governing safety around such excavations should be carefully followed.

Utility trenches should be backfilled with fill placed in lifts not exceeding 8 inches in uncompacted thickness. Trenches should be filled by placing a granular layer (shading) beneath and around the pipe, and then 6 to 12 inches of shading should be carefully placed and tamped above the pipe. The remaining portion of the trench should be backfilled with onsite or import soil. The backfill (above shading layers) should be placed and compacted by mechanical means as recommended in Section 6.07.3 of this report. All compaction operations should be performed by mechanical means only. Jetting should not be allowed. The preceding compaction recommendations are based on general geotechnical considerations. If local agency and/or utility company specifications require different or more stringent backfill requirements, those specifications should be followed.

A3GEO should observe utility trench backfilling and test compaction, as appropriate, to confirm and document that the work was performed in accordance with the specifications and the intent of our geotechnical recommendations.

**6.08 Exterior Flatwork**

Subgrades beneath exterior flatwork should be prepared in accordance with Section 6.07.4.

Slab reinforcing should be provided in accordance with the anticipated use and loading of the slab. We recommend that exterior slabs-on-grade be at least 4 inches thick over 6 inches of aggregate base and reinforced with steel bar or mesh reinforcement. Exterior slabs should be structurally independent from buildings.

Concrete slabs that may be subject to vehicle loadings should be designed in accordance with recommendations for rigid pavements. Appropriate jointing of concrete flatwork can encourage cracks to form at joints, reducing the potential for crack development between joints. Joints should be laid out in a square pattern at consistent intervals. Contraction and construction should be detailed and constructed in accordance with the guidelines of ACI Committee 302 (ACI, 2016). The lateral spacing between contraction joints should be 8 feet or less for a 4-inch thick slab. Root barriers adjacent to trees may be considered to reduce the potential for pavement heave from root growth.

Pedestrian walkways consisting of decomposed granite or similar finishing surface should be 4 inches or more in thickness or, alternatively, constructed over a base of 4 inches or more of aggregate base.

**6.09 Pavements**

6.09.1 Rigid Pavements

Rigid Portland cement concrete (PCC) pavements may also be used in driveway/loading areas. Concrete pavement sections based on methodologies developed by the Portland Cement Associate (PCA) are presented in the following table for a 20-year design period with appropriate periodic maintenance.

**Table 14 – Concrete Pavement Structural Sections**

| Loading Condition[1] | Design Period | Subgrade Modulus (pci)[2] | Concrete Pavement Section (inches) |
|---|---|---|---|
| ADTT = 10 (Traffic Category A - car parking areas and access lanes) | 20 years | 50 | 6 inches PCC[3]<br>6 inches AB[4] |
| ADTT = 300 (Traffic Category B - bus parking areas) | 20 years | 50 | 7 inches PCC[3]<br>6 inches AB[4] |
| ADTT = 300 (Traffic Category C - truck parking areas, bus entrance lanes) | 20 years | 50 | 7½ inches PCC[3]<br>6 inches AB[4] |

**Notes**:
[1] ADTT: Average Daily Truck Traffic. Trucks defined as vehicles with at least six wheels; excludes panel trucks, pickup trucks, and other four-wheel vehicles
[2] pci: pounds per cubic inch
[3] PCC: Portland cement Concrete
[4] AB is Class 2 Aggregate Base complying with Caltrans Standard Specification Section 26 (2018b).

The recommended section presumes that the concrete will have a 28-day flexural strength of 550 psi or an equivalent compressive strength of about 4,000 psi at 28 days. Aggregate base for pavement should be placed in lifts of no more than 8 inches in loose thickness and compacted to at least 95 percent relative compaction as determined by ASTM D-1557.

Appropriate jointing of concrete pavement can reduce the potential for crack development between joints. Joints should be laid out in a consistent square pattern. Contraction joints formed by premolded inserts, grooving plastic concrete, or saw-cutting at initial hardening, should extend to a depth equivalent to 25 percent of the slab thickness and 1 inch or more for thin slabs. Contraction joints should be reinforced with smooth dowels placed across the joint at mid-slab height. Construction joints subject to traffic loading should be reinforced with smooth dowels as for contraction joints. Construction joints within the middle third of the typical joint spacing pattern should be reinforced with tiebars. Contraction, construction, and isolation joints should be detailed and constructed in accordance with the guidelines of the ACI Committee 302 (ACI, 2015) and/or recommendations for Caltrans specifications for jointed plain concrete pavement (JPCP).

6.09.2 Interlocking Concrete Pavers

Interlocking concrete pavers (ICP) may be considered for the proposed pavements and walkways on the project. Recommended structural section alternatives, based on the methodologies in the Caltrans Pervious Pavement Design Guidance (2014) and in American Society for Civil Engineers (ASCE) Standard 58-10, are provided in the following table for a range of traffic levels. Concrete paver sections that include asphalt-treated permeable base (ATPB) should be used for crosswalk pavers and other situations where the pavers are laterally restrained by asphalt pavements.

**Table 15 – Interlocking Concrete Paver Section**

| Traffic Index | Alternative 1 | Alternative 2 |
|---|---|---|
| Walkway<br>(Category A) | 2⅜-inch paver<br>1-inch bedding<br>4½ inches Class 3 Perm | --- |
| Driveway Areas<br>(Category B) | 3⅛-inch paver<br>1-inch bedding<br>4½ inches Class 3 Perm<br>8½ inches Class 2 AB | 3⅛-inch paver<br>1-inch bedding<br>4 inches ATPB<br>4 inches AB |
| Parking Area for Heavy Vehicles<br>(Category C) | 3⅛-inch paver<br>1-inch bedding<br>4½ inches Class 3 Perm<br>24 inches Class 2 AB | 3⅛-inch paver<br>1-inch bedding<br>4 inches ATPB<br>8 inches AB |

**Notes**:
[1] Class 3 Perm is Class 3 Permeable material complying with Caltrans Standard Specification Section 68 (2018b).
[2] AB is Class 2 Aggregate Base complying with Caltrans Standard Specification Section 26 (2018b).
[3] ATPB is Asphalt Treated Permeable Base complying with Caltrans Standard Specification Section 29-2 (2018b).
[4] Where stormwater management and drainage is not the objective of the ICP design, Caltrans Class 2 Aggregate Base may be substituted for Class 3 permeable material provided in the table.

### 6.10 Flexible Utility Connections

This report includes estimates of earthquake-induced settlement, which can be considered generally representative of the adjacent ground outside of the site perimeter. It should be anticipated that seismic settlements may vary across the site and between the building and adjacent ground. Subsurface utilities that enter the site may experience abrupt settlement differentials at the site perimeter as a consequence of strong earthquake shaking. For this reason, we recommend that the Design Team evaluate the need for flexible connections/transitions where utilities enter the site to mitigate the potential post-earthquake damage

### 6.11 Moisture Vapor Barrier

The migration of moisture through slabs underlying enclosed spaces or overlain by moisture sensitive floor coverings should be discouraged by providing a moisture vapor retarding system between the subgrade soil and the bottom of slabs. It is our understanding that a vapor barrier system will be designed by others. If this is not the case, please contact us for additional geotechnical recommendations regarding vapor barrier design.

### 6.12 Drainage and Site Maintenance

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

Landscaping adjacent to foundations should include vegetation with low-water demands and irrigation should limited to that which is needed to sustain the plants. Trees should be restricted from the areas adjacent to foundations a distance equivalent to the canopy radius of the mature tree. Bioretention areas should not be located within a distance of 10 feet from structure foundations. Where bioretention areas are closer than 10 feet, a vertical water cut-off barrier (concrete or impermeable membrane) should be located between the bioretention area and foundation that extends to a depth of 4 feet below the bottom of the bioretention area. Alternatively, a horizontal impermeable membrane can be placed beneath the bioretention area that extends out a distance of 5 feet from the foundation.

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

## 6.13 Construction Monitoring and Instrumentation

An instrumentation program can be implemented to evaluate design assumptions, and monitor vibrations at adjacent structures, deformations of the excavations, and ground surface settlement. The monitoring program would include seismographs and an array of surface control points. The data obtained should be distributed to appropriate parties during the course of construction. To reduce the potential for damage claims from nearby property owners, an instrumentation and monitoring program should be implemented, consisting of the components presented in the following sections.

### 6.13.1 <u>Preconstruction Conditions Surveys</u>

We recommend preconstruction conditions surveys be completed before the beginning of construction on structures within approximately 50 feet of proposed construction activities. Preconstruction condition surveys should include the exterior and interior of the adjacent neighboring structures. Surveys should include photographs and measurements of relevant site features and hardscape features, including distress features, such as cracks and/or separations that may be present. Consideration may be given to videotaping the survey.

### 6.13.2 <u>Crack Meters</u>

Crack meters should be installed, subject to approval of the property owners, on existing exterior and interior cracks in existing structures during the pre-construction surveys or at a point prior to the start of construction. A crack meter monitoring plan should be developed by the design team prior to construction, and monitoring program threshold and limiting criteria should be incorporated into the Contract Documents.

### 6.13.3 <u>Survey Reference Points</u>

Survey reference points should be installed on the faces of existing adjacent building walls to monitor for potential movement. Additional survey reference points should be placed on adjacent streets, sidewalks, and at other locations determined by the design team. A survey monitoring plan should be developed by the design team prior to construction, and monitoring program threshold and limiting criteria should be incorporated into the Contract Documents. The survey targets should be installed near the excavations at approximately 20-foot spacings. We recommend that the contractor be responsible for maintaining total settlement or horizontal displacement at any survey point to less than ½ inch. If the settlements reach this limit, we recommend that a further review of construction methodologies be performed, and appropriate changes be made.

### 6.13.4 <u>Construction Vibration Monitoring</u>

General guidelines on vibration monitoring and thresholds are presented in this section for informational purposes. We recommend a vibration monitoring specialist be utilized for the project. The vibration monitoring specialist should develop a monitoring program along with threshold and limiting criteria, subject to the approval of the Owner, project structural engineer, and A3GEO.

Humans can detect vibrations at very low levels which may result in complaints and damage claims. Published data indicate that transient vibrations from construction activities, such as pile driving, are noticeable at peak particle velocities as low as 0.02 to 0.06 inches per second (ips). At peak particle velocities as low as 0.2 to 0.4 ips, the vibrations are disturbing and may result in complaints and damage claims. However, these vibration levels are typically below the peak particle velocity threshold considered to cause cosmetic damage to modern commercial/residential construction.

An additional concern is the possibility of settlement of the sand, silty sand, and sandy silt underlying structures during construction activities. This settlement may result in damage to the structures. Based on our experience with past projects in similar conditions, if the construction vibrations can be maintained below a peak particle velocity of 0.2 ips, the settlement can likely be limited to acceptable levels.

We recommend that vibration caused by construction activities be monitored in terms of peak particle velocity during construction with seismographs positioned near the adjacent structures and monitored during construction. Based on the type and condition of adjacent structures, an appropriate peak particle velocity threshold should be selected by the vibration monitoring specialist. If peak particle velocities exceed this threshold, construction activity should stop, and construction procedures should be re-evaluated to reduce the potential for excessive vibration.

### 6.14 Future Geotechnical Services

#### 6.14.1 Design-Phase Consultations and Plan Reviews

We recommend we be provided the opportunity to review Project plans and specifications as they are being developed in order to check conformance with the intent of our geotechnical recommendations and to provide timely input, in the event that revisions are needed. We should also perform a general review of the geotechnical aspects of the final plans and specifications, the results of which we should document in a formal plan review letter.

#### 6.14.2 Construction-Phase Geotechnical Services

As Geotechnical Engineer of Record, it is essential that we provide geotechnical services during construction to check whether geotechnical conditions are as anticipated, provide supplemental recommendations where necessary, and document that the geotechnical aspects of the work substantially conform to the approved Contract Documents and the intent of our geotechnical recommendations. Critical aspects of construction that A3GEO should observe include site preparation, ground improvement, subgrades to receive new fill, fill placement, drainage installations, and mat subgrade preparation. A3GEO should also review, comment upon and approve, where appropriate, contractor submittals (including material submittals and requests for information or clarification) that are geotechnical in nature.

## 7. LIMITATIONS

This report has been prepared for the exclusive use of the client and their consultants. The data and interpretations presented in this report were developed in accordance with generally accepted geotechnical and engineering geologic principles and practices. No other warranty, expressed or implied, is made. The findings of this report are valid as of the present date. However, the passing of time will likely change the conditions of the existing property due to natural processes or the works of man. In addition, due to legislation or the broadening of knowledge, changes in applicable or appropriate standards will occur. Accordingly, this report should not be relied upon after a period of three years without being reviewed by this office.

## 8. REFERENCES


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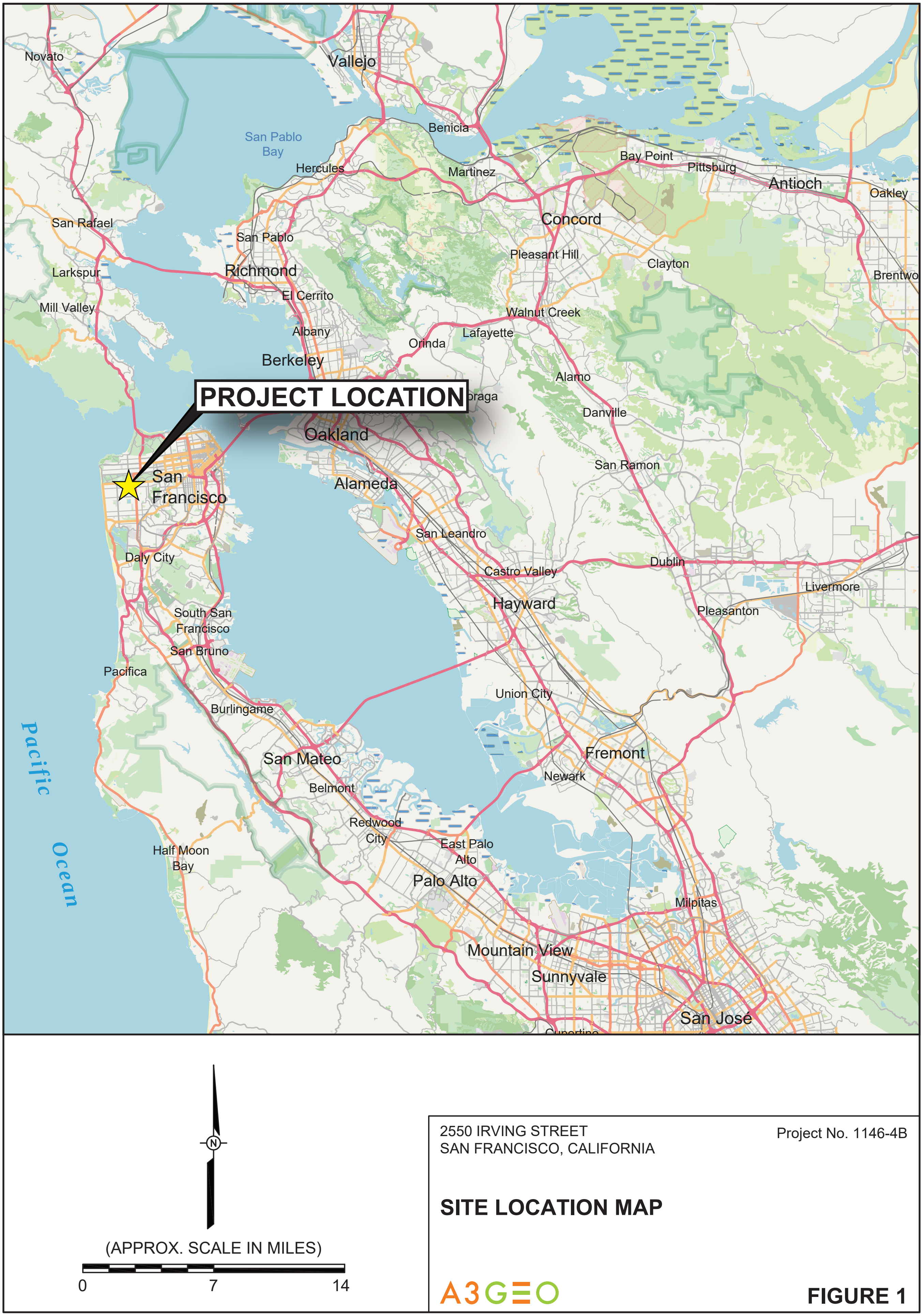
# FIGURES




2550 IRVING STREET
SAN FRANCISCO, CALIFORNIA

Project No. 1146-4B

**SITE LOCATION MAP**

A3GEO

**FIGURE 1**

Date: 8/21/2020


Base: USGS 7.5 Minute Topographic Quadrangles - San Francisco North and San Francisco South

(APPROX. SCALE IN FEET)

0 4,000 8,000

2550 IRVING STREET
SAN FRANCISCO, CALIFORNIA

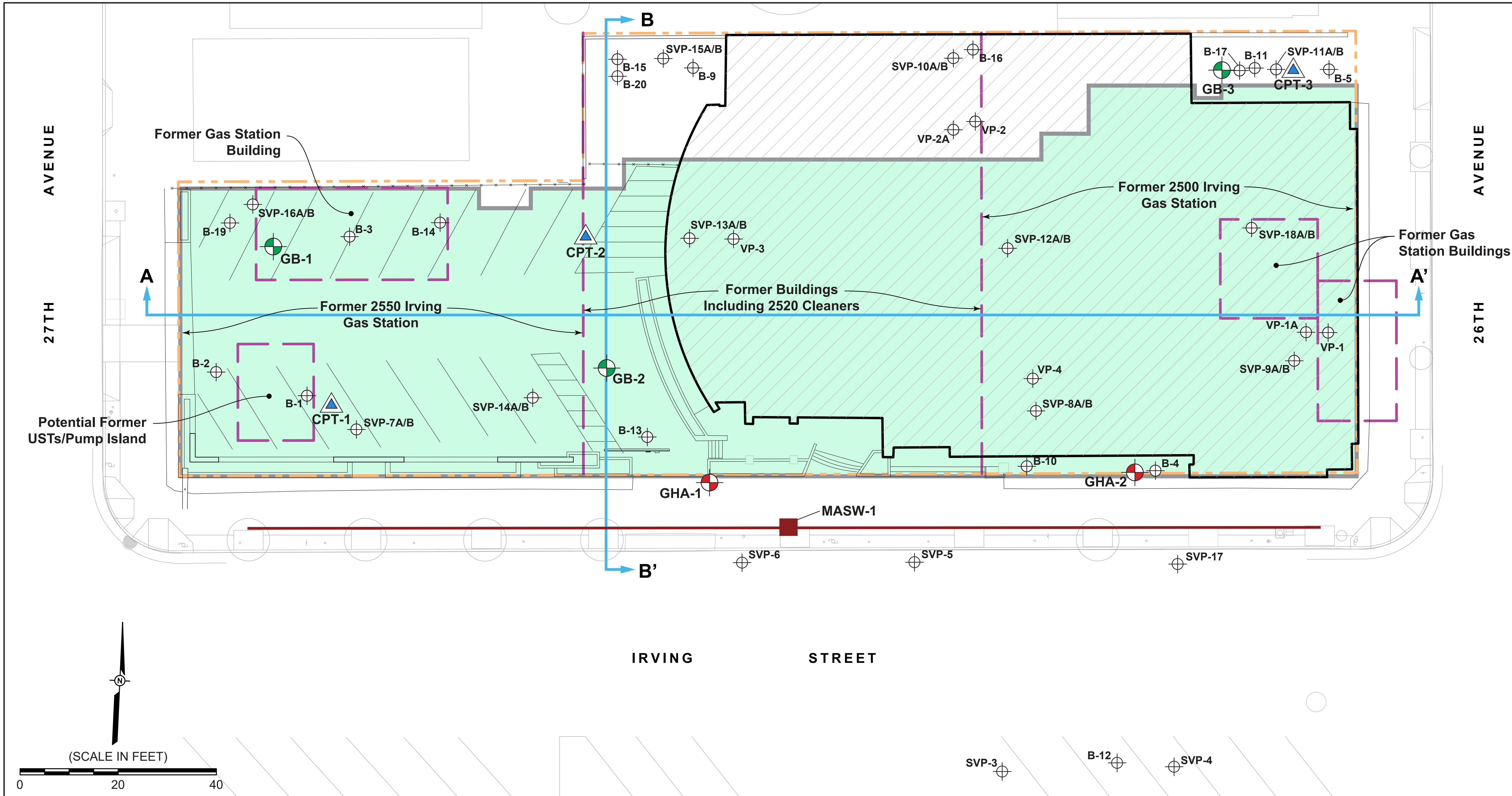
Project No. 1146-4B

## SITE VICINITY MAP

A3GEO

**FIGURE 2**

Date: 8/21/2020




**EXPLANATION**

- Geotechnical Boring (A3GEO, 2022)
- Geotechnical Hand-Auger Boring (A3GEO, 2022)
- Cone Penetrometer Test (CPT) Sounding (A3GEO, 2020)
- Environmental Exploration Locations (performed by others)
- Geophysical MASW Sounding Array (2022)
- Property Line
- Proposed Development
- Existing Building
- Approximate Location of Former Buildings
- B B' Location of Geologic Cross Section

**Notes**:
1. Base map from TNDC Drawing Set "2550 Irving St PRJ Submittal - Preliminary - Not For Construction" Revision 12/3/21.
2. Locations of features shown should be considered approximate.
3. Former structure locations and environmental exploration locations are based on environmental site assessment reports from AllWest Environmental and Path Forward Partners, Inc.

2550 IRVING STREET
SAN FRANCISCO, CALIFORNIA

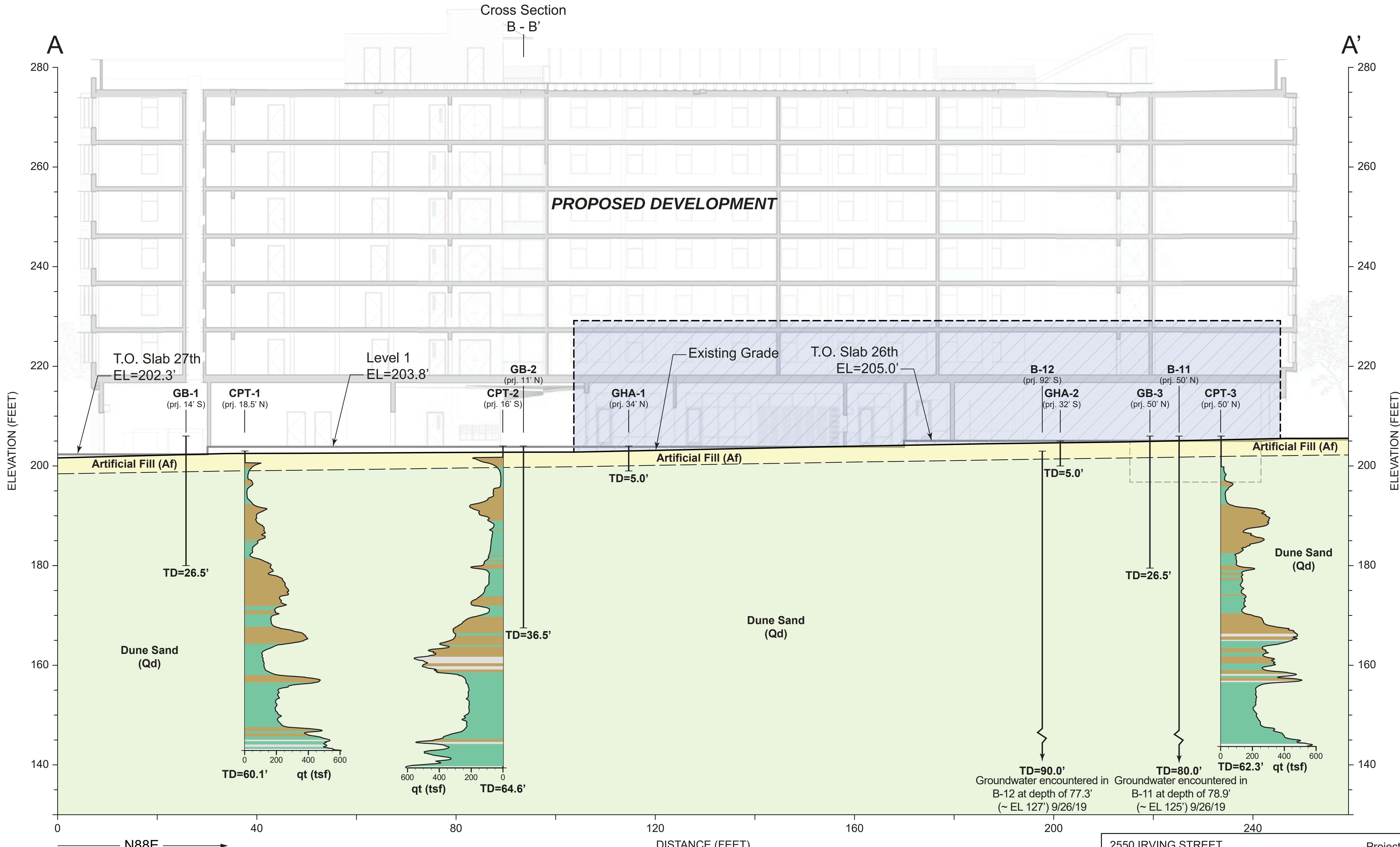
Project No. 1146-4B

**SITE PLAN**

A3GEO

**FIGURE 3**

Date: 3/20/2022




See Figure 3 for explanation of CPT symbols

Existing Building

(SCALE IN FEET)
0 20 40

**Notes**:
1. Elevation from TNDC Drawing Set "2550 Irving St PRJ Submittal - Preliminary - Not For Construction" Revision 12/3/21.
2. Locations of features shown should be considered approximate.
3. Elevation based on City of San Francisco Vertical Datum of 2013 (SFVD13)

2550 IRVING STREET
SAN FRANCISCO, CALIFORNIA

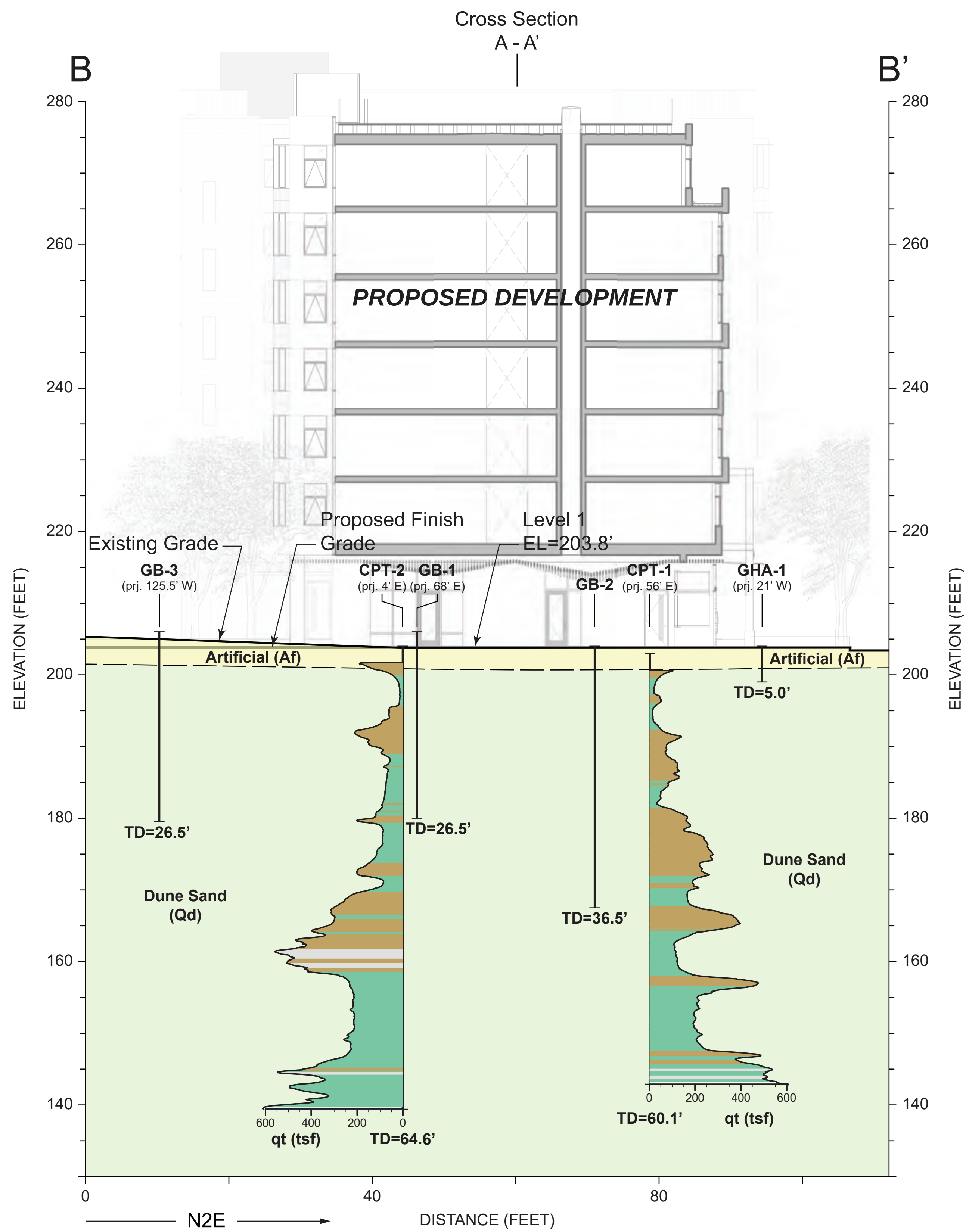
Project No. 1146-4B

**GEOLOGIC CROSS SECTION A - A'**

A3GEO

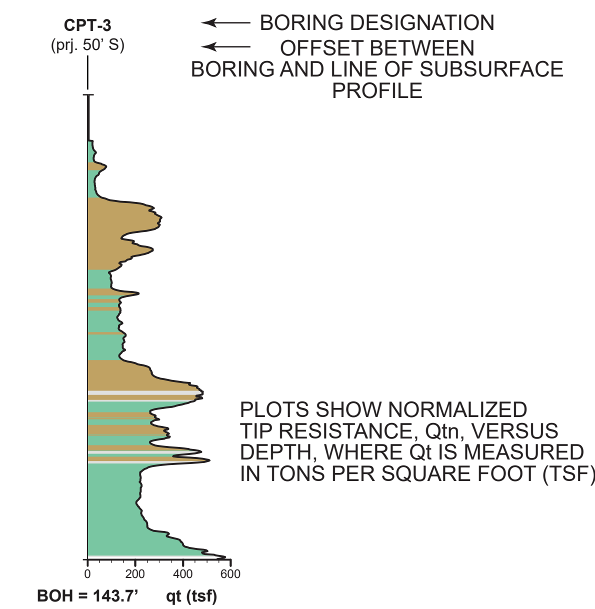
**FIGURE 4**

Date: 3/20/2022




COLORS INDICATE NORMALIZED SOIL BEHAVIOR TYPE ACCORDING TO THE LEGEND BELOW:

LEGEND
- Sensitive fine grained
- Organic material
- Clay
- Clayey silt to silty clay
- Silty sand to sandy silt
- Clean sand to silty sand
- Gravely sand to sand
- Very stiff sand to clayey sand
- Very stiff fine grained




(SCALE IN FEET)
0 20 40

**Notes**:
1. Elevation from TNDC Drawing Set "2550 Irving St PRJ Submittal - Preliminary - Not For Construction" Revision 12/3/21.
2. Locations of features shown should be considered approximate.
3. Elevation based on City of San Francisco Vertical Datum of 2013 (SFVD13)

2550 IRVING STREET
SAN FRANCISCO, CALIFORNIA

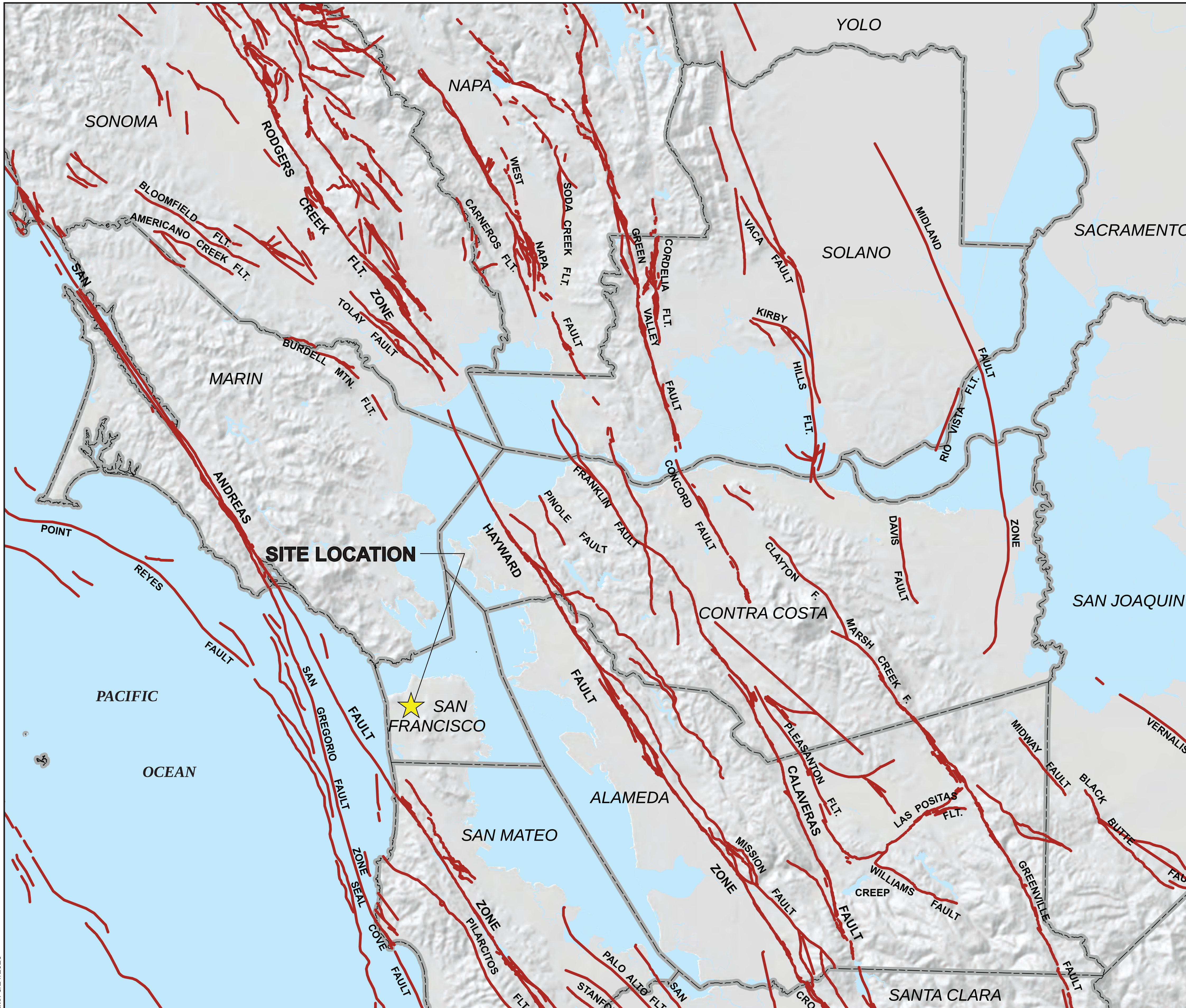
Project No. 1146-4B

**GEOLOGIC CROSS SECTION B - B'**

A3GEO

**FIGURE 5**

Date: 3/20/2022




LEGEND:

QUATERNARY FAULT

COUNTY BOUNDARY

NOTES:

1. Base map modified from USGS and California Geological Survey, 2006, Quaternary fault and fold database for the United States, accessed 25 January, 2018, from USGS website: http://earthquake.usgs.gov/hazards/qfaults/

N

(SCALE IN MILES)

0 8 16

2550 IRVING STREET
SAN FRANCISCO, CALIFORNIA

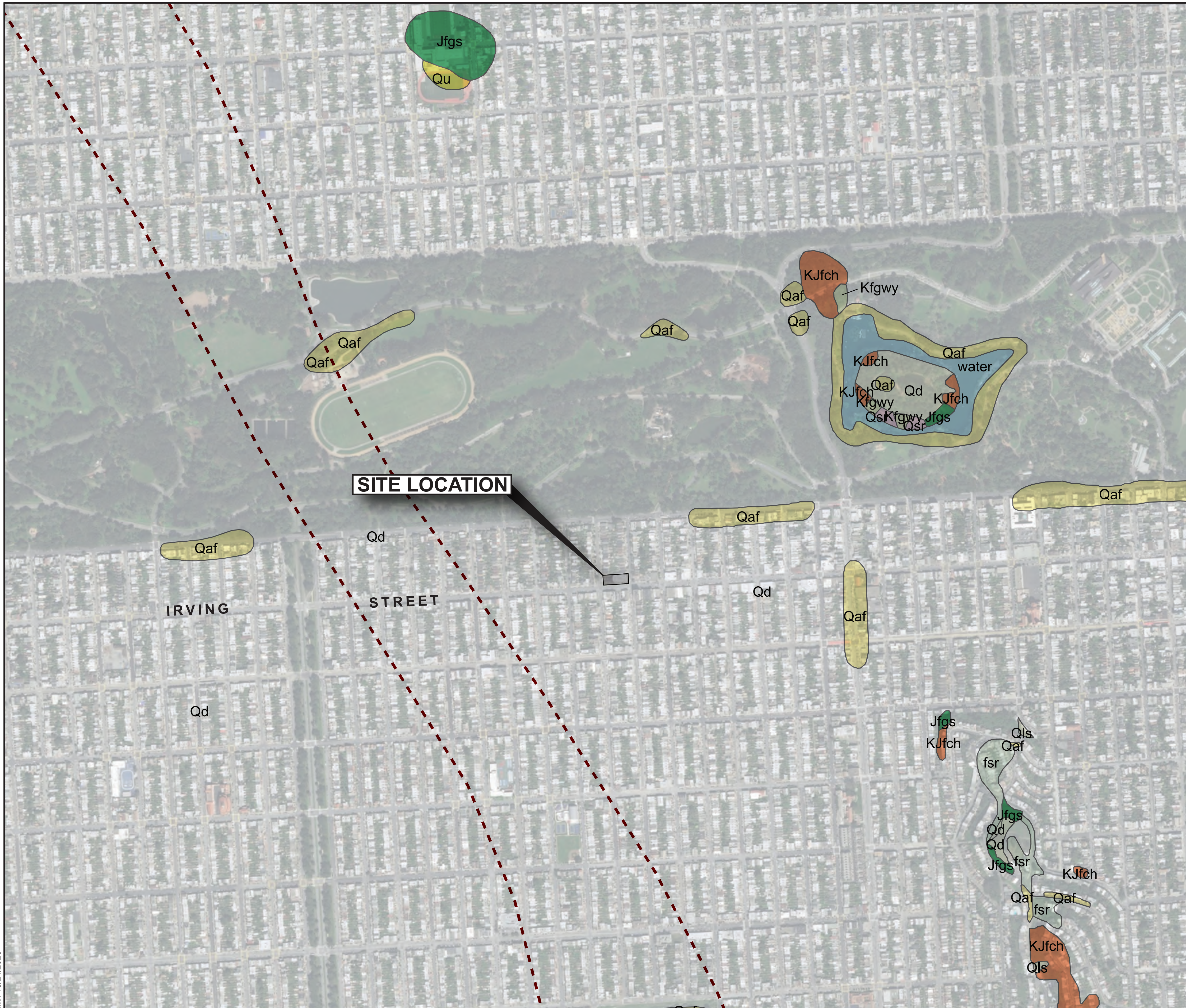
Project No. 1146-4B

## QUATERNARY FAULT MAP

A3GEO

FIGURE 6

Date: 8/21/2020




Date: 18/21/2020

**LEGEND:**

**SURFICIAL DEPOSITS (QUATERNARY)**

- **Qaf** ARTIFICIAL FILL
- **Qd** DUNE SAND
- **Qls** LANDSLIDE DEPOSITS
- **Qu** UNDIFFERENTIATED SURFICIAL DEPOSITS

**BEDROCK**
**FRANCISCAN COMPLEX**

- **Kfgwy** GRAYWACKE (CRETACEOUS)
- **KJfch** CHERT (CRETACEOUS & JURASSIC)
- **Jfgs** GREENSTONE (JURASSIC)
- **fsr** MELANGE

**MAP SYMBOLS**

- GEOLOGIC CONTACT
- FAULT APPROX. LOCATED

**Notes:**

1. MAP MODIFIED FROM BLAKE, M.C., 2000, GEOLOGIC MAP AND MAP DATABASE OF PARTS OF MARIN, SAN FRANCISCO, ALAMEDA, CONTRA COSTA, AND SONOMA COUNTIES, CALIFORNIA, USGS MISCELLANEOUS FIELD STUDY MF-2337.

N

(SCALE IN FEET)

0 1,000 2,000

2550 IRVING STREET
SAN FRANCISCO, CALIFORNIA

Project No. 1146-4B

**REGIONAL GEOLOGIC MAP**

A3GEO

**FIGURE 7**


**LEGEND:**

**EARTHQUAKE-INDUCED LANDSLIDES**

AREAS WHERE PREVIOUS OCCURRENCE OF LANDSLIDE MOVEMENT, OR LOCAL TOPOGRAPHIC, GEOLOGICAL, GEOTECHNICAL AND SUBSURFACE WATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(C) WOULD BE REQUIRED

**Notes:**

1. MODIFIED FROM CALIFORNIA GEOLOGICAL SURVEY (CGS), SEISMIC HAZARD ZONES, CITY AND COUNTY OF SAN FRANCISCO, RELEASED: NOVEMBER 17, 2000

(SCALE IN FEET)

0 1,000 2,000

2550 IRVING STREET
SAN FRANCISCO, CALIFORNIA

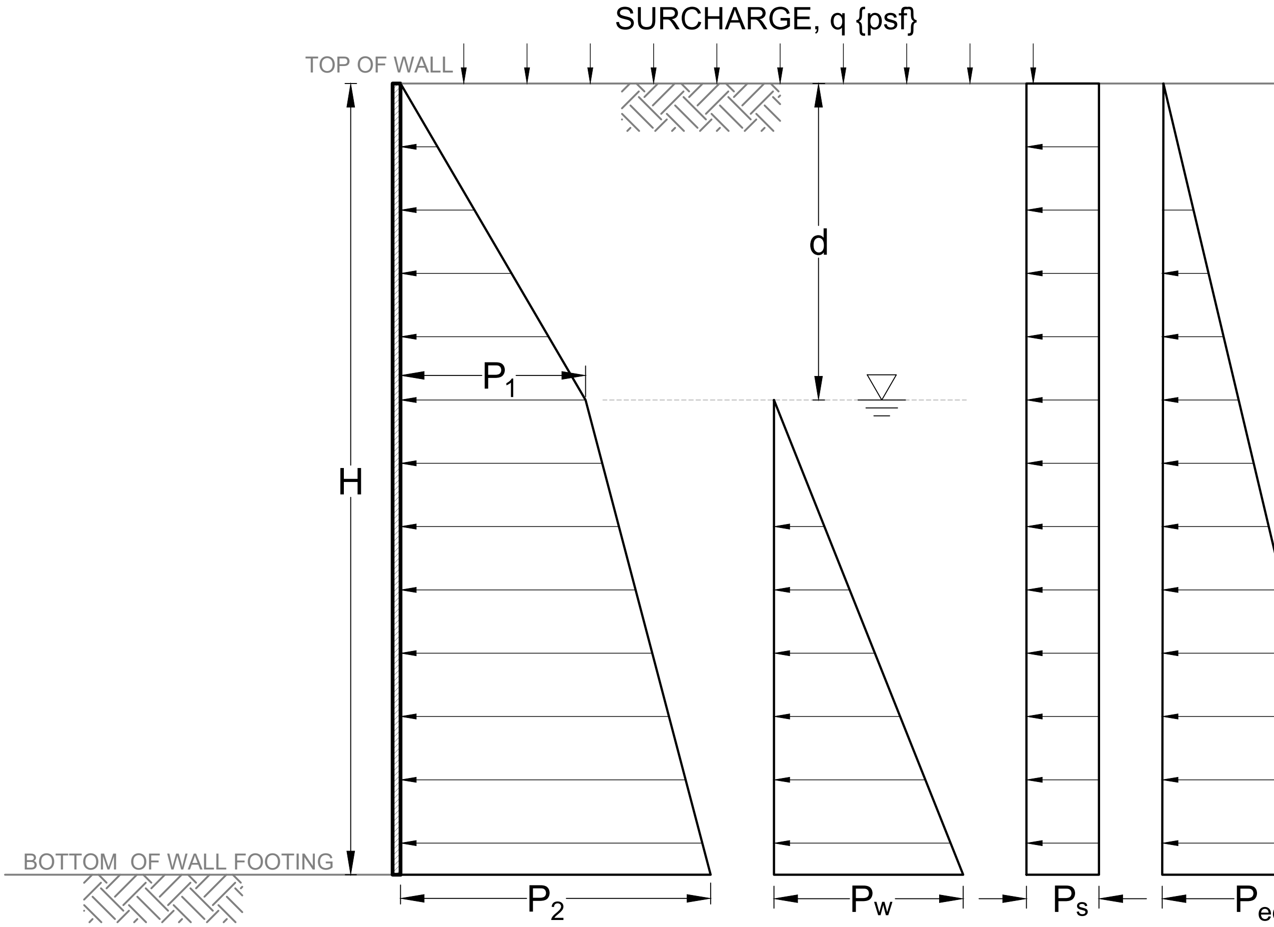
Project No. 1146-4B

**SEISMIC HAZARD ZONES OF REQUIRED INVESTIGATION**

A3GEO

**FIGURE 8**

Date: 18/21/2020




NOTES:

1. $P_1 = 40*d$ {psf}
2. $P_2 = P_1 + 19*(H-d)$ {psf}
3. $P_w = 62.4*(H-d)$ {psf}
4. $P_S = \frac{1}{3} * q$ {psf}
5. $P_{eq} = 20*H$ {psf}
6. d = DEPTH TO BOTTOM OF WALL BACKDRAIN

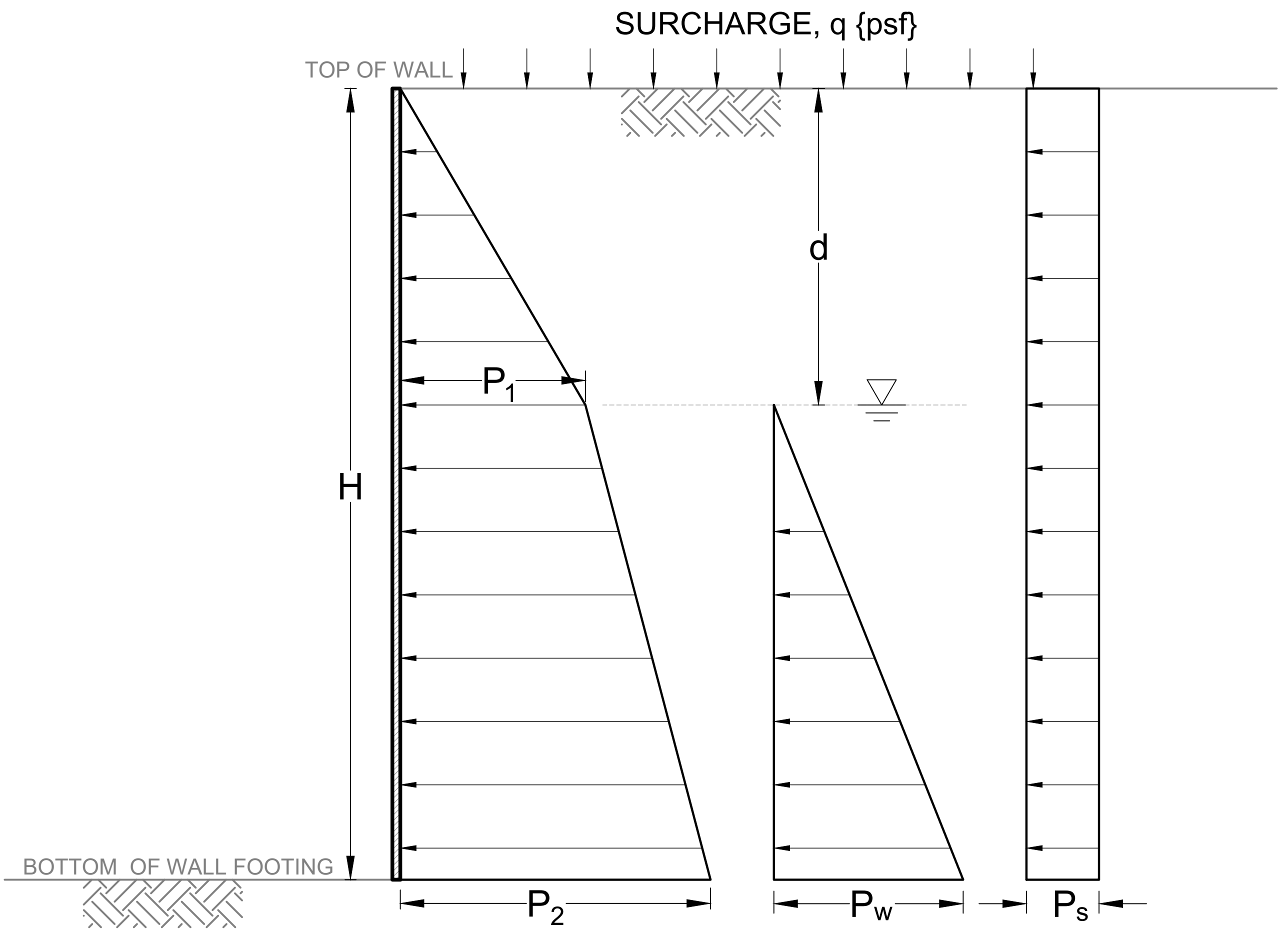
2550 IRVING STREET
SAN FRANCISCO, CALIFORNIA

PR0JECT NO. 1146-4B

**RETAINING WALL LATERAL EARTH PRESSURE DISTRIBUTION - ACTIVE (CANTILEVER)**

A3GEO

**FIGURE 9**




NOTES:

1. $P_1 = 60*d$ {psf}
2. $P_2 = P_1 + 29*(H-d)$ {psf}
3. $P_w = 62.4*(H-d)$ {psf}
4. $P_S = \frac{1}{2} * q$ {psf}
5. d = DEPTH TO BOTTOM OF WALL BACKDRAIN

| | |
|---|---|
| 2550 IRVING STREET<br>SAN FRANCISCO, CALIFORNIA | PR0JECT NO. 1146-4B |

**RETAINING WALL LATERAL EARTH PRESSURE DISTRIBUTION - AT-REST (RESTRAINED)**

A3GEO

**FIGURE 10**

# APPENDIX A
# A3GEO Exploratory Boring Logs

## UNIFIED SOIL CLASSIFICATION CHART

| MAJOR DIVISIONS | | | | TYPICAL NAMES |
|---|---|---|---|---|
| COARSE GRAINED SOILS: more than 50% retained on No. 200 sieve | COARSE GRAINED SOILS: 50% or more of coarse fraction on No. 4 sieve | CLEAN GRAVELS | GW | Well graded gravels and gravel-sand mixtures, little or no fines |
| | | | GP | Poorly graded gravels and gravel-sand mixtures, little or no fines |
| | | GRAVELS WITH SAND | GM | Silty gravels and gravel-sand-silt mixtures |
| | | | GC | Clayey gravels and gravel-sand-clay mixtures |
| | SANDS: more than 50% passing on No. 4 sieve | CLEAN SANDS | SW | Well graded sands and gravelly sand, little or no fines |
| | | | SP | Poorly graded sands and gravelly sand, little or no fines |
| | | SANDS WITH FINES | SM | Silty sands, sand-silt mixtures |
| | | | SC | Clayey sands, sand-clay mixtures |
| FINE GRAINED SOILS: 50% or more passing No. 200 sieve | SILTS AND CLAY: Liquid Limit 50% or less | | ML | Inorganic silts, very fine sands, rock flour, silty or clayey fine sands |
| | | | CL | Inorganic clays or low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays |
| | | | OL | Organic silts and organic silty clays of low plasticity |
| | SILTS AND CLAY: Liquid Limit 50% or greater | | MH | Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic clays |
| | | | CH | Inorganic clays of high plasticity, fat clays |
| | | | OH | Organic clays of medium to high plasticity |
| HIGHLY ORGANIC SOILS | | | PT | Peat, muck, and other highly organic soils |

## BOUNDARY CLASSIFICATION AND GRAIN SIZES

| SILT OR CLAY | SAND | | | GRAVEL | | COBBLES | BOULDERS |
|---|---|---|---|---|---|---|---|
| | FINE | MEDIUM | COARSE | FINE | COARSE | | |

U.S. Standard No. 200 — No. 40 — No. 10 — No. 4 — 3/4" — 3" — 12"
Sieve Sizes 0.075 mm — 0.425 mm — 2 mm — 3/16"

## SYMBOLS

| | | |
|---|---|---|
| Modified California (MC) Sampler (3" O.D.) | ROCK CORE (RC) | Disturbed Sample |
| Standard Penetration Test: SPT (2" O.D.) | Shelby Tube, pushed or used Ostenberg Sampler | Water Levels<br>At time of drilling<br>At end of drilling<br>After drilling |

## ABBREVIATIONS

| Item | Meaning |
|---|---|
| LL | Liquid Limit (%) (ASTM D 4318) |
| PI | Plasticity Index (%) (ASTM D 4318) |
| -200 | Passing No. 200 (%) (ASTM D 1140) |
| TXCU | Laboratory consolidated undrained triaxial test of undrained shear strength (psf) (ASTM D 4767) |
| TXUU | Laboratory unconsolidated, undrained triaxial test of undrained shear strength (psf) (ASTM D 2850) |
| psf/tsf | pounds per square foot / tons per square foot |
| psi | pounds per square inch |
| OD | Outside Diameter |
| ID | Inside Diameter |

## NOTES

1. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
2. Modified California (MC) blow counts were adjusted by multiplying field blow counts by a factor of 0.63.
3. Recorded blow counts have not been adjusted for hammer energy.

A3GEO

A3GEO, Inc.
821 Bancroft Way
Berkeley, CA 94710
Telephone: 510-705-1664

# BORING NUMBER GB-1



**CLIENT** Tenderloin Neighborhood Development Corporation
**PROJECT NAME** 2550 Irving St Project
**PROJECT NUMBER** 1146-4B
**PROJECT LOCATION** San Francisco, CA
**DATE STARTED** 2/17/22 **COMPLETED** 2/17/22
**GROUND ELEVATION** 206 ft SFVD13 **HOLE SIZE** 6
**DRILLING CONTRACTOR** Clear Heart Drilling
**GROUND WATER LEVELS:**
**DRILLING METHOD** Hollow Stem Auger
**AT TIME OF DRILLING** --- Not Encountered
**LOGGED BY** DB **CHECKED BY**
**AT END OF DRILLING** --- Not Encountered
**NOTES**
**AFTER DRILLING** --- Not Encountered

| DEPTH (ft) | GRAPHIC LOG | MATERIAL DESCRIPTION | SAMPLE TYPE | ADJUSTED BLOW COUNTS (N VALUE) | DRY UNIT WT. (pcf) | POCKET PEN. (tsf) | MOISTURE CONTENT (%) | % RECOVERED | OTHER LAB TESTS / NOTES |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | 4" Asphalt Cement over 8" Aggregate Base | | | | | | | |
| | | **ARTIFICIAL FILL** POORLY GRADED SAND (SP) - medium dense, light brown, slightly moist, mostly fine-grained sand, trace fines | | | | | | | |
| | | **DUNE SAND** POORLY GRADED SAND (SP) - medium dense, light brown, slightly moist, mostly fine-grained sand, trace fines | MC | 13 | 104 | | 1 | 100 | Gravel=0% Sand=97% -#200=3% |
| 5 | | - loose at 4.5 ft | SPT | 6 | | | | 100 | |
| | | - medium dense at 7.5 ft | SPT | 11 | | | | 100 | |
| 10 | | - loose at 10 ft | MC | 6 | 102 | | 3 | 100 | Gravel=0% Sand=100% -#200=0% |
| 15 | | - dense at 15 ft | SPT | 31 | | | | 100 | |
| 20 | | - dense at 20 ft | SPT | 36 | | | | 100 | |
| 25 | | - loose at 25 ft, dark brown, moist | SPT | 9 | | | | 100 | |

Bottom of borehole at 26.5 feet.
1. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
2. Modified California (MC) blow counts were adjusted by multiplying field blow counts by a factor of 0.63.
3. Elevations were estimated using the "ALTA/NSPS Land Site Survey - 2550 Irving Street, City and County of San Francisco, California" drawing dated January 21, 2021 by Luk and Associates and reference San Francisco Vertical Datum of 2013 (SFVD13).
4. The hole was backfilled with cement grout according to the permit requirements.

A3GEO

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Berkeley, CA 94710
Telephone: 510-705-1664

# BORING NUMBER GB-2



**CLIENT** Tenderloin Neighborhood Development Corporation
**PROJECT NAME** 2550 Irving St Project
**PROJECT NUMBER** 1146-4B
**PROJECT LOCATION** San Francisco, CA
**DATE STARTED** 2/17/22 **COMPLETED** 2/17/22
**GROUND ELEVATION** 204 ft SFVD13 **HOLE SIZE** 6
**DRILLING CONTRACTOR** Clear Heart Drilling
**GROUND WATER LEVELS:**
**DRILLING METHOD** Hollow Stem Auger
**AT TIME OF DRILLING** --- Not Encountered
**LOGGED BY** DB **CHECKED BY**
**AT END OF DRILLING** --- Not Encountered
**NOTES**
**AFTER DRILLING** --- Not Encountered

| DEPTH (ft) | GRAPHIC LOG | MATERIAL DESCRIPTION | SAMPLE TYPE | ADJUSTED BLOW COUNTS (N VALUE) | DRY UNIT WT. (pcf) | POCKET PEN. (tsf) | MOISTURE CONTENT (%) | % RECOVERED | OTHER LAB TESTS / NOTES |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | 3" Asphalt Cement over 4" Aggregate Base | | | | | | | |
| | | **FILL**<br>POORLY GRADED SAND (SP) - medium dense, light brown, slightly moist, mostly fine-grained sand, trace fines<br>- pieces of steel and concrete debris found at 2 ft | SPT | 11 | | | | 89 | Corrosivity Test |
| 5 | | **DUNE SAND**<br>POORLY GRADED SAND (SP) - medium dense, light brown, slightly moist, mostly fine-grained sand, trace fines<br>- very loose from 5 to 10 ft | MC | 1 | 99 | | 3 | 100 | Gravel=0%<br>Sand=99%<br>-#200=1% |
| 10 | | | SPT | 2 | | | | 100 | |
| 15 | | - medium dense from 15 to 30 ft | SPT | 14 | | | | 100 | |
| 20 | | | MC | 23 | 107 | | 3 | 100 | Gravel=0%<br>Sand=99%<br>-#200=1% |
| 25 | | | SPT | 22 | | | | 100 | |
| 30 | | | SPT | 24 | | | | 100 | |

*(Continued Next Page)*

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Telephone: 510-705-1664

# BORING NUMBER GB-2



**CLIENT** Tenderloin Neighborhood Development Corporation
**PROJECT NAME** 2550 Irving St Project
**PROJECT NUMBER** 1146-4B
**PROJECT LOCATION** San Francisco, CA
**DATE STARTED** 2/17/22 **COMPLETED** 2/17/22
**GROUND ELEVATION** 204 ft SFVD13 **HOLE SIZE** 6
**DRILLING CONTRACTOR** Clear Heart Drilling
**GROUND WATER LEVELS:**
**DRILLING METHOD** Hollow Stem Auger
**AT TIME OF DRILLING** --- Not Encountered
**LOGGED BY** DB **CHECKED BY**
**AT END OF DRILLING** --- Not Encountered
**NOTES**
**AFTER DRILLING** --- Not Encountered

| DEPTH (ft) | GRAPHIC LOG | MATERIAL DESCRIPTION | SAMPLE TYPE | ADJUSTED BLOW COUNTS (N VALUE) | DRY UNIT WT. (pcf) | POCKET PEN. (tsf) | MOISTURE CONTENT (%) | % RECOVERED | OTHER LAB TESTS / NOTES |
|---|---|---|---|---|---|---|---|---|---|
| | | **DUNE SAND**<br>POORLY GRADED SAND (SP) - medium dense, light brown, slightly moist, mostly fine-grained sand, trace fines *(continued)* | | | | | | | |
| 35 | | - dense at 35 ft | SPT | 40 | | | | 100 | Gravel=0%<br>Sand=95%<br>-#200=5% |

Bottom of borehole at 36.5 feet.
1. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
2. Modified California (MC) blow counts were adjusted by multiplying field blow counts by a factor of 0.63.
3. Elevations were estimated using the "ALTA/NSPS Land Site Survey - 2550 Irving Street, City and County of San Francisco, California" drawing dated January 21, 2021 by Luk and Associates and reference San Francisco Vertical Datum of 2013 (SFVD13).
4. The hole was backfilled with cement grout according to the permit requirements.

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# BORING NUMBER GB-3



**CLIENT** Tenderloin Neighborhood Development Corporation
**PROJECT NAME** 2550 Irving St Project
**PROJECT NUMBER** 1146-4B
**PROJECT LOCATION** San Francisco, CA
**DATE STARTED** 2/17/22 **COMPLETED** 2/17/22
**GROUND ELEVATION** 206 ft SFVD13 **HOLE SIZE** 6
**DRILLING CONTRACTOR** Clear Heart Drilling
**GROUND WATER LEVELS:**
**DRILLING METHOD** Hollow Stem Auger
**AT TIME OF DRILLING** --- Not Encountered
**LOGGED BY** DB **CHECKED BY**
**AT END OF DRILLING** --- Not Encountered
**NOTES**
**AFTER DRILLING** --- Not Encountered

| DEPTH (ft) | GRAPHIC LOG | MATERIAL DESCRIPTION | SAMPLE TYPE | ADJUSTED BLOW COUNTS (N VALUE) | DRY UNIT WT. (pcf) | POCKET PEN. (tsf) | MOISTURE CONTENT (%) | % RECOVERED | OTHER LAB TESTS / NOTES |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | 6" Concrete | | | | | | | Concrete Core 0 to 0.5 ft |
| | | **FILL**<br>POORLY GRADED SAND WITH GRAVEL (GP-GM) - medium dense, light brown, dry | GB | | | | | 100 | Hand Auger 0.5 to 5 ft |
| | | POORLY GRADED SAND (SP) - loose to medium dense, light brown, slightly moist, mostly fine-grained sand, trace fines | GB | | | | | 100 | |
| 5 | | POORLY GRADED SAND (SP) - loose, light brown, slightly moist, mostly fine-grained sand, trace fines | SPT | 10 | | | | 100 | |
| | | | MC | 4 | 101 | | 2 | 100 | Gravel=0%<br>Sand=99%<br>-#200=1% |
| 10 | | | SPT | 8 | | | | 100 | |
| 15 | | - medium dense from 15 to 25 ft | MC | 26 | 109 | | 2 | 100 | Gravel=0%<br>Sand=99%<br>-#200=1% |
| 20 | | | SPT | 28 | | | | 100 | |
| 25 | | | MC | 20 | 109 | | 2 | 100 | Gravel=0%<br>Sand=99%<br>-#200=1% |

Bottom of borehole at 26.5 feet.
1. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
2. Modified California (MC) blow counts were adjusted by multiplying field blow counts by a factor of 0.63.
3. Elevations were estimated using the "ALTA/NSPS Land Site Survey - 2550 Irving Street, City and County of San Francisco, California" drawing dated January 21, 2021 by Luk and Associates and reference San Francisco Vertical Datum of 2013 (SFVD13).
4. The hole was backfilled with cement grout according to the permit requirements.

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# BORING NUMBER GHA-1



**CLIENT** Tenderloin Neighborhood Development Corporation
**PROJECT NAME** 2550 Irving St Project
**PROJECT NUMBER** 1146-4B
**PROJECT LOCATION** San Francisco, CA
**DATE STARTED** 2/10/22 **COMPLETED** 2/10/22
**GROUND ELEVATION** 204 ft SFVD13 **HOLE SIZE** 4
**DRILLING CONTRACTOR** A3GEO, Inc.
**GROUND WATER LEVELS:**
**DRILLING METHOD** Hand Auger
**AT TIME OF DRILLING** --- Not Encountered
**LOGGED BY** DB **CHECKED BY**
**AT END OF DRILLING** --- Not Encountered
**NOTES**
**AFTER DRILLING** --- Not Encountered

| DEPTH (ft) | GRAPHIC LOG | MATERIAL DESCRIPTION | SAMPLE TYPE | ADJUSTED BLOW COUNTS (N VALUE) | DRY UNIT WT. (pcf) | POCKET PEN. (tsf) | MOISTURE CONTENT (%) | % RECOVERED | OTHER LAB TESTS / NOTES |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | **FILL** | | | | | | | |
| | | POORLY GRADED SAND WITH SILT (SP-SM) - loose, grayish brown, dry, mostly fine-grained sand, with root fragments | GB | | | | | 100 | |
| | | | GB | | | | | 100 | |
| | | POORLY GRADED SAND (SP) - loose, light brown, slightly moist, mostly fine-grained sand, trace fines | GB | | | | | 100 | Gravel=0%<br>Sand=99%<br>-#200=1% |
| | | **DUNE SAND** | | | | | | | |
| | | POORLY GRADED SAND (SP) - loose, light brown, slightly moist, mostly fine-grained sand, trace fines | | | | | | | |
| 5 | | | GB | | | | | 100 | |

Bottom of borehole at 5.0 feet.
1. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
2. Elevations were estimated using the "ALTA/NSPS Land Site Survey - 2550 Irving Street, City and County of San Francisco, California" drawing dated January 21, 2021 by Luk and Associates and reference San Francisco Vertical Datum of 2013 (SFVD13).

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# BORING NUMBER GHA-2



**CLIENT** Tenderloin Neighborhood Development Corporation
**PROJECT NAME** 2550 Irving St Project
**PROJECT NUMBER** 1146-4B
**PROJECT LOCATION** San Francisco, CA
**DATE STARTED** 2/10/22 **COMPLETED** 2/10/22
**GROUND ELEVATION** 205 ft SFVD13 **HOLE SIZE** 4
**DRILLING CONTRACTOR** A3GEO, Inc.
**GROUND WATER LEVELS:**
**DRILLING METHOD** Hand Auger
**AT TIME OF DRILLING** --- Not Encountered
**LOGGED BY** DB **CHECKED BY**
**AT END OF DRILLING** --- Not Encountered
**NOTES**
**AFTER DRILLING** --- Not Encountered

| DEPTH (ft) | GRAPHIC LOG | MATERIAL DESCRIPTION | SAMPLE TYPE | ADJUSTED BLOW COUNTS (N VALUE) | DRY UNIT WT. (pcf) | POCKET PEN. (tsf) | MOISTURE CONTENT (%) | % RECOVERED | OTHER LAB TESTS / NOTES |
|---|---|---|---|---|---|---|---|---|---|
| 0 | | **FILL**<br>POORLY GRADED SAND WITH SILT (SP-SM) - loose, grayish brown, dry, mostly fine-grained sand, with root fragments | GB<br>GB | | | | | 100<br>100 | |
| | | POORLY GRADED SAND (SP) - loose, light brown, slightly moist, mostly fine-grained sand, trace fines | GB | | | | | 100 | |
| 5 | | **DUNE SAND**<br>POORLY GRADED SAND (SP) - loose, light brown, slightly moist, mostly fine-grained sand, trace fines, trace fine gravel | GB | | | | | 100 | Gravel=3%<br>Sand=95%<br>-#200=2% |

Bottom of borehole at 5.0 feet.
1. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
2. Elevations were estimated using the "ALTA/NSPS Land Site Survey - 2550 Irving Street, City and County of San Francisco, California" drawing dated January 21, 2021 by Luk and Associates and reference San Francisco Vertical Datum of 2013 (SFVD13).

# APPENDIX B
# Cone Penetration Test Logs

# PRESENTATION OF SITE INVESTIGATION RESULTS

## 2550 Irving Street, SF

*Prepared for*:

A3GEO, Inc.

ConeTec Job No: 20-56-21274

Project Start Date: 24-Aug-2020
Project End Date: 24-Aug-2020
Report Date: 25-Aug-2020


*Prepared by:*

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**Introduction**

The enclosed report presents the results of the site investigation program conducted by ConeTec Inc. for A3GEO, Inc. of Berkeley, CA. The program consisted of cone penetration testing (CPTu) at three (3) locations.

**Project Information**

| Project | |
|---|---|
| Client | A3GEO, Inc. |
| Project | 2550 Irving Street, SF |
| ConeTec Project # | 20-56-21274 |

An aerial overview from Google Earth including the CPT test locations is presented below.


| Rig Description | Deployment System | Test Type |
|---|---|---|
| CPT truck rig (C15) | 30-ton truck mounted cylinder | CPTu |

| Coordinates | | |
|---|---|---|
| Test Type | Collection Method | EPSG Number |
| CPTu | Consumer grade GPS | 32610 |

| Cone Penetrometers Used for this Project | | | | | | |
|---|---|---|---|---|---|---|
| Cone Description | Cone Number | Cross Sectional Area ($cm^2$) | Sleeve Area ($cm^2$) | Tip Capacity (bar) | Sleeve Capacity (bar) | Pore Pressure Capacity (psi) |
| 483:T1500F15U500 | 483 | 15 | 225 | 1500 | 15 | 500 |
| The CPT summary shows the cone used on each sounding. | | | | | | |

| Cone Penetration Test | |
|---|---|
| Depth reference | Depths are referenced to the existing ground surface at the time of test. |
| Tip and sleeve data offset | 0.1 Meter<br>This has been accounted for in the CPT data files. |
| Additional Comments | Advanced plots with Ic, Phi, Su(Nkt), and N1(60)Ic, as well as Soil Behavior Type (SBT) Scatter plots have been included in the data release package. |

| Calculated Geotechnical Parameter Tables | |
|---|---|
| Additional information | The Normalized Soil Behaviour Type Chart based on $Q_{tn}$ (SBT $Q_{tn}$) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPTu parameters have been generated and are provided in Excel format files in the release folder. The CPTu parameter calculations are based on values of corrected tip resistance ($q_t$) sleeve friction ($f_s$) and pore pressure ($u_2$).<br><br>Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour type zones and the assumed equilibrium pore pressure profile.<br><br>Soils were classified as either drained or undrained based on the $Q_{tn}$ Normalized Soil Behaviour Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4). |

**Limitations**

This report has been prepared for the exclusive use of A3GEO, Inc. (Client) for the project titled “2550 Irving Street, SF”. The report’s contents may not be relied upon by any other party without the express written permission of ConeTec, Inc. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting, and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.

Cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd., a subsidiary of ConeTec.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in 5 $cm^2$, 10 $cm^2$ and 15 $cm^2$ tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first appendix. The 15 $cm^2$ penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 $cm^2$ piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross-sectional area (typically forty-four millimeter diameter over a length of thirty-two millimeter with tapered leading and trailing edges) located at a distance of 585 millimeters above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a sixty-degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "$u_2$" position (ASTM Type 2). The filter is six millimeters thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

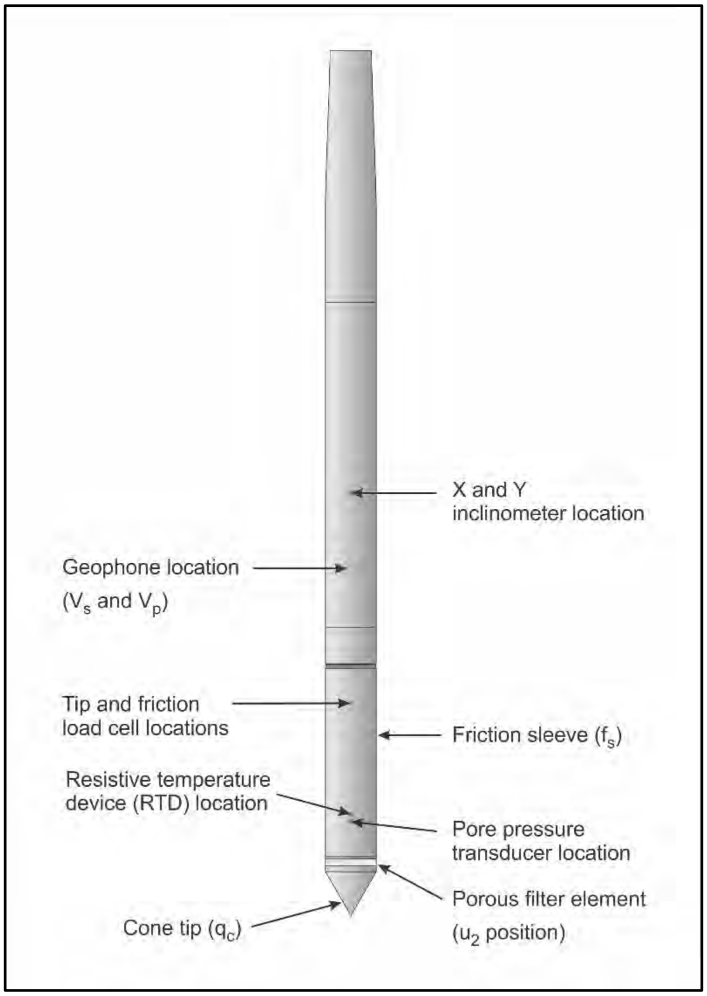



Figure CPTu. Piezocone Penetrometer (15 $cm^2$)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a sixteen bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording interval is 2.5 centimeters; custom recording intervals are possible. The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance ($q_c$)
- Sleeve friction ($f_s$)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPT operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of two centimeters per second, within acceptable tolerances. Typically, one-meter length rods with an outer diameter of 1.5 inches (38.1 millimeters) are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil under vacuum pressure prior to use
- Recorded baselines are checked with an independent multi-meter
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance ($q_t$), sleeve friction ($f_s$) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson et al. (1986) and Robertson (1990, 2009). It should be noted that it is not always possible to accurately identify a soil behavior type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behavior type.

The recorded tip resistance ($q_c$) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance ($q_t$) according to the following expression presented in Robertson et al. (1986):

$$q_t = q_c + (1\text{-}a) \bullet u_2$$

where: $q_t$ is the corrected tip resistance
$q_c$ is the recorded tip resistance
$u_2$ is the recorded dynamic pore pressure behind the tip ($u_2$ position)
a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction ($f_s$) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (Rf) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of files with calculated geotechnical parameters were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the methods used is also included in the data release folder.

For additional information on CPTu interpretations and calculated geotechnical parameters, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

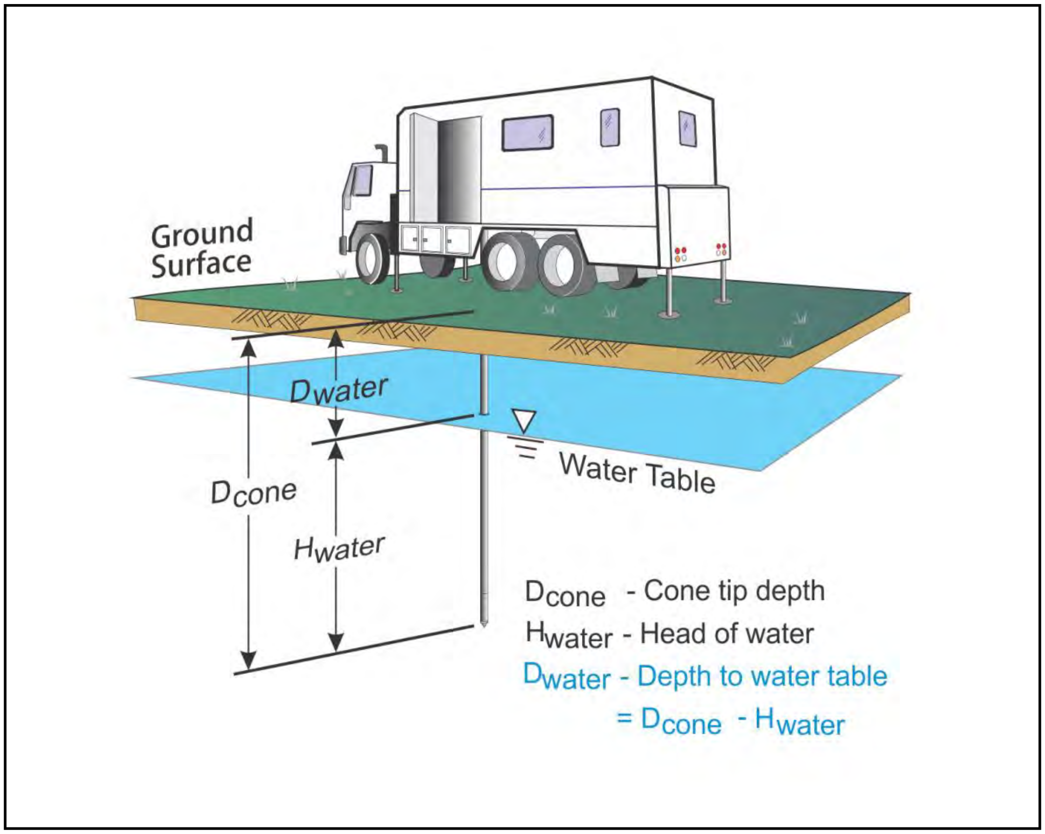



Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behavior.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

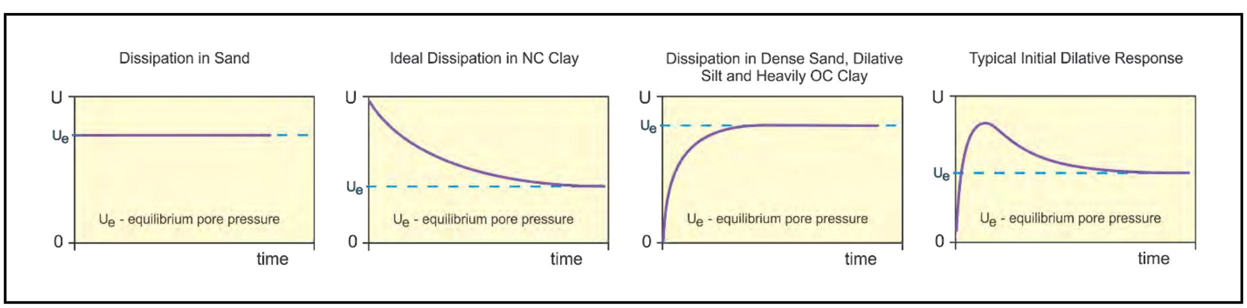



Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure ($u_{eq}$) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve in Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as $t_{100}$. In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to $t_{100}$. A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T*) may be used to calculate the coefficient of consolidation ($c_h$) at various degrees of dissipation resulting in the expression for $c_h$ shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

T* is the dimensionless time factor (Table Time Factor)

a is the radius of the cone

$I_r$ is the rigidity index

t is the time at the degree of consolidation

Table Time Factor. T* versus degree of dissipation (Teh and Houlsby (1991))

| Degree of Dissipation (%) | 20 | 30 | 40 | 50 | 60 | 70 | 80 |
|---|---|---|---|---|---|---|---|
| T* ($u_2$) | 0.038 | 0.078 | 0.142 | 0.245 | 0.439 | 0.804 | 1.60 |

The coefficient of consolidation is typically analyzed using the time ($t_{50}$) corresponding to a degree of dissipation of 50% ($u_{50}$). In order to determine $t_{50}$, dissipation tests must be taken to a pressure less than $u_{50}$. The $u_{50}$ value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as $u_{100}$. To estimate $u_{50}$, both the initial maximum pore pressure and $u_{100}$ must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at $t_{100}$) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly ($u_{100}$), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of $c_h$ (Teh and Houlsby (1991)), $t_{50}$ values are estimated from the corresponding pore pressure dissipation curve and a rigidity index ($I_r$) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining $t_{50}$. In cases where the time to peak is excessive, $t_{50}$ values are not calculated.

Due to possible inherent uncertainties in estimating $I_r$, the equilibrium pore pressure and the effect of an initial dilatory response on calculating $t_{50}$, other methods should be applied to confirm the results for $c_h$.

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM International, West Conshohocken, PA. DOI: 10.1520/D5778-12.

Burns, S.E. and Mayne, P.W., 1998, "Monotonic and dilatory pore pressure decay during piezocone tests", Canadian Geotechnical Journal 26 (4): 1063-1073. DOI: 1063-1073/T98-062.

Burns, S.E. and Mayne, P.W., 2002, "Analytical cavity expansion-critical state model cone dissipation in fine-grained soils", Soils & Foundations, Vol. 42(2): 131-137.

Jones, G.A. and Van Zyl, D.J.A., 1981, "The piezometer probe: a useful investigation tool", Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, "Cone Penetration Testing in Geotechnical Practice", Blackie Academic and Professional.

Mayne, P.W., 2013, "Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests", Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420. DOI: 10.1061/9780784412770.027.

Mayne, P.W. and Peuchen, J., 2012, "Unit weight trends with cone resistance in soft to firm clays", Geotechnical and Geophysical Site Characterization *4,* Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

Mayne, P.W., 2014, "Interpretation of geotechnical parameters from seismic piezocone tests", CPT'14 Keynote Address, Las Vegas, NV, May 2014.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27: 151-158. DOI: 10.1139/T90-014.

Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, "Estimating coefficient of consolidation from piezocone tests", Canadian Geotechnical Journal, 29(4): 539-550. DOI: 10.1139/T92-061.

Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, Volume 46: 1337-1355. DOI: 10.1139/T09-065.

Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, "An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils", Canadian Geotechnical Journal, 36(2): 369-381. DOI: 10.1139/T98-105.

Teh, C.I., and Houlsby, G.T., 1991, "An analytical study of the cone penetration test in clay", Geotechnique, 41(1): 17-34. DOI: 10.1680/geot.1991.41.1.17.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots with Ic, Su(Nkt) and N1(60)Ic
- Soil Behavior Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

| | |
|---|---|
| Job No: | 20-56-21274 |
| Client: | A3GEO, Inc. |
| Project: | 2550 Irving Street, SF |
| Start Date: | 24-Aug-2020 |
| End Date: | 24-Aug-2020 |

## *CONE PENETRATION TEST SUMMARY*

| Sounding ID | File Name | Date | Cone | Assumed Phreatic Surface[1] (ft) | Final Depth (ft) | Northing[2] (m) | Easting[2] (m) | Elevation[3] (ft) | Refer to Notation Number |
|---|---|---|---|---|---|---|---|---|---|
| CPT-01 | 20-56-21274_CP01 | 24-Aug-2020 | 483:T1500F15U500 | >60.2 | 60.12 | 4179681 | 545319 | 201 | |
| CPT-02 | 20-56-21274_CP02 | 24-Aug-2020 | 483:T1500F15U500 | >64.6 | 64.55 | 4179691 | 545330 | 203 | |
| CPT-03 | 20-56-21274_CP03 | 24-Aug-2020 | 483:T1500F15U500 | >62.4 | 62.34 | 4179709 | 545375 | 207 | |

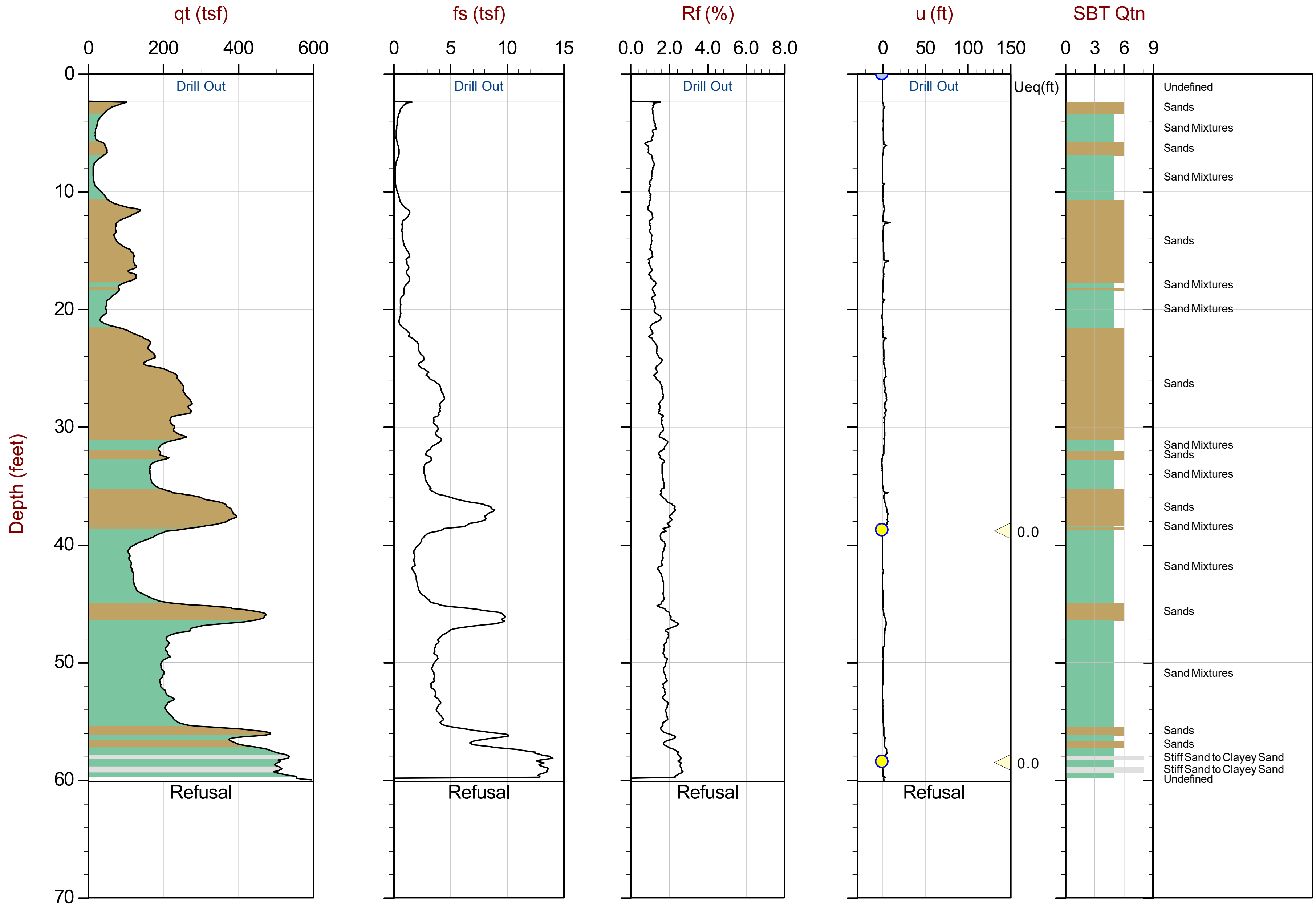
1. The assumed phreatic surface was based on the results of the shallowest pore pressure dissipation test performed within the sounding. The soundings are assumed to be dry for the calculated parameters.
2. The coordinates were acquired using consumer grade GPS equipment, datum: WGS 1984 / UTM Zone 10 North.
3. Elevations are referenced to the ground surface and are derived from the Google Earth Elevation for the recorded coordinates.

CONETEC

A3GEO

Job No: 20-56-21274
Date: 2020-08-24 07:40
Site: 2550 Irving Street, SF

Sounding: CPT-01
Cone: 483:T1500F15U500




Max Depth: 18.325 m / 60.12 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21274_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4179681m E: 545319m Elev: 201

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

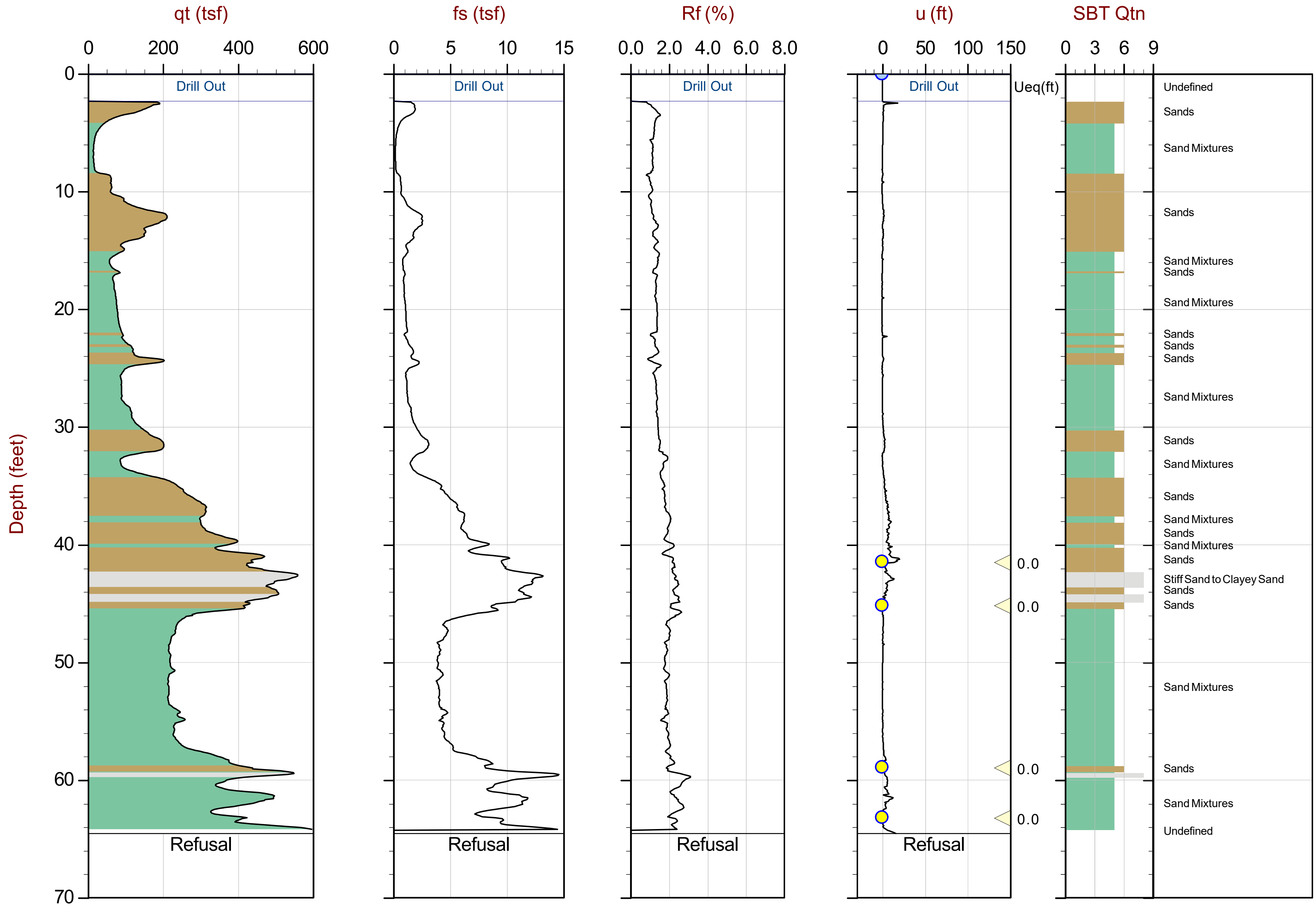
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

A3GEO

Job No: 20-56-21274
Date: 2020-08-24 09:08
Site: 2550 Irving Street, SF

Sounding: CPT-02
Cone: 483:T1500F15U500




Max Depth: 19.675 m / 64.55 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21274_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4179691m E: 545330m Elev: 203

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

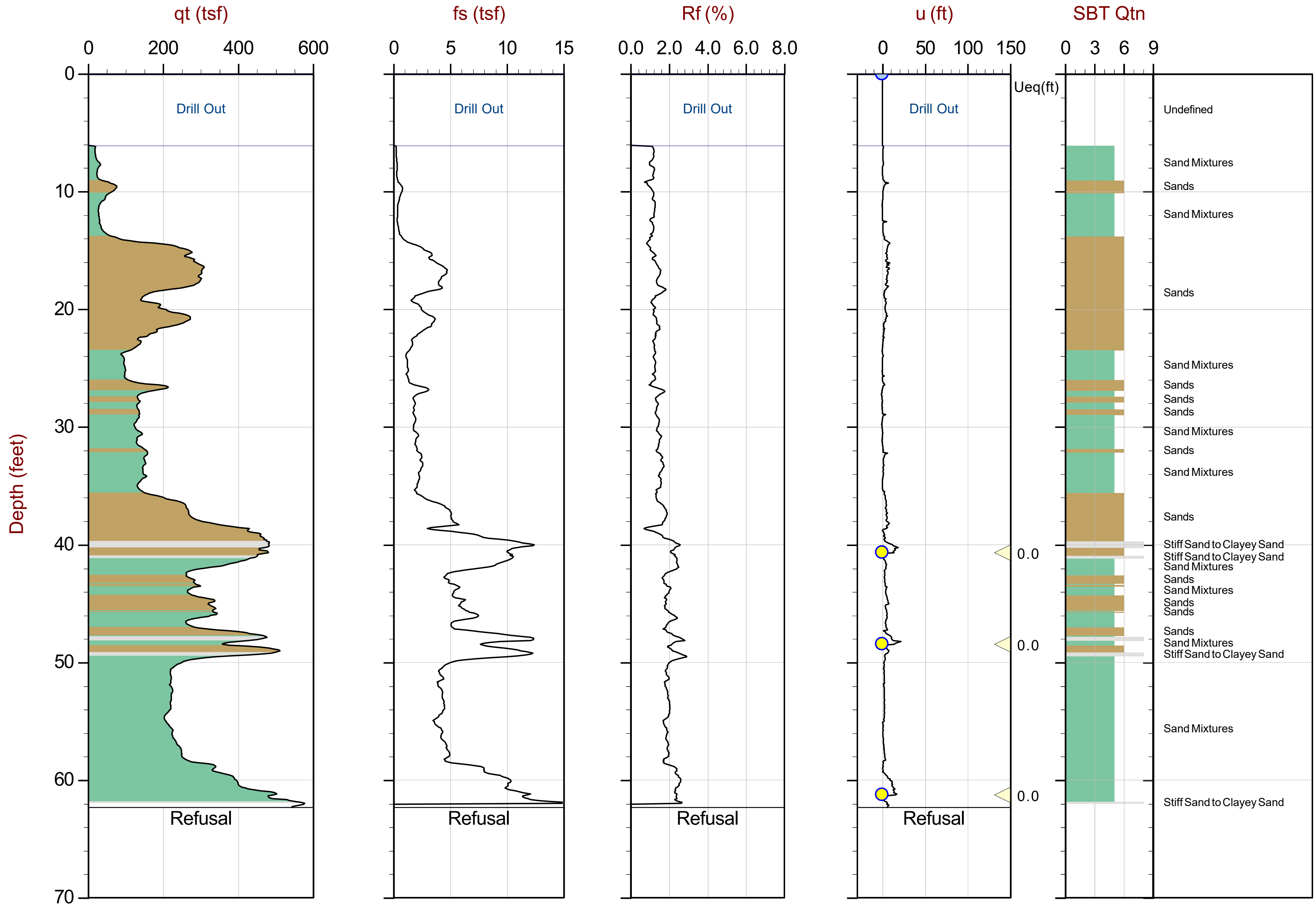
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

A3GEO

Job No: 20-56-21274
Date: 2020-08-24 11:36
Site: 2550 Irving Street, SF

Sounding: CPT-03
Cone: 483:T1500F15U500




Max Depth: 19.000 m / 62.34 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21274_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4179709m E: 545375m Elev: 207

Equilibrium Pore Pressure (Ueq) · Assumed Ueq · Dissipation, Ueq achieved · Dissipation, Ueq not achieved · Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

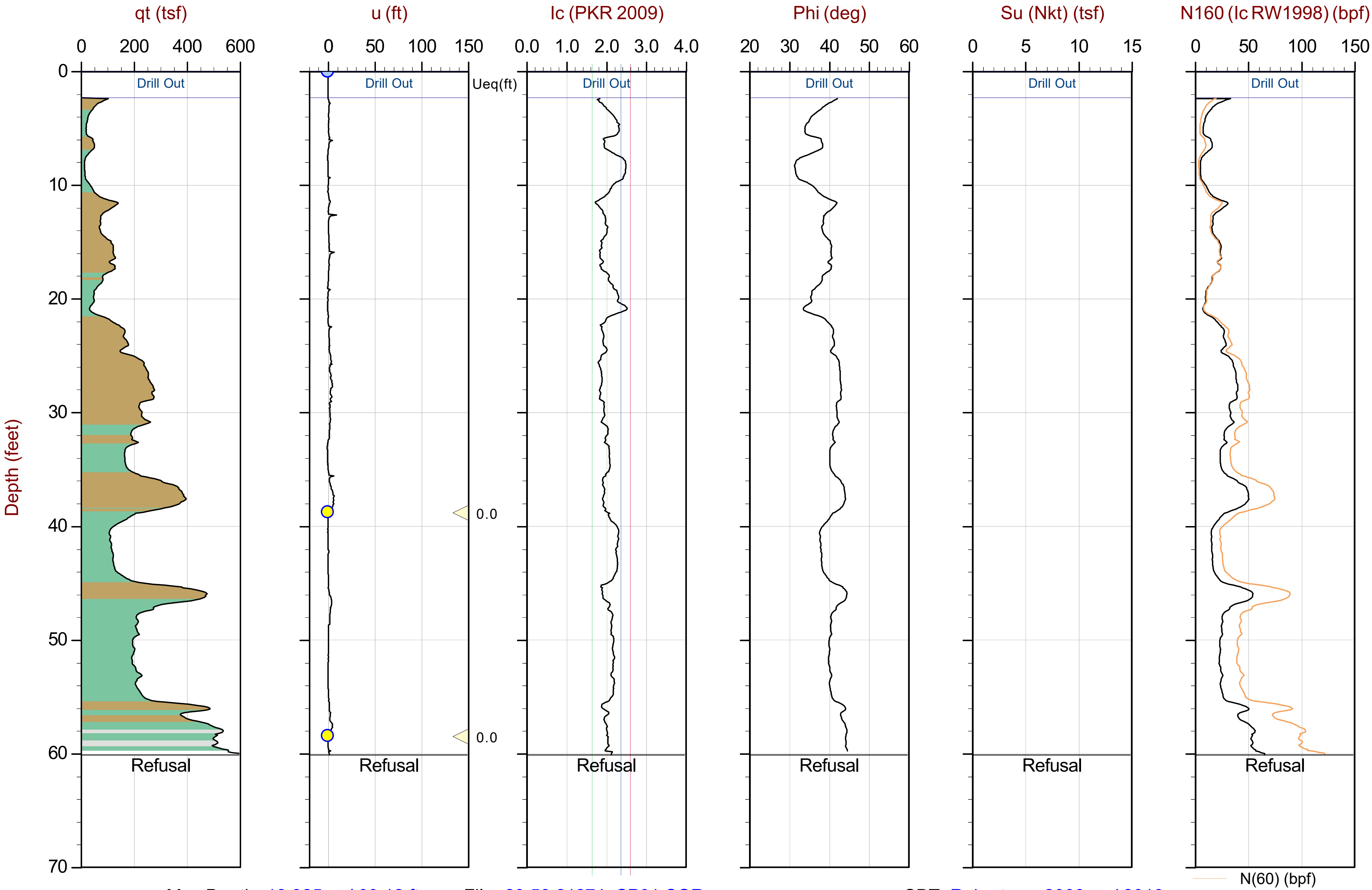
Advanced Cone Penetration Test Plots with Ic, Phi, Su(Nkt), and N1(60)Ic

CONETEC

A3GEO

Job No: 20-56-21274
Date: 2020-08-24 07:40
Site: 2550 Irving Street, SF

Sounding: CPT-01
Cone: 483:T1500F15U500




Max Depth: 18.325 m / 60.12 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21274_CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4179681m E: 545319m Elev: 201

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

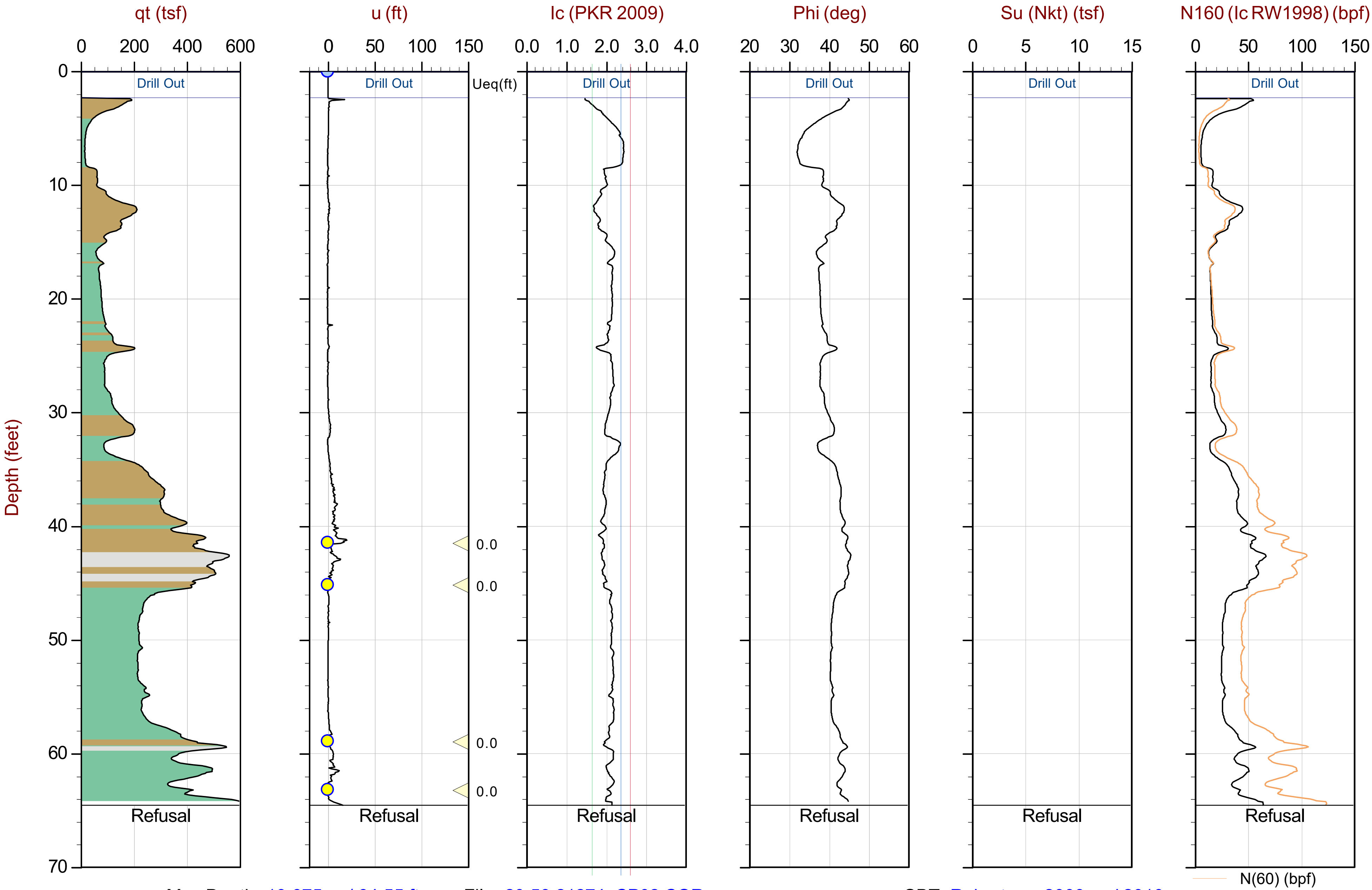
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

A3GEO

Job No: 20-56-21274
Date: 2020-08-24 09:08
Site: 2550 Irving Street, SF

Sounding: CPT-02
Cone: 483:T1500F15U500




Max Depth: 19.675 m / 64.55 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21274_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4179691m E: 545330m Elev: 203

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

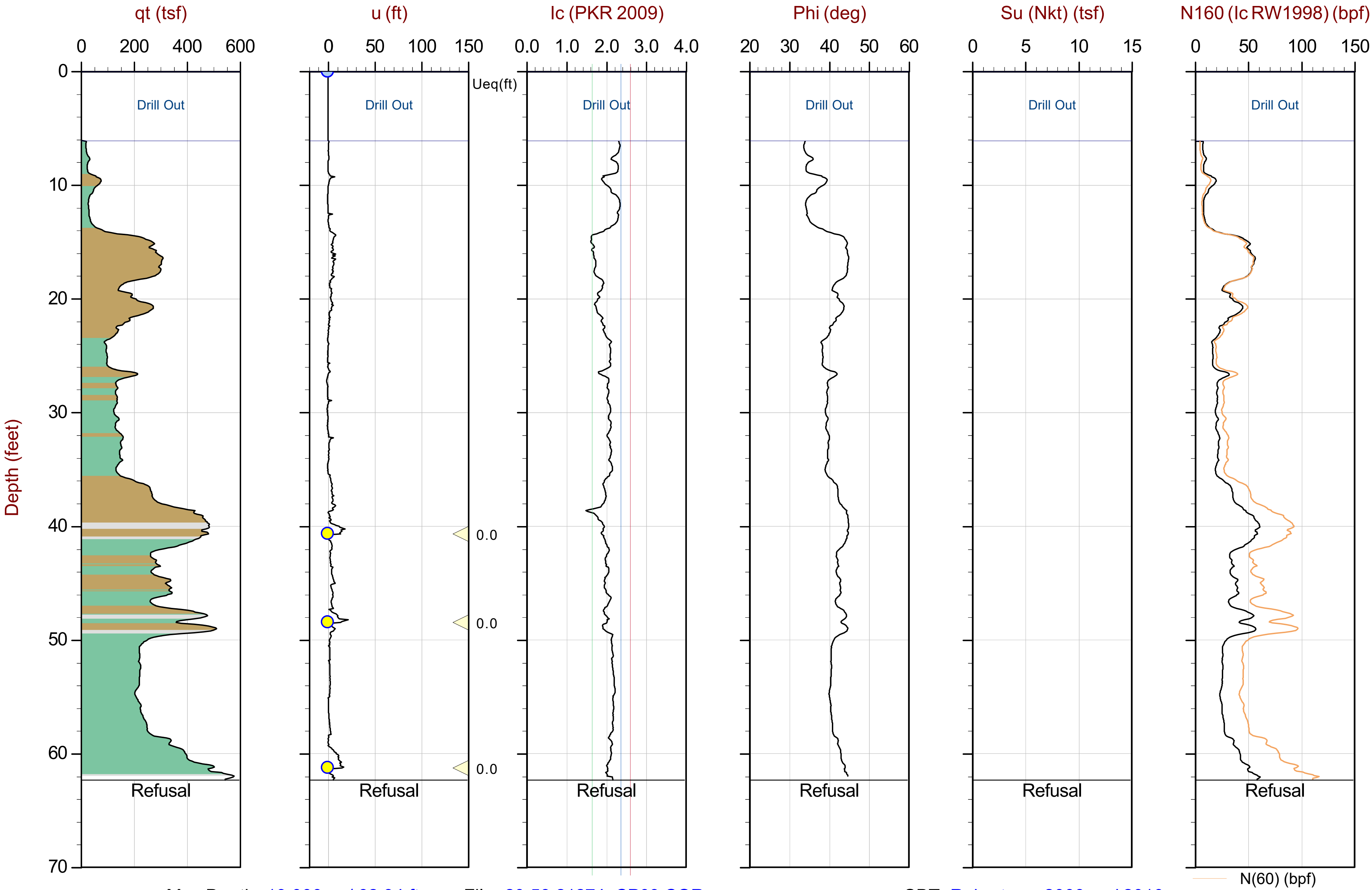
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

A3GEO

Job No: 20-56-21274
Date: 2020-08-24 11:36
Site: 2550 Irving Street, SF

Sounding: CPT-03
Cone: 483:T1500F15U500




Max Depth: 19.000 m / 62.34 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-21274_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4179709m E: 545375m Elev: 207

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

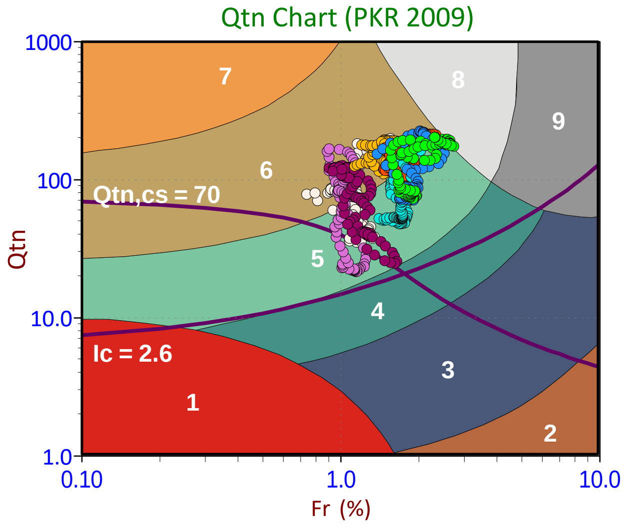
Soil Behavior Type (SBT) Scatter Plots

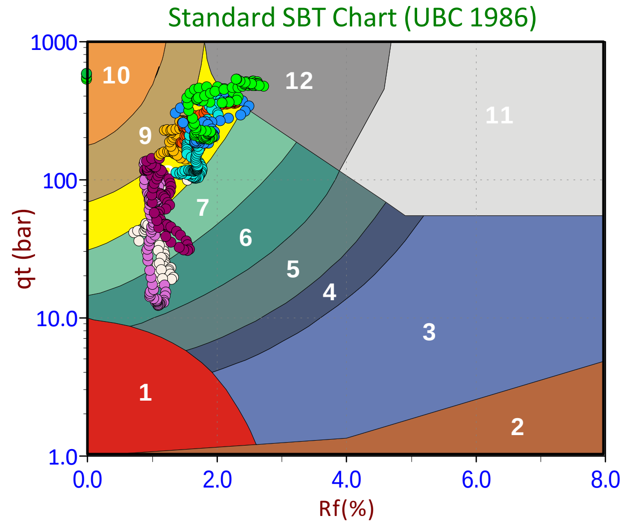
CONETEC

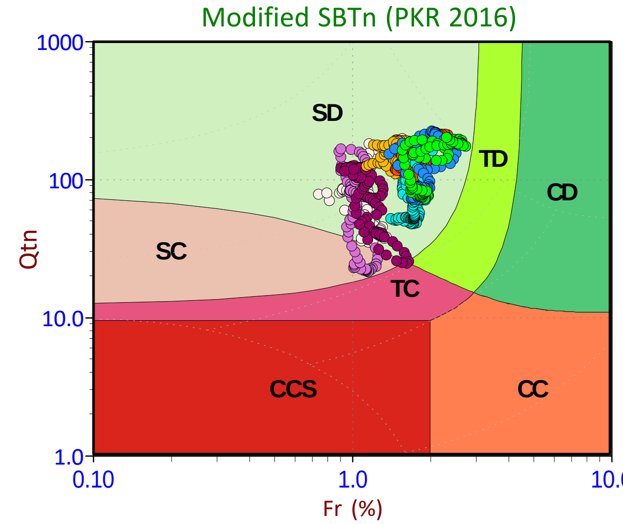
A3GEO

Job No: 20-56-21274
Date: 2020-08-24 07:40
Site: 2550 Irving Street, SF

Sounding: CPT-01
Cone: 483:T1500F15U500




Depth Ranges
- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

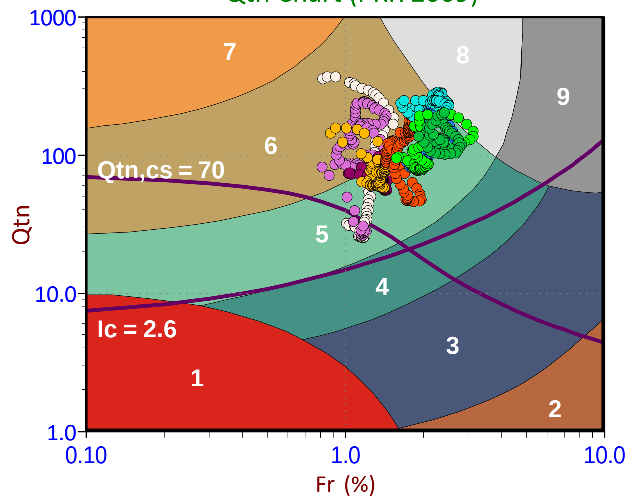
CONETEC

# A3GEO

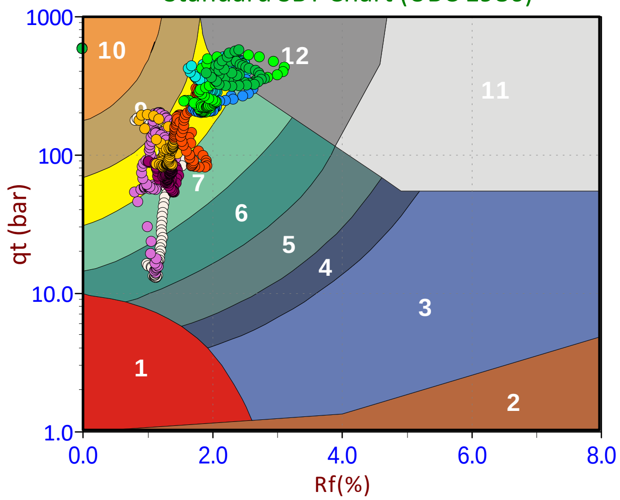
Job No: 20-56-21274
Date: 2020-08-24 09:08
Site: 2550 Irving Street, SF

Sounding: CPT-02
Cone: 483:T1500F15U500

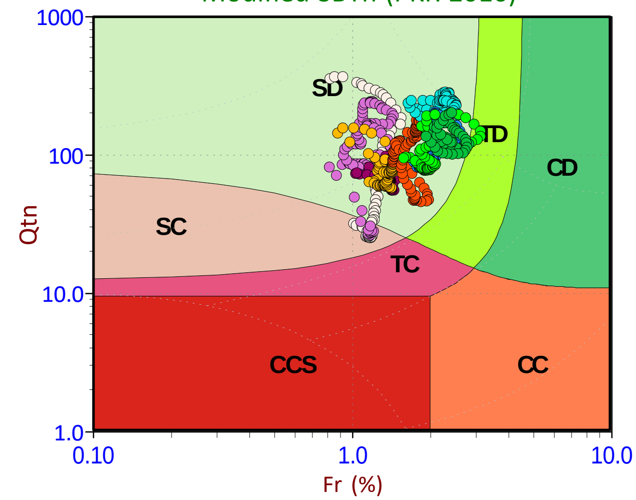
## Qtn Chart (PKR 2009)




## Standard SBT Chart (UBC 1986)




## Modified SBTn (PKR 2016)




**Depth Ranges**

- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

**Legend**

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

**Legend**

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

**Legend**

- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

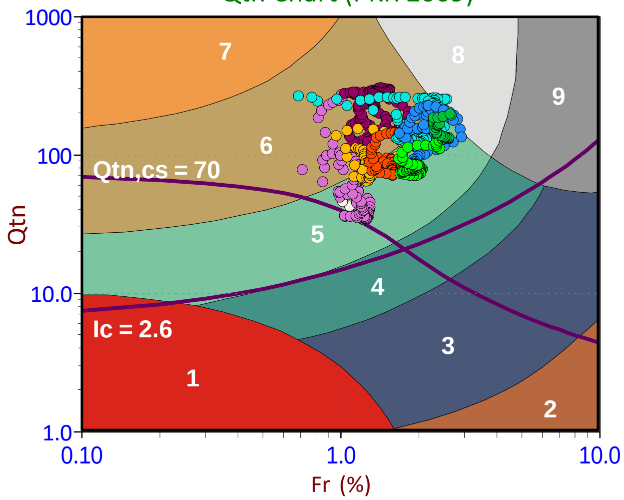
CONETEC

# A3GEO

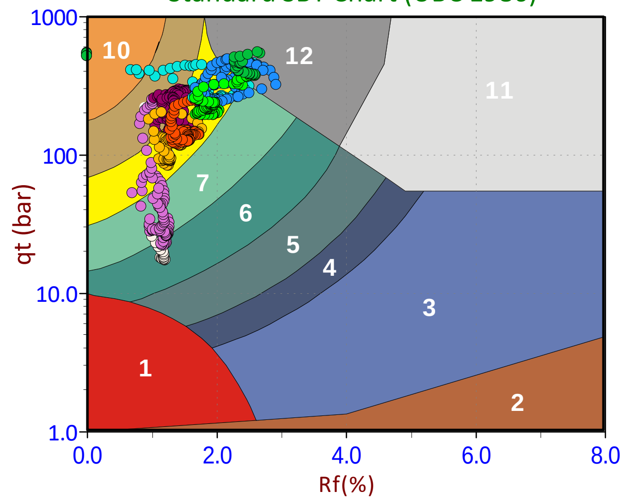
Job No: 20-56-21274
Date: 2020-08-24 11:36
Site: 2550 Irving Street, SF

Sounding: CPT-03
Cone: 483:T1500F15U500

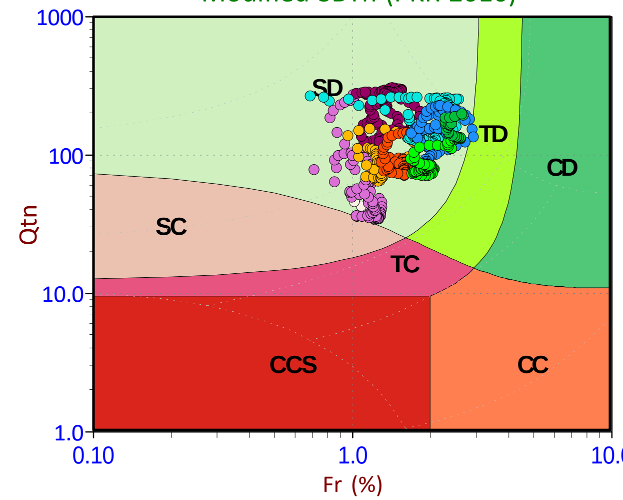
## Qtn Chart (PKR 2009)




## Standard SBT Chart (UBC 1986)




## Modified SBTn (PKR 2016)




### Depth Ranges

- >0.0 to 7.5 ft
- >7.5 to 15.0 ft
- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
- >37.5 to 45.0 ft
- >45.0 to 52.5 ft
- >52.5 to 60.0 ft
- >60.0 to 67.5 ft
- >67.5 to 75.0 ft
- >75.0 ft

### Legend

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

### Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

### Legend

- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

ConeTec

| | |
|---|---|
| Job No: | 20-56-21274 |
| Client: | A3GEO, Inc. |
| Project: | 2550 Irving Street, SF |
| Start Date: | 24-Aug-2020 |
| End Date: | 24-Aug-2020 |

## *CPTu PORE PRESSURE DISSIPATION SUMMARY*

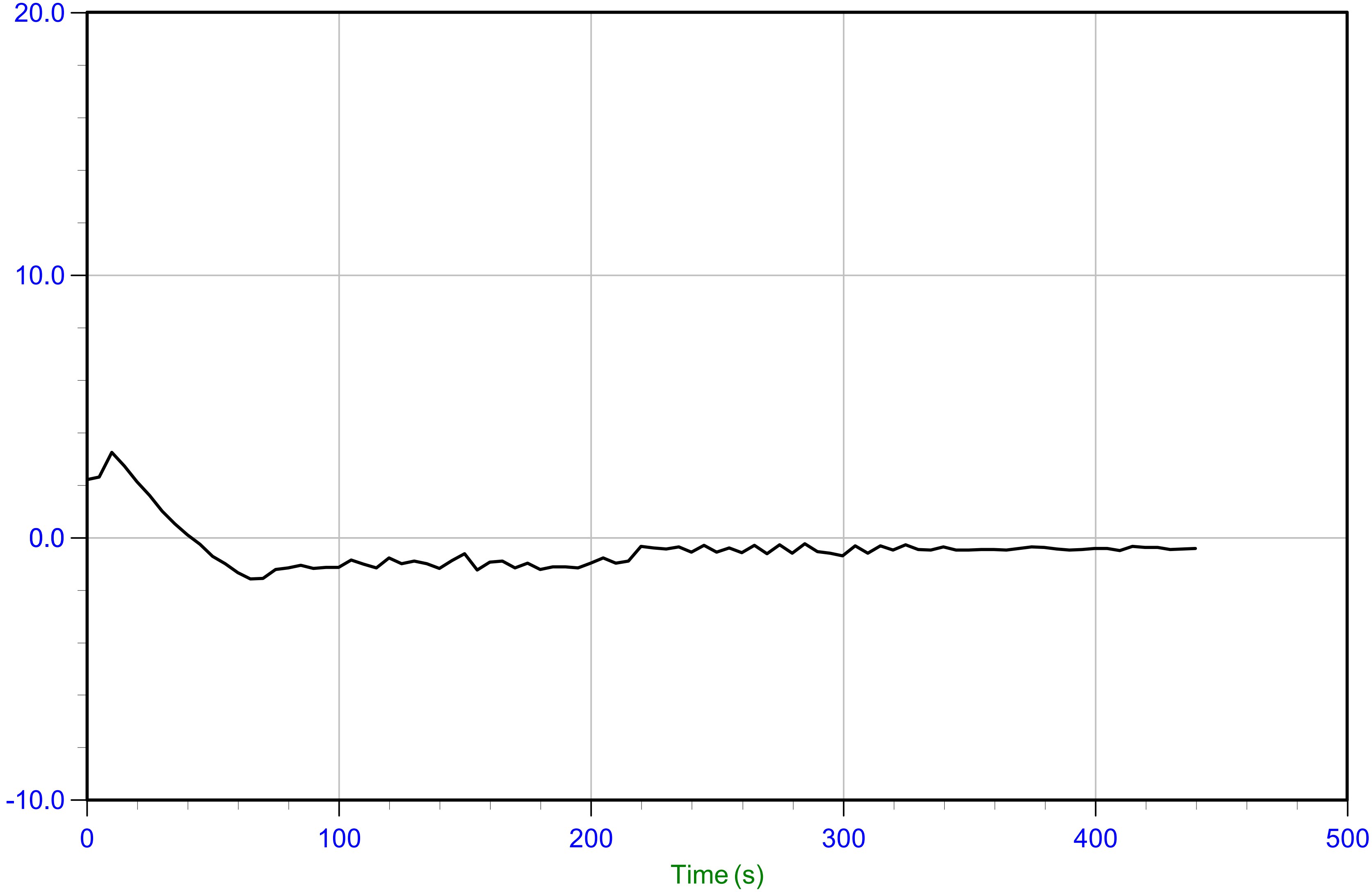
| Sounding ID | File Name | Cone Area ($cm^2$) | Duration (s) | Test Depth (ft) | Estimated Equilibrium Pore Pressure $U_{eq}$ (ft) | Calculated Phreatic Surface (ft) |
|---|---|---|---|---|---|---|
| CPT-01 | 20-56-21274_CP01 | 15 | 440 | 38.80 | 0.0 | |
| CPT-01 | 20-56-21274_CP01 | 15 | 370 | 58.48 | 0.0 | |
| CPT-02 | 20-56-21274_CP02 | 15 | 470 | 41.50 | 0.0 | |
| CPT-02 | 20-56-21274_CP02 | 15 | 400 | 45.19 | 0.0 | |
| CPT-02 | 20-56-21274_CP02 | 15 | 425 | 58.97 | 0.0 | |
| CPT-02 | 20-56-21274_CP02 | 15 | 400 | 63.24 | 0.0 | |
| CPT-03 | 20-56-21274_CP03 | 15 | 435 | 40.68 | 0.0 | |
| CPT-03 | 20-56-21274_CP03 | 15 | 560 | 48.47 | 0.0 | |
| CPT-03 | 20-56-21274_CP03 | 15 | 490 | 61.27 | 0.0 | |

CONETEC

A3GEO

Job No: 20-56-21274
Date: 08/24/2020 07:40
Site: 2550 Irving Street, SF

Sounding: CPT-01
Cone: 483:T1500F15U500 Area=15 cm²




Trace Summary:

Filename: 20-56-21274_CP01.PPF
Depth: 11.825 m / 38.795 ft
Duration: 440.0 s

u Min: -1.6 ft
u Max: 3.2 ft
u Final: -0.4 ft

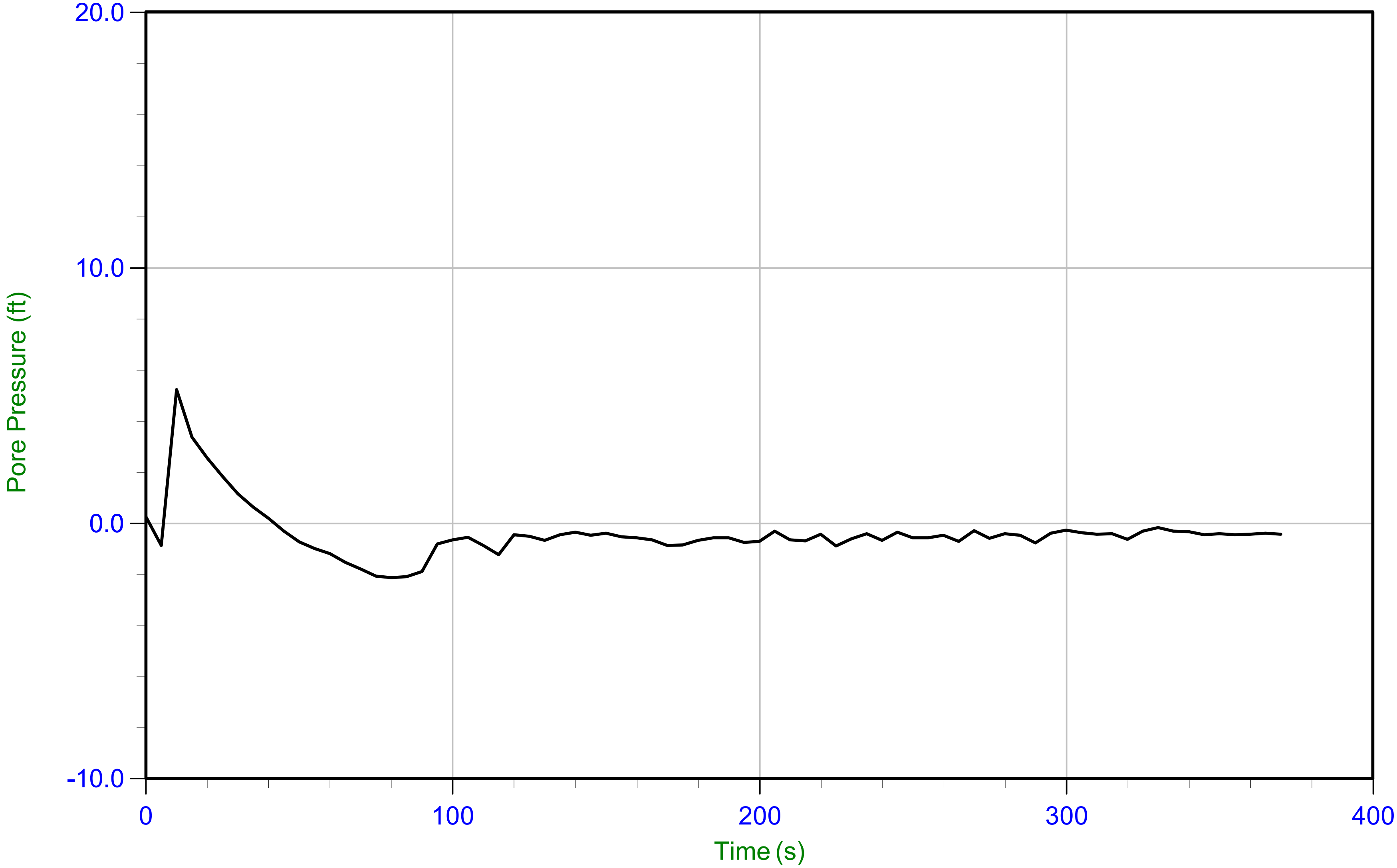
WT: 11.825 m / 38.795 ft
Ueq: 0.0 ft

CONETEC

A3GEO

Job No: 20-56-21274
Date: 08/24/2020 07:40
Site: 2550 Irving Street, SF

Sounding: CPT-01
Cone: 483:T1500F15U500 Area=15 cm²




Trace Summary:

Filename: 20-56-21274_CP01.PPF
Depth: 17.825 m / 58.480 ft
Duration: 370.0 s

u Min: -2.1 ft
u Max: 5.2 ft
u Final: -0.4 ft

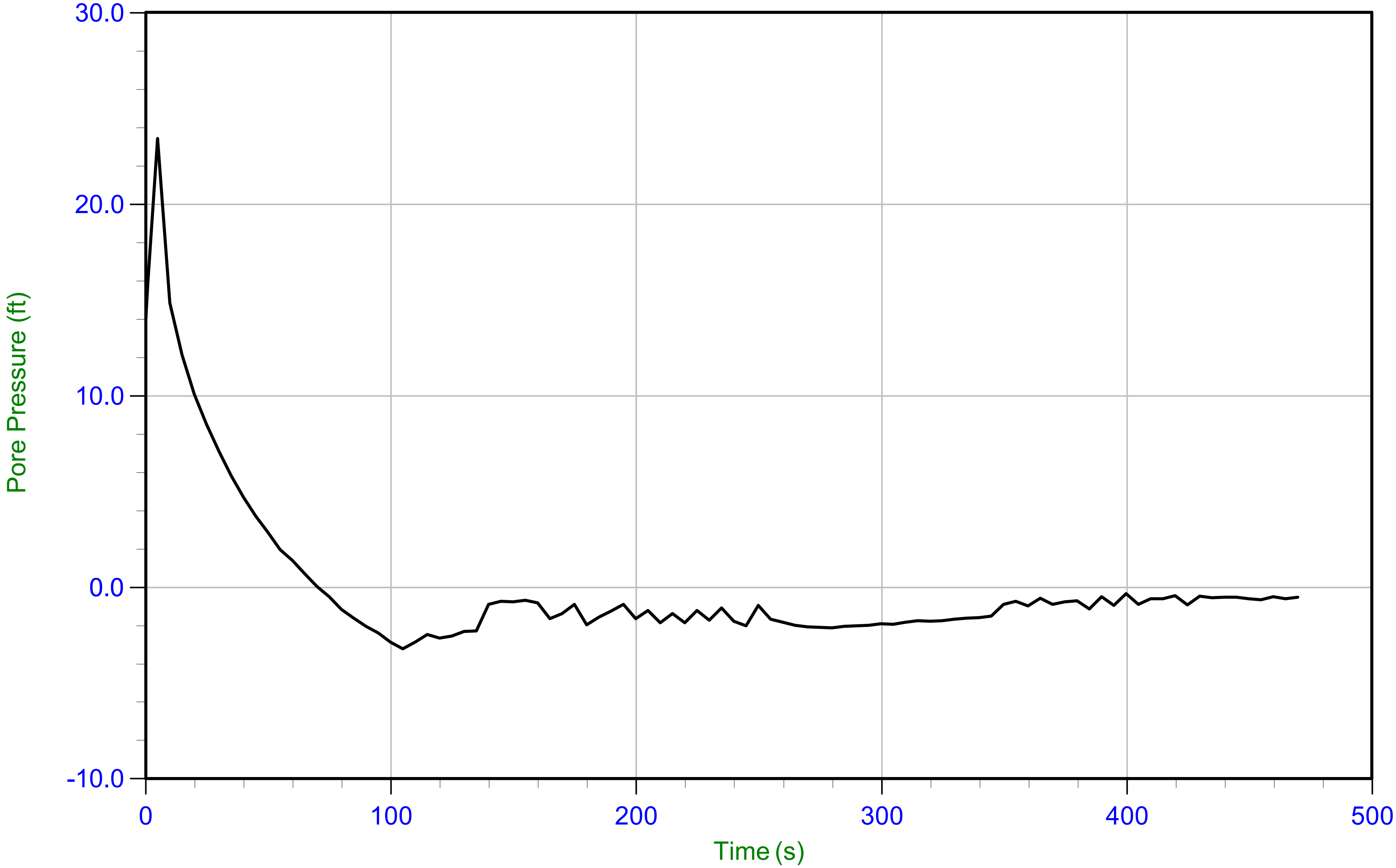
WT: 17.825 m / 58.480 ft
Ueq: 0.0 ft

CONETEC

A3GEO

Job No: 20-56-21274
Date: 08/24/2020 09:08
Site: 2550 Irving Street, SF

Sounding: CPT-02
Cone: 483:T1500F15U500 Area=15 cm²




Trace Summary:

Filename: 20-56-21274_CP02.PPF
Depth: 12.650 m / 41.502 ft
Duration: 470.0 s

u Min: -3.2 ft
u Max: 23.4 ft
u Final: -0.5 ft

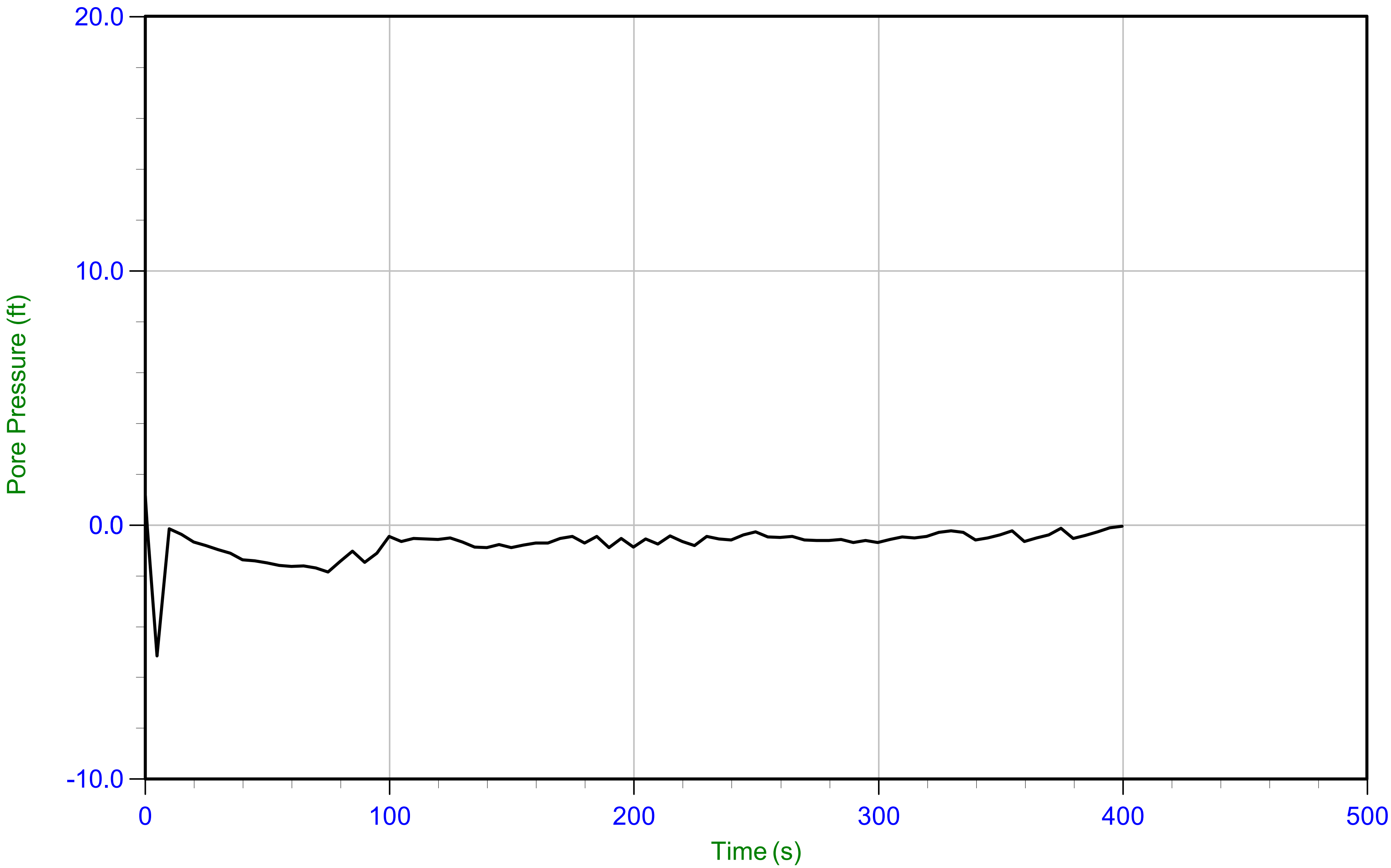
WT: 12.650 m / 41.502 ft
Ueq: 0.0 ft

ConeTec

A3GEO

Job No: 20-56-21274
Date: 08/24/2020 09:08
Site: 2550 Irving Street, SF

Sounding: CPT-02
Cone: 483:T1500F15U500 Area=15 cm²



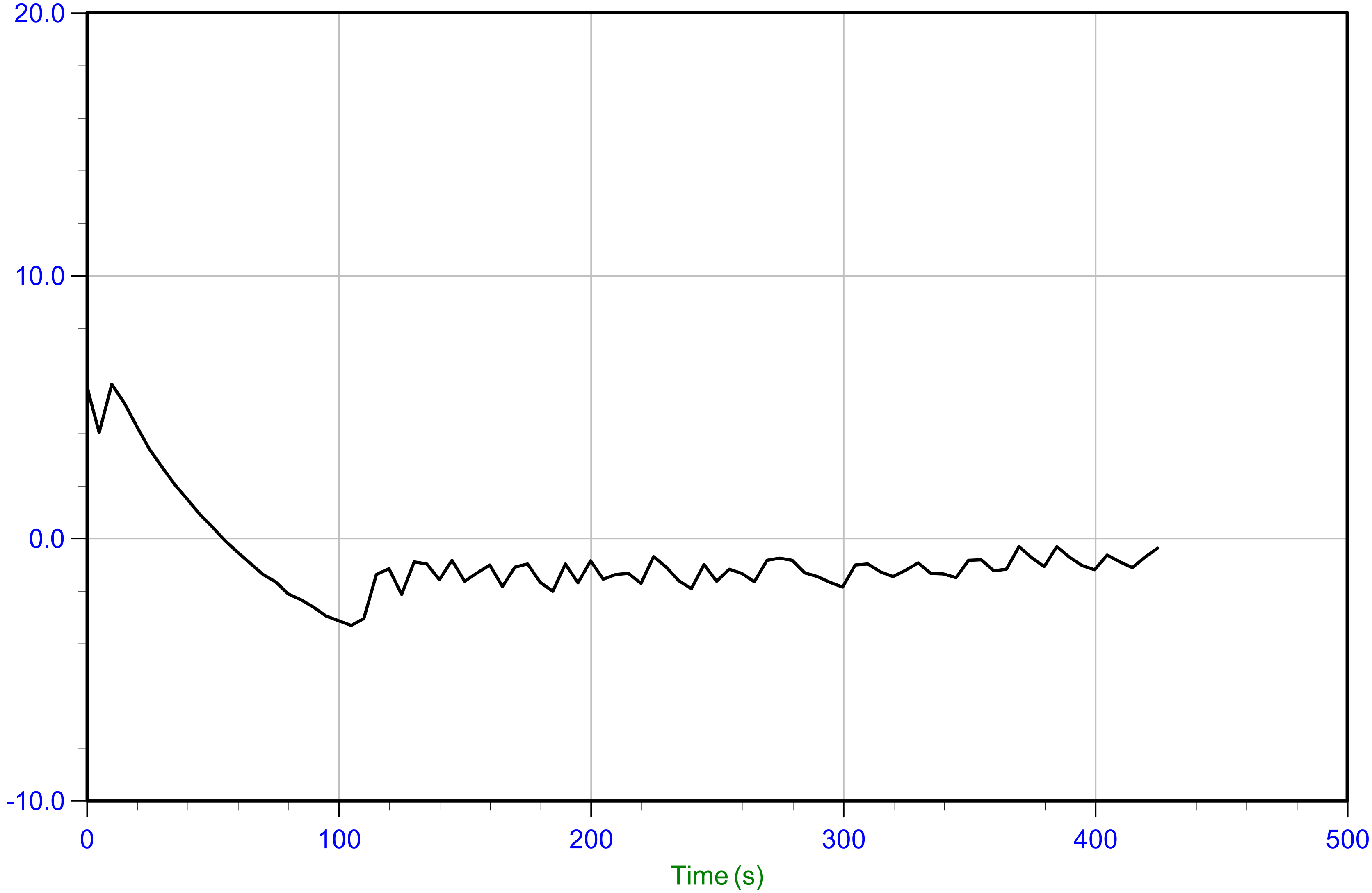

Trace Summary:

Filename: 20-56-21274_CP02.PPF
Depth: 13.775 m / 45.193 ft
Duration: 400.0 s

u Min: -5.1 ft
u Max: 1.2 ft
u Final: -0.0 ft

WT: 13.775 m / 45.193 ft
Ueq: 0.0 ft

| | | |
|---|---|---|
| A3GEO | Job No: 20-56-21274<br>Date: 08/24/2020 09:08<br>Site: 2550 Irving Street, SF | Sounding: CPT-02<br>Cone: 483:T1500F15U500 Area=15 cm² |




Trace Summary:

Filename: 20-56-21274_CP02.PPF
Depth: 17.975 m / 58.972 ft
Duration: 425.0 s

u Min: -3.3 ft
u Max: 5.9 ft
u Final: -0.4 ft

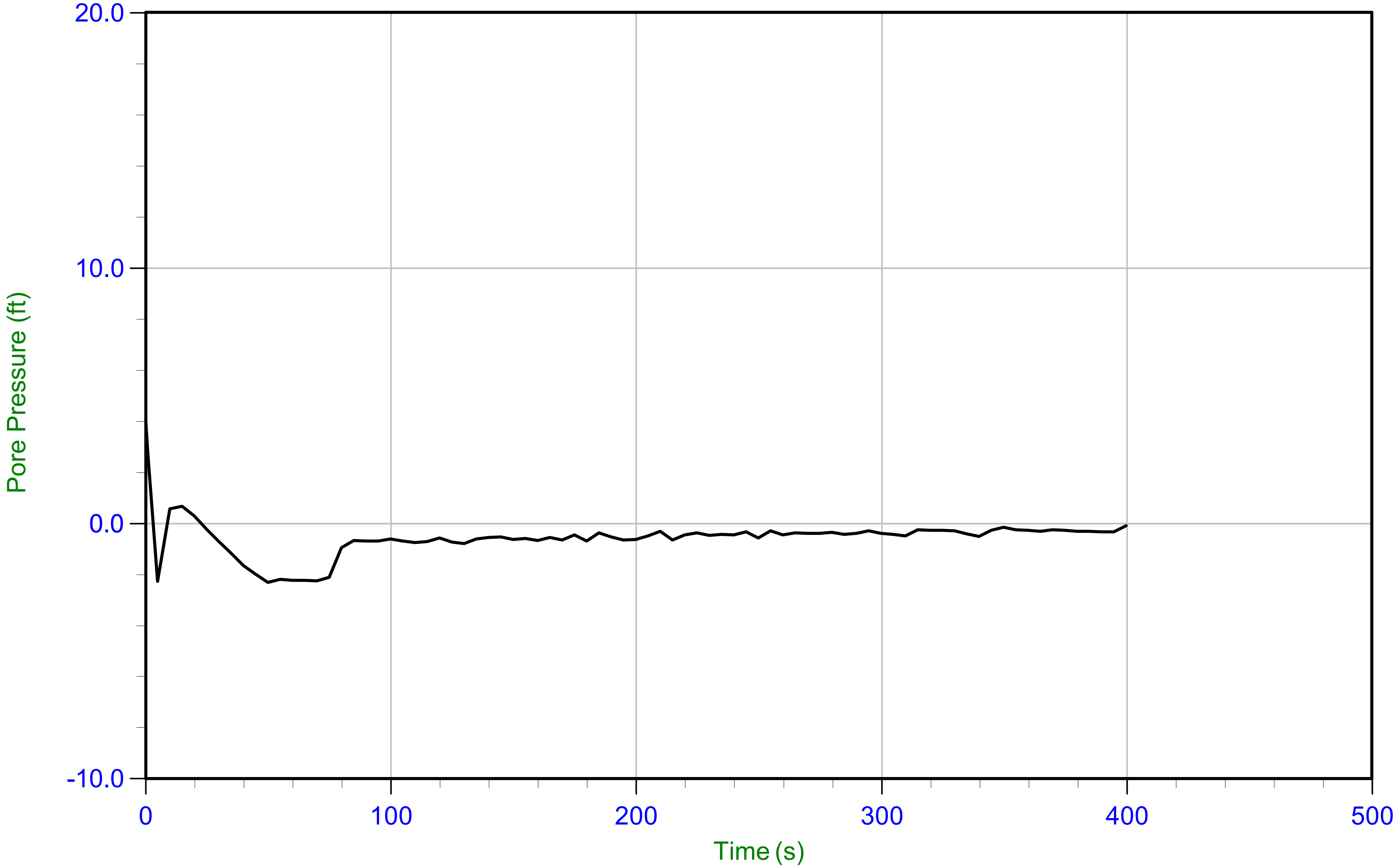
WT: 17.975 m / 58.972 ft
Ueq: 0.0 ft

ConeTec

A3GEO

Job No: 20-56-21274
Date: 08/24/2020 09:08
Site: 2550 Irving Street, SF

Sounding: CPT-02
Cone: 483:T1500F15U500 Area=15 cm²



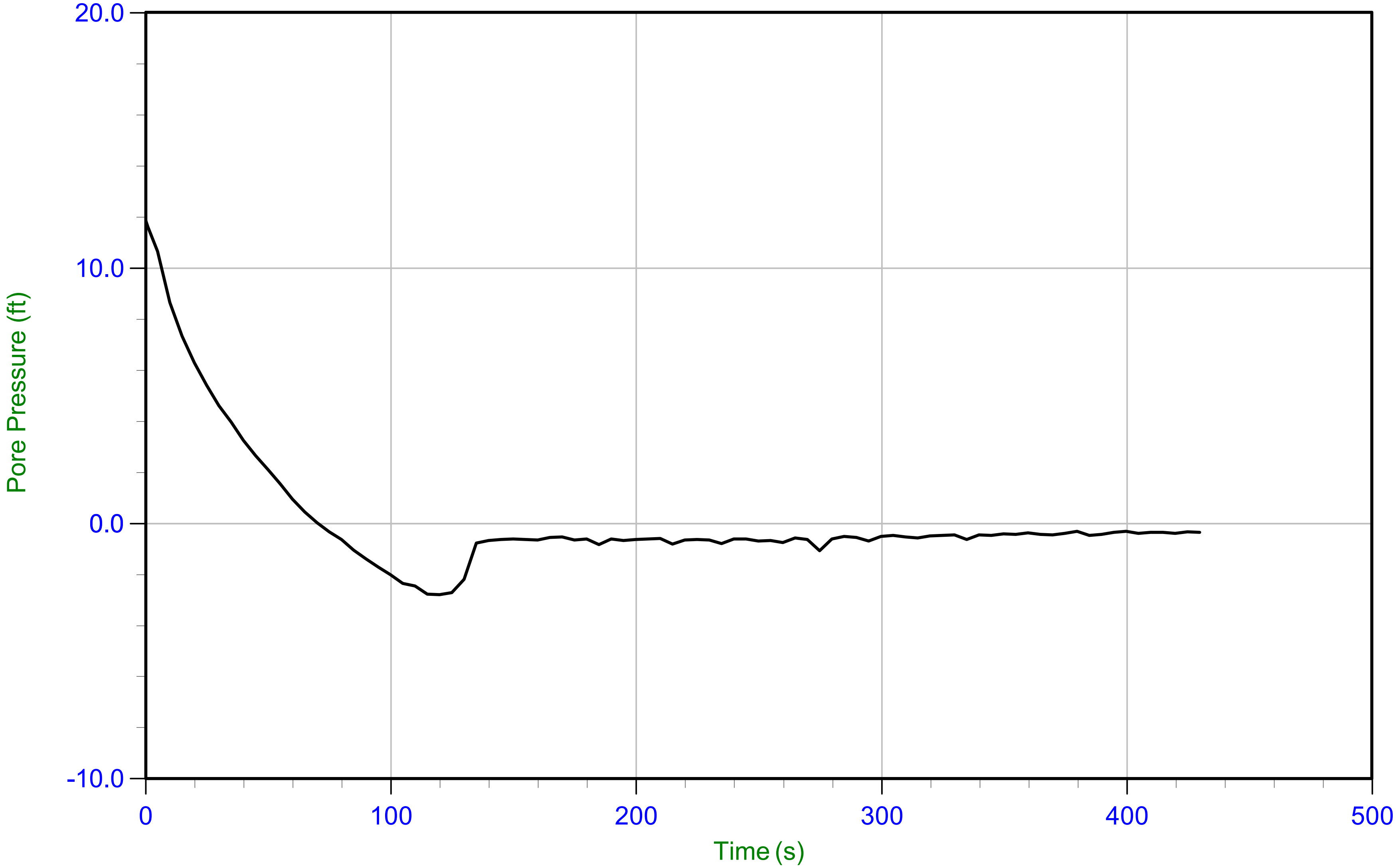

Trace Summary:

Filename: 20-56-21274_CP02.PPF
Depth: 19.275 m / 63.237 ft
Duration: 400.0 s

u Min: -2.3 ft
u Max: 4.0 ft
u Final: -0.1 ft

WT: 19.275 m / 63.237 ft
Ueq: 0.0 ft

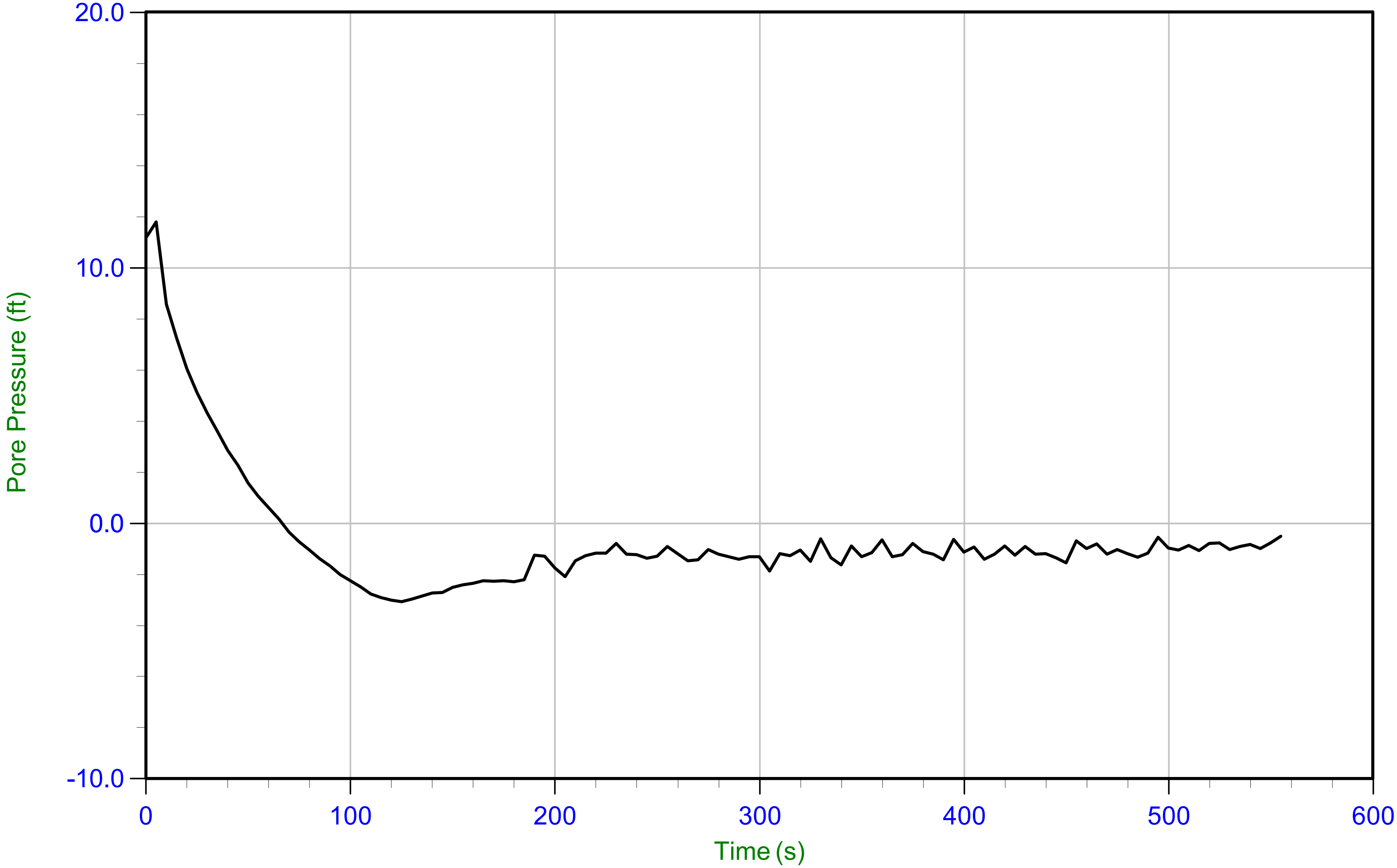
| | | |
|---|---|---|
| A3GEO | Job No: 20-56-21274<br>Date: 08/24/2020 11:36<br>Site: 2550 Irving Street, SF | Sounding: CPT-03<br>Cone: 483:T1500F15U500 Area=15 cm² |




Trace Summary:

| | | |
|---|---|---|
| Filename: 20-56-21274_CP03.PPF | u Min: -2.8 ft | WT: 12.400 m / 40.682 ft |
| Depth: 12.400 m / 40.682 ft | u Max: 11.8 ft | Ueq: 0.0 ft |
| Duration: 430.0 s | u Final: -0.4 ft | |

| | | |
|---|---|---|
| A3GEO | Job No: 20-56-21274<br>Date: 08/24/2020 11:36<br>Site: 2550 Irving Street, SF | Sounding: CPT-03<br>Cone: 483:T1500F15U500 Area=15 cm² |




Trace Summary:

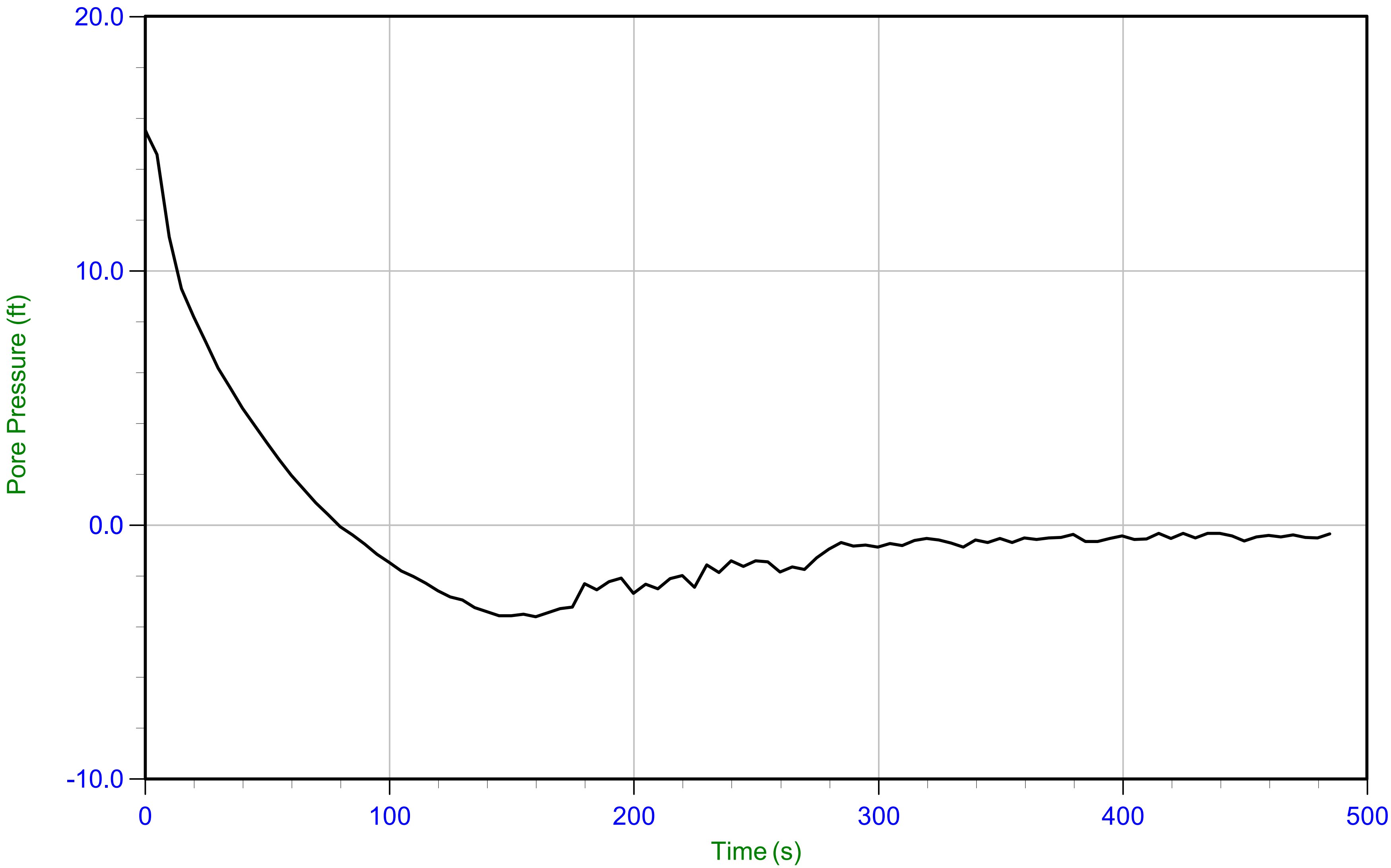
| | | |
|---|---|---|
| Filename: 20-56-21274_CP03.PPF | u Min: -3.1 ft | WT: 14.775 m / 48.474 ft |
| Depth: 14.775 m / 48.474 ft | u Max: 11.8 ft | Ueq: 0.0 ft |
| Duration: 555.0 s | u Final: -0.5 ft | |

CONETEC

A3GEO

Job No: 20-56-21274
Date: 08/24/2020 11:36
Site: 2550 Irving Street, SF

Sounding: CPT-03
Cone: 483:T1500F15U500 Area=15 cm²




Trace Summary:

Filename: 20-56-21274_CP03.PPF
Depth: 18.675 m / 61.269 ft
Duration: 485.0 s

u Min: -3.6 ft
u Max: 15.5 ft
u Final: -0.3 ft

WT: 18.675 m / 61.269 ft
Ueq: 0.0 ft

# APPENDIX C
# Seismic MASW Survey Report

# Geophysical Report

**Seismic MASW Survey**
**2550 Irving Street**
**San Francisco, California**

February 25, 2022
NORCAL JOB NO. NS225008

**Prepared for:**

A3GEO
821 Bancroft Way
Berkeley, California 94710

**Prepared by:**

NORCAL GEOPHYSICAL CONSULTANTS INC.
A Terracon COMPANY

NORCAL Geophysical Consultants, Inc. 321A Blodgett Street Cotati, California 94931
P (707) 796-7170 F (707) 796-7175 norcalgeophysical.com

February 25, 2022

A3GEO
821 Bancroft Way
Berkeley, California 94710

Subject: Seismic MASW Survey
2550 Irving Street
San Francisco, California

NORCAL Project No. NS225008

Attention: Mr. Dillon Braud

Dear Mr. Braud:

This report presents the findings of a seismic MASW survey performed by NORCAL Geophysical Consultants, Inc. for A3GEO, Inc. (A3GEO) at 2550 Irving Street in San Francisco, California. We understand that the results of this survey will be used to aid in assessing the Seismic Site Class. This will help to determine the design parameters for future site improvements. This work was authorized under an A3GEO Subcontractor Agreement dated January 27, 2022 for A3GEO Project No. 1146-4B. NORCAL Professional Geophysicists David T. Hagin (CA PGp No. 1033) and Charles Carter (CA PGp No. 1051) performed the survey on February 10, 2022.

The scope of NORCAL's services for this project consisted of using geophysical methods to characterize the subsurface. The accuracy of our findings is subject to specific site conditions and limitations inherent to the techniques used. We performed our services in a manner consistent with the standard of care ordinarily exercised by members of the profession currently employing similar methods. No warranty, with respect to the performance of services or products delivered under this agreement, expressed or implied, is made by NORCAL.

We appreciate having the opportunity to provide our services for this project. If you have any questions or require additional geophysical services, please do not hesitate to call on us.

Sincerely,
**NORCAL Geophysical Consultants, Inc.**

David T. Hagin
California Professional Geophysicist
PGp 1033

PROFESSIONAL GEOPHYSICIST
DAVID T HAGIN
No. 1033
STATE OF CALIFORNIA

2-25-2022

Donald J. Kirker

Donald J. Kirker, Reviewer
California Professional Geophysicist
PGp No. 997

PROFESSIONAL GEOPHYSICIST
DONALD KIRKER
No. 997
STATE OF CALIFORNIA

2-25-2022

# Geophysical Report

**Seismic MASW Survey**
**2550 Irving Street**
**San Francisco, California**

February 25, 2022

## 1.0 INTRODUCTION

The Seismic Multichannel Analysis of Surface Waves (MASW) survey measures the shear-wave velocities of the subsurface as a function of depth. The survey method is a sounding, producing one-dimensional (1-D) data that are presented in tabular and graphic form as a layered shear wave model. The location of an MASW sounding is considered to be the center of the geophone array. This survey consisted of a single MASW sounding, MASW-1. Descriptions of the MASW methodology, our data acquisition and analysis procedures, and the instrumentation we employed are provided in **Appendix A – MASW Survey**.

A map showing the site vicinity and the location of the seismic geophone array comprising the MASW-1 sounding is shown on **Plate 1 – Site Location Map**.

## 2.0 SITE CONDITIONS

The following description of site conditions is derived from our observations during the survey and a review of publicly available aerial photographs, geologic and topographic maps.

| Item | Description |
|---|---|
| **Site Information** | The site consists of a two-story structure on the eastern side of the lot and a parking lot on the western side. The approximate coordinates of the center of the site are: (37°45'47.9"N 122°29'06.6"W). |
| **Current Ground Cover** | The survey was located on a concrete sidewalk along the northern side of Irving Street, between 26th and 27th Avenues. |
| **Existing Topography** | Based on our Trimble Geo-7X GPS, Google Earth and site observations, the survey area is flat, with a surface elevation of about 201-ft (NAVD88). |
| **Site Geology** | Available geologic maps (USGS, 2003; CGS 2010) and discussions with A3GEO personnel indicate that the site geology consists of Quaternary sand deposits (dune sand). |

## 3.0 SCOPE OF WORK

Our scope of work includes acquiring MASW data for a single sounding denoted MASW-1, as shown on Plate 1. The sounding location was determined by A3GEO. Our scope of work also includes processing and interpreting the MASW data, as well as presenting our findings in a written report.

## 4.0 RESULTS

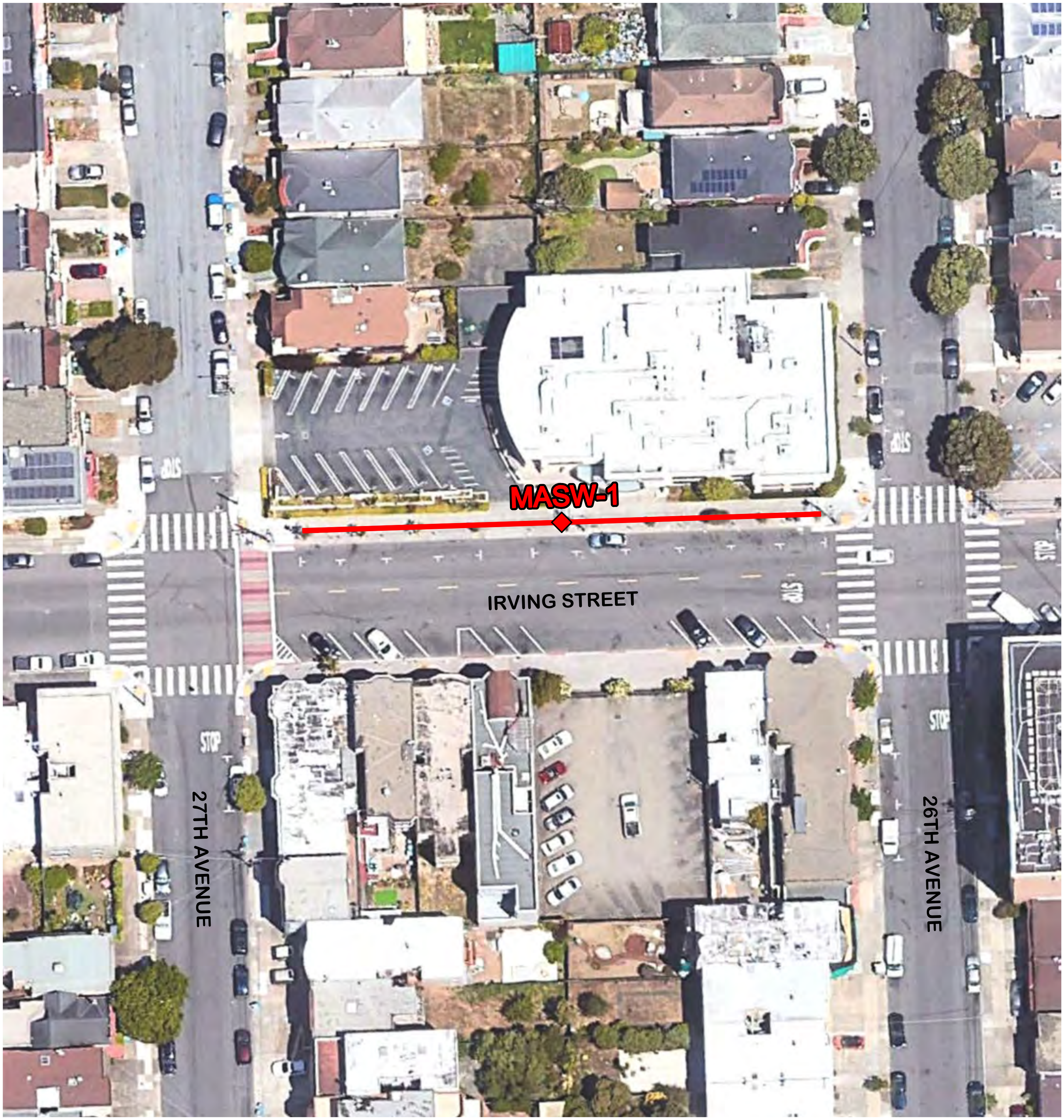
The results of the MASW survey are listed in Table A, below. The left column contains the depth ranges for each layer (feet below ground surface) and the right column comprises the associated shear (S-) wave values in feet per second (ft/sec). The results are also presented graphically by the step chart shown on **Plate 2 – MASW Sounding**.

**Table A :** *MASW-1 Seismic S-Wave Velocity vs Depth*

| DEPTH RANGE (FT) | S-WAVE VELOCITY (FT/SEC) |
|---|---|
| 0 - 2 | 560 |
| 2 - 6 | 520 |
| 6 - 9 | 580 |
| 9 - 14 | 750 |
| 14 - 20 | 810 |
| 20 - 28 | 830 |
| 28 - 37 | 920 |
| 37 - 49 | 1,030 |
| 49 - 63 | 1,050 |
| 63 - 100 | 1,190 |

The measured Vs values are relatively low, ranging from a low of 520 ft/sec to a maximum of 1,190 ft/sec. The values generally increase with increasing depth; however, a seismic velocity inversion (decreasing Vs with depth) is apparent at a depth of 2 feet.

The standard method of reporting MASW data is to consider the location of the 1D velocity vs. depth model as the center point of the MASW array. However, this does not mean that the measured velocity values represent materials solely beneath that location. In fact, the subsurface conditions underlying the entire length of the array, and for several tens of feet to either side, contribute to the measured velocity values.




| LEGEND | | | |
|---|---|---|---|
| ◆ | MASW SOUNDING LOCATION | ▬ | MASW SOUNDING ARRAY |

SCALE

0 60 120

(1 inch = 60 feet)

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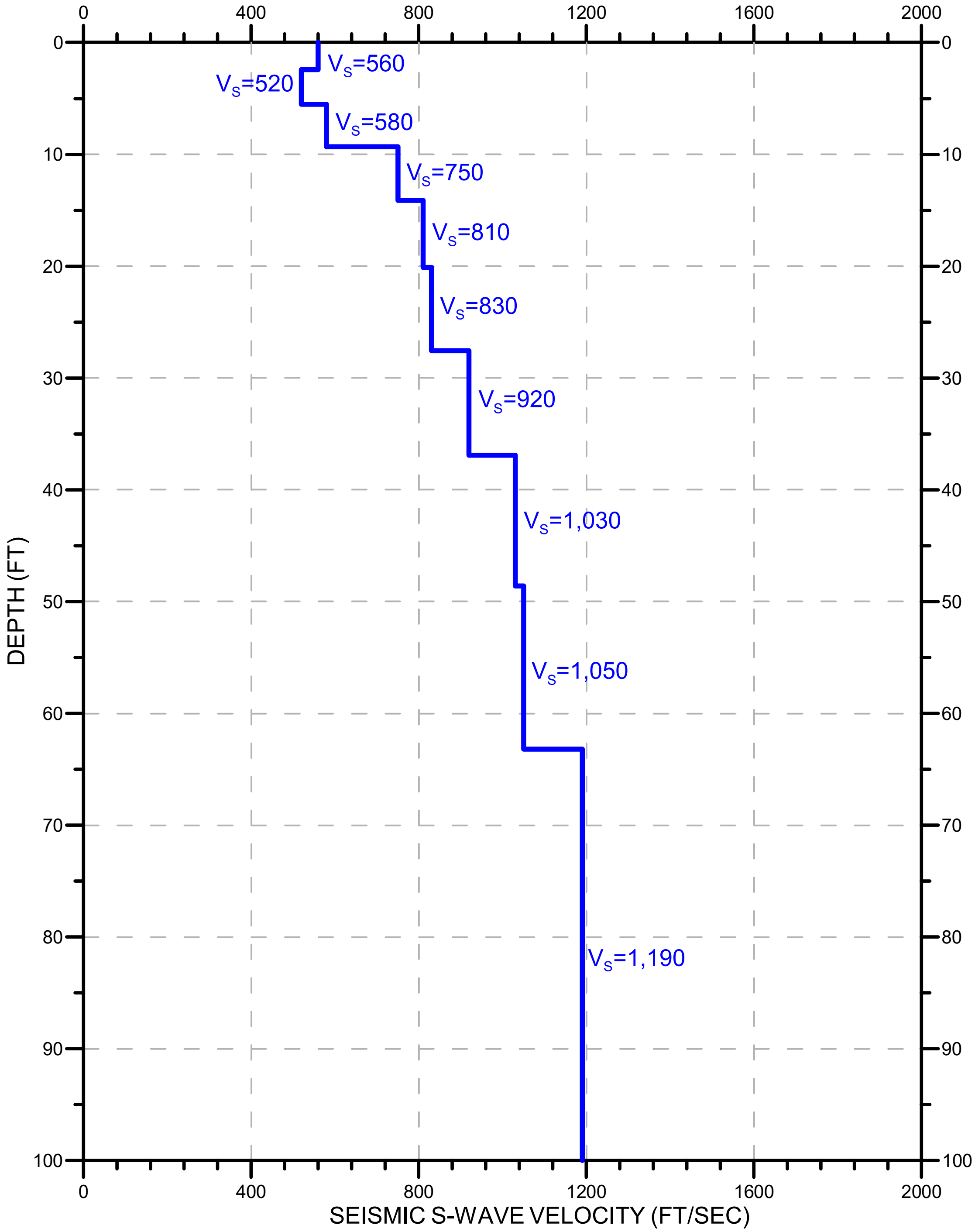
DAVID T HAGIN
No. 1033
STATE OF CALIFORNIA

# SITE LOCATION MAP
# MASW SURVEY
# 2550 IRVING STREET

| | | |
|---|---|---|
| LOCATION: SAN FRANCISCO, CALIFORNIA | | PLATE |
| CLIENT: A3GEO | | |
| JOB #. NS225008 | DATE: FEBRUARY 2022 | 1 |
| DRAWN BY: H.PHILSON | APPROVED BY: DTH | |
| David Hagin | 2/22/2022 | |

## MASW-1




| LEGEND | |
|---|---|
| ▬▬ | SEISMIC S-WAVE VELOCITY |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DAVID T HAGIN
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STATE OF CALIFORNIA

MASW SOUNDING
MASW-1
2550 IRVING STREET

| | | |
|---|---|---|
| LOCATION: SAN FRANCISCO, CALIFORNIA | | |
| CLIENT: A3GEO | | |
| JOB #: NS225008 | DATE: FEBRUARY 2022 | PLATE |
| DRAWN BY: H.PHILSON | APPROVED BY: DTH | 2 |
| David Hagin | 2/22/2022 | |

# APPENDIX A:
## MASW Sounding

# APPENDIX A:
## MASW Sounding

## 1.0 METHODOLOGY

When seismic energy is generated at or near the ground surface, both body and surface waves are produced. Body waves expand omni-directionally throughout the subsurface. They consist of both compressional (P) and shear (S) waves. Surface waves (e.g., Rayleigh, Love, etc.) radiate along the ground surface at velocities that are proportional to shear wave velocity (Vs). Rayleigh waves are characterized by retrograde elliptical particle motion, and travel at approximately 0.9 times the velocity of S-waves.

If a vertical impact source is used, approximately two-thirds of the seismic energy that is produced is in the form of ground roll. As a result, surface waves are typically the most prominent signal on multi-channel seismic records. In addition, surface waves have dispersion properties that body waves lack. That is, different wavelengths have different penetration depths and, therefore, propagate at different velocities. By analyzing the dispersion of surface waves, it is possible to obtain an S-wave versus depth velocity profile. Since s-wave velocity is directly proportional to shear modulus, this provides a direct indication in the variation of stiffness (or rigidity) of subsurface materials.

Surface waves can be recorded and analyzed using a method referred to as Multichannel Analysis of Surface Waves (MASW). This method is used to collect surface wave data using a fixed array of geophones and shot points. This is referred to as a sounding, and results in a one-dimensional (1-D) model depicting variation in S-wave velocity versus depth beneath the center of the array. However, the subsurface conditions underlying the entire length of the array, and for several tens of feet to either side, contribute to the measured velocity values. The method requires an energy source that is capable of producing ground roll and geophones that are capable of detecting low frequencies (<10 Hz) signals.

## 2.0 DATA ACQUISITION

We acquired a single MASW sounding, denoted MASW-1, as determined by A3GEO personnel and shown on Plate 1. The MASW sounding was acquired with four-shot points and 24-geophones distributed in a collinear array with a total length of 210-ft, as shown in Figure 1, below.

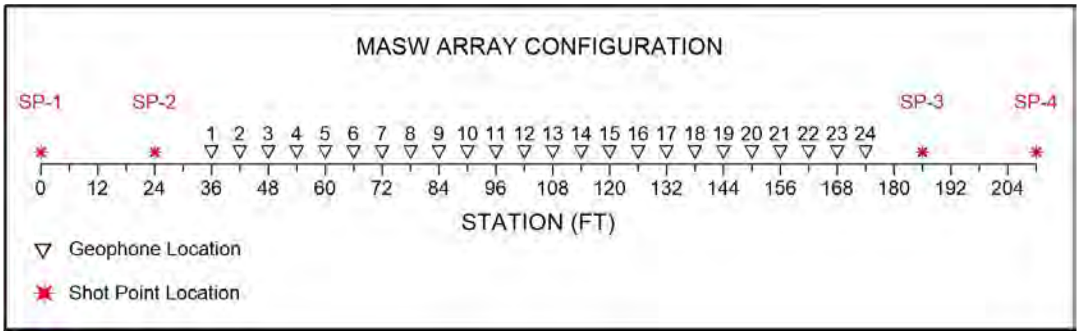


**Figure 1:** *MASW Array Configuration.*

Seismic energy was produced at each shot point using a 16-pound sledgehammer striking a metal plate on the ground surface. The resulting seismic waveforms were detected by Oyo ***Geospace*** geophones with a natural frequency of 4.5-Hz and recorded using a Geometrics ***Geode*** 24-channel distributed array engineering seismograph. The seismic waveforms were digitized, pre-processed and amplified by the Geode and transmitted via a ruggedized Ethernet cable to a field computer. The recorded data were archived for subsequent processing and displayed on the computers LCD screen in the form of seismograms for quality assurance purposes.

The position of the MASW array is shown on Plate 1 by the red line. The center point of the array, which is considered the sounding location, is represented by the red diamond.

## 3.0 DATA ANALYSIS

The seismic wave-traces (shot gathers) recorded at each shot point were analyzed using the computer program ***SURFSEIS*** developed by the Kansas Geological Survey (Version 5.0, 2016). This interactive program converts the data acquired from all four shot points in a given sounding into a dispersion curve representing phase velocity versus frequency. This curve is then inverted to produce a 1D model indicating S-wave velocity versus depth. The steps involved in this procedure are as follows:

1) The shot gathers are converted to KGS format.
2) Stations are assigned to the geophone and shot point locations.
3) The resulting records are viewed to determine their overall quality. If necessary, portions of the records are muted to remove interference from refractions, reflections and higher mode events.
4) For each formatted (and/or muted) record, the program produces what is referred to as an “overtone plot”. This is a colored cross-section indicating phase velocity versus frequency and amplitude. The vertical axis represents phase velocity (increasing upward); the horizontal axis represents frequency (increasing to the right); and signal amplitude is

indicated by various colors, with the hottest colors (orange to red to dark brown) representing the greatest signal to noise ratio. Typically, the strongest signals align in a curved pattern with a symmetry with the shape of a “hockey stick” where the blade is pointing upward at the lower end of the frequency spectrum (higher velocity at greater depth) and the handle projects to the right in the direction of increasing frequencies indicating lower velocities.

5) The overtone plots compiled from the four shot points are reviewed to determine their overall quality and the best among them (possibly all) are merged to form a single overtone. This enhances the overall signal to noise ratio of the survey and incorporates data from both ends of the spread (if feasible).
6) The resulting overtone plot is used as a guide in deriving a dispersion curve representing phase velocity versus frequency. This is done by fitting the curve along the center of the hockey stick where the signal to noise ratio is highest.
7) The resulting dispersion curve is inverted through an iterative process to compute a 1D model representing S-wave velocity versus depth.

The velocities in each depth range for MASW-1 are tabulated in Table A in the main body of the report.

# APPENDIX D
# Existing Geo-Environmental Boring Logs

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San Francisco, CA 94110
Telephone: 415.391.2510

# BORING NUMBER B-1



**CLIENT** SFPolice Credit Union
**PROJECT NAME** Irving-SF Sub
**PROJECT NUMBER** 19061.23
**PROJECT LOCATION** 2500-2550 Irving, San Francisco
**DATE STARTED** 5/21/19 **COMPLETED** 5/21/19
**GROUND ELEVATION** **HOLE SIZE** 2"
**DRILLING CONTRACTOR** ECA
**GROUND WATER LEVELS:**
**DRILLING METHOD** Direct Push Technology
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Leonard Niles, PG, CHG
**AT END OF DRILLING** ---
**NOTES** Borehole grouted with neat Portland cement
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | | MATERIAL DESCRIPTION | Environmental Data |
|---|---|---|---|---|---|---|---|
| 0 | | | | | 0.2 | Approximately 2" asphalt | |
| | UD B-1@1-1.5 | 100 | SW | | 2.5 | (SW) Tan to brown, fine to coarse grained sand, well graded, no plasticity | |
| 5 | UD B-1@4.5-5 | 100 | SP | | 6.0 | (SP) Brown, fine-grained sand, no plasticity | PID = 0 |
| 10 | | | SP | | 10.0 | (SP) Same as above interval | PID = 0 |
| | UD B-1@9.5-10 | 100 | | | | Bottom of borehole at 10.0 feet. | PID = 0 |

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# BORING NUMBER B-2



**CLIENT** SFPolice Credit Union
**PROJECT NAME** Irving-SF Sub
**PROJECT NUMBER** 19061.23
**PROJECT LOCATION** 2500-2550 Irving, San Francisco
**DATE STARTED** 5/21/19 **COMPLETED** 5/21/19
**GROUND ELEVATION** **HOLE SIZE** 2"
**DRILLING CONTRACTOR** ECA
**GROUND WATER LEVELS:**
**DRILLING METHOD** Direct Push Technology
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Leonard Niles, PG, CHG
**AT END OF DRILLING** ---
**NOTES** Borehole grouted with neat Portland cement
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | Environmental Data |
|---|---|---|---|---|---|---|
| 0 | | | | 0.2 | Approximately 2" asphalt | |
| | | | SW | 2.5 | (SW) Brown, coarse grained sand, well graded, no plasticity | |
| 5 | UD B-2@1-1.5 | 100 | SP | 6.0 | (SP) Brown to light brown, very fine grained sand, no plasticity | PID = 0 |
| 10 | UD B-2@4.5-5 | 100 | SP | 10.0 | (SP) Same as above interval | PID = 0 |
| | UD B-2@9.5-10 | 100 | | | Bottom of borehole at 10.0 feet. | PID = 0 |

GENERAL BH / TP / WELL - GINT STD US.GDT - 6/27/19 10:32 - C:\USERS\PUBLIC\DOCUMENTS\BENTLEY\GINT\PROJECTS\19061.23 2550 IRVING SF.GPJ

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San Francisco, CA 94110
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# BORING NUMBER B-3



**CLIENT** SFPolice Credit Union
**PROJECT NAME** Irving-SF Sub
**PROJECT NUMBER** 19061.23
**PROJECT LOCATION** 2500-2550 Irving, San Francisco
**DATE STARTED** 5/21/19 **COMPLETED** 5/21/19
**GROUND ELEVATION** **HOLE SIZE** 2"
**DRILLING CONTRACTOR** ECA
**GROUND WATER LEVELS:**
**DRILLING METHOD** Direct Push Technology
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Leonard Niles, PG, CHG
**AT END OF DRILLING** ---
**NOTES** Borehole grouted with neat Portland cement
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | | MATERIAL DESCRIPTION | Environmental Data |
|---|---|---|---|---|---|---|---|
| 0 | | | | | 0.2 | Approximately 2" asphalt | |
| | | | SW | | 2.5 | (SW) Brown, coarse to fine grained sand with minor gravel, no plasticity | |
| | UD B-3@1-1.5 | 100 | SP | | 6.0 | (SP) Brown to tan, fine grained sand, low plasticity | PID = 0 |
| 5 | | | | | | | |
| | UD B-3@4.5-5 | 100 | SP | | 10.0 | (SP) Same as above interval | PID = 0 |
| 10 | UD B-3@9.5-10 | 100 | | | | Bottom of borehole at 10.0 feet. | PID = 0 |

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# BORING NUMBER B-4



**CLIENT** SFPolice Credit Union
**PROJECT NAME** Irving-SF Sub
**PROJECT NUMBER** 19061.23
**PROJECT LOCATION** 2500-2550 Irving, San Francisco
**DATE STARTED** 5/21/19 **COMPLETED** 5/21/19
**GROUND ELEVATION** **HOLE SIZE** 2"
**DRILLING CONTRACTOR** ECA
**GROUND WATER LEVELS:**
**DRILLING METHOD** Direct Push Technology
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Leonard Niles, PG, CHG
**AT END OF DRILLING** ---
**NOTES** Borehole grouted with neat Portland cement
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | | MATERIAL DESCRIPTION | Environmental Data |
|---|---|---|---|---|---|---|---|
| 0 | | | | | 0.2 | Approximately 2" asphalt | |
| | | | SP | | 2.5 | (SP) Brown, fine grained, poorly graded sand, no plasticity | |
| 5 | UD B-4@1-1.5 | 100 | SP | | 6.0 | (SP) Same as above interval | PID = 0 |
| 10 | UD B-4@4.5-5 | 100 | SP | | 10.0 | (SP) Same as above interval | PID = 0 |
| | UD B-4@9.5-10 | | | | | Bottom of borehole at 10.0 feet. | PID = 0 |

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# BORING NUMBER B-5



**CLIENT** SFPolice Credit Union
**PROJECT NAME** Irving-SF Sub
**PROJECT NUMBER** 19061.23
**PROJECT LOCATION** 2500-2550 Irving, San Francisco
**DATE STARTED** 5/21/19 **COMPLETED** 5/21/19
**GROUND ELEVATION** **HOLE SIZE** 2"
**DRILLING CONTRACTOR** ECA
**GROUND WATER LEVELS:**
**DRILLING METHOD** Direct Push Technology
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Leonard Niles, PG, CHG
**AT END OF DRILLING** ---
**NOTES** Borehole grouted with neat Portland cement
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | Environmental Data |
|---|---|---|---|---|---|---|
| 0 | | | | 0.2 | Approximately 2" asphalt | |
| | | | SW | 2.5 | (SW) Brown to dark brown/black, coarse grained sand, well graded, low plasticity | |
| 5 | UD B-5@1-1.5 | 100 | SP | 6.0 | (SP) Same as above interval | PID = 0 |
| 10 | UD B-5@4.5-5 | 100 | SP | 10.0 | (SP) Same as above interval | PID = 0 |
| | UD B-5@9.5-10 | 100 | | | Bottom of borehole at 10.0 feet. | PID = 0 |

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# BORING NUMBER B-9



**CLIENT** SFPCU
**PROJECT NAME** Irving II
**PROJECT NUMBER** 19086.23.1
**PROJECT LOCATION** 2550 Irving Street, San Francisco, CA
**DATE STARTED** 7/17/19 **COMPLETED** 7/17/19
**GROUND ELEVATION** **HOLE SIZE** 2"
**DRILLING CONTRACTOR** ECA
**GROUND WATER LEVELS:**
**DRILLING METHOD** Direct Push Technology
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Leonard Niles, PG, CHG
**AT END OF DRILLING** ---
**NOTES** Borehole grouted with neat cement
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | Environmental Data |
|---|---|---|---|---|---|---|
| 0 – 0.1 | | | | | Ashpalt 1.5" thick | |
| 0.1 – 4.0 | MC B-9@1-1.5 | 100 | SP | | (SP) Sand, brown, fine-grained, no plasticity, moist, no odor or staining | PID = 0 |
| 4.0 – 8.0 | | | SP | | (SP) Same as above | |
| 8.0 – 12.0 | MC B-9@9.5-10 | 100 | SP | | (SP) Same as above, color becoming lighter brown | PID = 0 |
| 12.0 – 16.0 | | | SP | | (SP) Same as above, sand is becoming slightly more coarse | PID = 0<br>PID = 0 |
| 16.0 – 20.0 | | | SP | | (SP) Same as above, color changing to a tan brown | PID = 0 |
| 20.0 – 24.0 | MC B-9@19.5-20 | 100 | SP | | (SP) Sand, brown to light brown, fine-grained, moist, no plasticity, no odor or staining | PID = 0 |
| 24.0 – 28.0 | | | SP | | (SP) Same as above, sand is becoming medium-grained | |
| 28.0 – 32.0 | MC B-9@29.5-30 | 100 | SP | | (SP) Same as above | PID = 0 |
| 32.0 – 35 | | | SP | | (SP) Same as above, no groundwater is noticed, the core itself has no sign of moisture | |

*(Continued Next Page)*

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# BORING NUMBER B-9



**CLIENT** SFPCU
**PROJECT NAME** Irving II
**PROJECT NUMBER** 19086.23.1
**PROJECT LOCATION** 2550 Irving Street, San Francisco, CA

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | Environmental Data |
|---|---|---|---|---|---|---|
| 35 | | | SP | 36.0 | (SP) Same as above, no groundwater is noticed, the core itself has no sign of moisture *(continued)* | PID = 0 |
| | | | SP | 40.0 | (SP) Same as above, no groundwater encountered | PID = 0 |
| 40 | MC B-9@39.5-40 | 100 | SP | 42.0 | (SP) Same as above | |
| 45, 50 | | | SP | 52.0 | (SP) Sand, brown/light brown, fine-grained, moist, no plasticity, no odor or staining, no saturation and no sign of moisture on core. Refusal at 52' bgs. Casing set at 52' bgs, screen from 32'-52', no groundwater encountered. Casing removed and borehole grouted. | PID = 0 |

Bottom of borehole at 52.0 feet.

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# BORING NUMBER B-10



**CLIENT** SFPCU
**PROJECT NAME** Irving II
**PROJECT NUMBER** 19086.23.1
**PROJECT LOCATION** 2550 Irving Street, San Francisco, CA
**DATE STARTED** 7/18/19 **COMPLETED** 7/18/19
**GROUND ELEVATION** **HOLE SIZE** 2"
**DRILLING CONTRACTOR** ECA
**GROUND WATER LEVELS:**
**DRILLING METHOD** Direct Push Technology
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Leonard Niles, PG, CHG
**AT END OF DRILLING** ---
**NOTES** Borehole grouted with neat cement
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | Environmental Data |
|---|---|---|---|---|---|---|
| 0 | MC B-10@1-1.5 | 100 | SP | | (SP) (no asphalt, boring in landscaped area) Sand, light brown, fine-grained, dry, no plasticity, no odor or staining, minor tree/plant matter from 0-2' bgs | PID = 0 |
| 4.0 | | | | | | |
| 5 | MC B-10@4.5-5 | 100 | SP | | (SP) Sand, light brown to brown, fine-grained, dry, no plasticity, no odor or staining | PID = 0.1 |
| 8.0 | | | SP | | (SP) Same as above, sand is still very dry | |
| 10 | MC B-10@9.5-10 | 100 | | | | PID = 0 |
| 12.0 | | | SP | | (SP) Same as above | |
| 15 | | | | | | |
| 16.0 | | | SP | | (SP) Sand, brown/tan, fine-grained, moist, no plasticity, no odor or staining | |
| 20 | MC B-10@19.5-20 | 100 | | | | PID = 0 |
| 20.0 | | | SP | | (SP) Same as above | |
| 24.0 | | | SP | | (SP) Sand, brown, fine-grained, no plasticity, moist, no odor or staining | PID = 0 |
| 25 | | | | | | |
| 28.0 | | | SP | | (SP) Same as above, sand is light brown to brown with no sign of groundwater | |
| 30 | MC B-10@29.5-30 | 100 | | | | PID = 0 |
| 32.0 | | | SP | | (SP) Same as above | |
| 35 | | | | | | |

*(Continued Next Page)*

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# BORING NUMBER B-10



**CLIENT** SFPCU

**PROJECT NAME** Irving II

**PROJECT NUMBER** 19086.23.1

**PROJECT LOCATION** 2550 Irving Street, San Francisco, CA

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | | MATERIAL DESCRIPTION | Environmental Data |
|---|---|---|---|---|---|---|---|
| 35 | | | SP | | | (SP) Same as above *(continued)* | |
| | | | SP | | 36.0 | (SP) Sand, brown, fine-grained, moist, no plasticity, no odor or staining, no apparent signs of moisture on the core. Installed casing, screen from 20'-40' bgs, casing in place for ~1.5 hours, no sign of groundwater. Removed casing and grouted borehole. | |
| 40 | MC B-10@39.5-40 | 100 | | | 40.0 | | PID = 0.1 |

Bottom of borehole at 40.0 feet.

# BORING NUMBER B-11



AllWest Environmental
2141 Mission Street, Suite 100
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Telephone: 415.391.2510

**CLIENT** The Police Credit Union
**PROJECT NAME** Irving GW
**PROJECT NUMBER** 19126.23
**PROJECT LOCATION** 2550 Irving St., San Francisco, CA
**DATE STARTED** 9/26/19 **COMPLETED** 9/27/19
**GROUND ELEVATION** **HOLE SIZE** 10"
**DRILLING CONTRACTOR** Woodward
**GROUND WATER LEVELS:**
**DRILLING METHOD** Hollow Stem Auger
**AT TIME OF DRILLING** 80.00 ft
**LOGGED BY** Sam Calloway **CHECKED BY** Leonard Niles, PG, CHG
**AT END OF DRILLING** ---
**NOTES** GW at 80', sample B-11 (GW) collected. No soil samples; logged from cuttings.
**AFTER DRILLING** 78.86 ft

| DEPTH (ft) | SAMPLE TYPE NUMBER | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | Environmental Data |
|---|---|---|---|---|---|
| 0 | | | 0.5 | 6" concrete | |
| | | SW | 5.0 | (SW) Brown, medium to coarse-grained sand, no odor or plasticity, sand is well graded | PID = 0 |
| 5 | | SP | 10.0 | (SP) Tan to brown, fine-grained sand, poorly graded, no odor or plasticity | |
| 10 | | SP | 15.0 | (SP) Same as above | |
| 15 | | SP | 21.0 | (SP) Same as above. No sign of GW | PID = 0<br>PID = 0 |
| 20 | | SW | 25.0 | (SW) Tannish brown, medium to fine-grained, well graded sand, no odor or plasticity | PID = 0 |
| 25 | | SW | 30.0 | (SW) Same as above | |
| 30 | | SW | 35.0 | (SW) Same as above | PID = 0 |
| 35 | | | | | |

*(Continued Next Page)*

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# BORING NUMBER B-11



**CLIENT** The Police Credit Union
**PROJECT NAME** Irving GW
**PROJECT NUMBER** 19126.23
**PROJECT LOCATION** 2550 Irving St., San Francisco, CA

| DEPTH (ft) | SAMPLE TYPE NUMBER | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | Environmental Data |
|---|---|---|---|---|---|
| 35 | | SW | | (SW) Same as above | PID = 0 |
| 40 | | | | | PID = 0 |
| 42.0 | | SP | | (SP) Brownish tan, fine-grained poorly graded sand, no odor or plasticity | |
| 45 | | | | | PID = 0.2 |
| 50.0 | | SP | | (SP) Same as above. No sign of GW | |
| 55 | | | | | PID = 0 |
| 60.0 | | SP | | (SP) Same as above | |
| 65 | | | | | PID = 0 |
| 70.0 | | SP | | (SP) Same as above. At 80' bgs GW was reached. Groundwater in boring rose to 78.86' bgs. | |
| 75 | | | | | |

*(Continued Next Page)*

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# BORING NUMBER B-11



**CLIENT** The Police Credit Union | **PROJECT NAME** Irving GW

**PROJECT NUMBER** 19126.23 | **PROJECT LOCATION** 2550 Irving St., San Francisco, CA

| DEPTH (ft) | SAMPLE TYPE NUMBER | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | Environmental Data |
|---|---|---|---|---|---|
| 75 | | SP | | (SP) Same as above. At 80' bgs GW was reached. Groundwater in boring rose to 78.86' bgs. *(continued)* | |
| 80 | | | | 80.0 | |

Bottom of borehole at 80.0 feet.

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# BORING NUMBER B-12



**CLIENT** The Police Credit Union
**PROJECT NAME** Irving GW
**PROJECT NUMBER** 19126.23
**PROJECT LOCATION** 2550 Irving St., San Francisco, CA
**DATE STARTED** 9/26/19 **COMPLETED** 9/27/19
**GROUND ELEVATION** **HOLE SIZE** 8"
**DRILLING CONTRACTOR** Woodward
**GROUND WATER LEVELS:**
**DRILLING METHOD** Hollow Stem Auger
**AT TIME OF DRILLING** 90.00 ft
**LOGGED BY** Sam Calloway **CHECKED BY** Leonard Niles, PG, CHG
**AT END OF DRILLING** ---
**NOTES** Static GW level 77.30
**AFTER DRILLING** 77.30 ft

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | Environmental Data |
|---|---|---|---|---|---|---|
| 0 | | | | 0.5 | 3" asphalt | |
| | | | | 1.0 | 9" concrete | |
| | | | SW | | (SW) Brown, medium to fine-grained sand, well graded, no odor or plasticity | |
| 5 | SS B-12@4.5-5 | 100 | SP | 5.0 | (SP) Brown, fine-grained sand, poorly graded, no odor or staining, no plasticity | PID = 0 |
| 10 | SS B-12@9.5-10 | 100 | SP | 10.0 | (SP) Same as above | PID = 0.2 |
| 15 | SS B-12@14.5-15 | 100 | SP | 15.0 | (SP) Same as above | PID = 0 |
| 20 | SS B-12@19.5-20 | 100 | SP | 20.0 | (SP) Brown, fine-grained sand, well sorted (poorly graded), no odor or staining, no plasticity | PID = 0 |
| 25 | SS B-12@24.5-25 | | SP | 25.0 | (SP) Same as above | PID = 0 |
| 30 | | | SP | 30.0 | (SP) Same as above. Color becoming slightly lighter, but still consistant with previous lithology. No sign of groundwater | PID = 0 |
| 35 | | | | | | |



*(Continued Next Page)*

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# BORING NUMBER B-12



**CLIENT** The Police Credit Union

**PROJECT NAME** Irving GW

**PROJECT NUMBER** 19126.23

**PROJECT LOCATION** 2550 Irving St., San Francisco, CA

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | | MATERIAL DESCRIPTION | Environmental Data |
|---|---|---|---|---|---|---|---|
| 35 | | | SP | | | (SP) Same as above. Color becoming slightly lighter, but still consistant with previous lithology. No sign of groundwater *(continued)* | PID = 0 |
| 40 | | | | | | | PID = 0.1 |
| | | | SP | | 42.0 | (SP) Tannish brown, fine-grained sand, poorly graded, no odor or staining, no plasticity | |
| 45 | | | | | | | PID = 0.2 |
| 50 | | | SP | | 50.0 | (SP) Same as above | |
| 55 | | | | | | | PID = 0 |
| 60 | | | SW | | 60.0 | (SW) Brown to light brown, medium to mostly fine-grained sand, well graded, no odor or staining, no plasticity | |
| 65 | | | | | | | PID = 0 |
| 70 | | | SW | | 70.0 | (SW) Same as above. Checked for groundwater at 75' bgs, no sign, continuing to drill deeper | |
| 75 | | | | | | | |

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# BORING NUMBER B-12



**CLIENT** The Police Credit Union
**PROJECT NAME** Irving GW
**PROJECT NUMBER** 19126.23
**PROJECT LOCATION** 2550 Irving St., San Francisco, CA

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | Environmental Data |
|---|---|---|---|---|---|---|
| 75 | | | SW | | (SW) Same as above. Checked for groundwater at 75' bgs, no sign, continuing to drill deeper *(continued)* | PID = 0 |
| 80 | | | SP | 80.0 | (SP) Brown to light brown, fine-grained sand, poorly sorted, no odor or staining, no plasticity. Encountered groundwater at 90' bgs, static GW measured at 77.30' bgs. | |
| 85 | | | | | | |
| 90 | | | | 90.0 | | |

Bottom of borehole at 90.0 feet.

# BORING NUMBER SVP-3



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**CLIENT** Police Credit Union
**PROJECT NAME** PCU Subsurface
**PROJECT NUMBER** 202006.23
**PROJECT LOCATION** 2525 Irving Street, San Francisco, CA
**DATE STARTED** 5/28/20 **COMPLETED** 5/28/20
**GROUND ELEVATION** **HOLE SIZE** 2" inches
**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.)
**GROUND WATER LEVELS:**
**DRILLING METHOD** DPT (direct push technology), 1.6" ID MacroCore
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles
**AT END OF DRILLING** ---
**NOTES** Temp SVP @ 15' bgs, removed, grouted boring w/ cement
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0 | | | | | 0.7 Asphalt ~8" thick | |
| | | | SP | | (SP) Brown fine-grained sand, no odor or staining; asphalt debris from 0-1.5' (fill) | PID = 0.1 |
| 5 | MC SVP-3@4.5-5 | 100 | SP | | 4.0 (SP) Same as above | PID = 0.1 |
| 10 | MC SVP-3@9.5-10 | 100 | SP | | 8.0 (SP) Same as above | PID = 0.3 |
| 15 | MC SVP-3@14.5-15 | 100 | SP | | 12.0 (SP) Same as above | PID = 0.3 |
| | | | | | 15.5 | |

Bottom of borehole at 15.5 feet.

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# BORING NUMBER SVP-4



**CLIENT** Police Credit Union
**PROJECT NAME** PCU Subsurface
**PROJECT NUMBER** 202006.23
**PROJECT LOCATION** 2511 Irving Street, San Francisco, CA
**DATE STARTED** 5/28/20 **COMPLETED** 5/28/20
**GROUND ELEVATION** **HOLE SIZE** 2" inches
**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.)
**GROUND WATER LEVELS:**
**DRILLING METHOD** DPT (direct push technology), 1.6" ID MacroCore
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles
**AT END OF DRILLING** ---
**NOTES** Temp SVP @ 15' bgs, removed, grouted boring w/ cement
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0 | | | | | 0.7 Asphalt ~8" thick | |
| | | | SP | | (SP) Brown, fine-grained sand, no odor or staining, very minor asphalt throughout. Asphalt debis with some sand 0-1.5 | PID = 0 |
| | | | | | 4.0 | |
| 5 | MC SVP-4@4.5-5 | 100 | SP | | (SP) Brown fine-grained sand, no asphalt present, no odor or staining | PID = 0 |
| | | | | | 8.0 | |
| 10 | MC SVP-4@9.5-10 | 100 | SP | | (SP) Same as above | PID = 0.1 |
| | | | | | 12.0 | |
| 15 | MC SVP-4@14.5-15 | 100 | SP | | (SP) Same as above | PID = 0.1 |
| | | | | | 15.5 | |

Bottom of borehole at 15.5 feet.

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# BORING NUMBER SVP-5



**CLIENT** Police Credit Union
**PROJECT NAME** PCU Subsurface
**PROJECT NUMBER** 202006.23
**PROJECT LOCATION** 2550 Irving Street, San Francisco, CA
**DATE STARTED** 5/28/20 **COMPLETED** 5/28/20
**GROUND ELEVATION** **HOLE SIZE** 2" inches
**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.)
**GROUND WATER LEVELS:**
**DRILLING METHOD** DPT (direct push technology), 1.6" ID MacroCore
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles
**AT END OF DRILLING** ---
**NOTES** Temp SVP @ 15' bgs, removed, grouted boring w/ cement
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0 | | | | | 0.7 Asphalt ~8" thick | |
| | | | SP | | (SP) Brown fine-grained sand, no odor or staining. Asphalt debris 0.5-1' bgs (fill) | PID = 0 |
| 5 | MC SVP-5@4.5-5 | 100 | SP | | 4.0 (SP) Brown fine-grained sand, no odor or staining | PID = 0 |
| 10 | MC SVP-5@9.5-10 | 100 | SP | | 8.0 (SP) Same as above | PID = 0 |
| 15 | MC SVP-5@14.5-15 | 100 | SP | | 12.0 (SP) Same as above | PID = 0 |
| | | | | | 15.5 | |

Bottom of borehole at 15.5 feet.

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# BORING NUMBER SVP-6



**CLIENT** Police Credit Union
**PROJECT NAME** PCU Subsurface
**PROJECT NUMBER** 202006.23
**PROJECT LOCATION** 2550 Irving Street, San Francisco, CA
**DATE STARTED** 5/28/20 **COMPLETED** 5/28/20
**GROUND ELEVATION** **HOLE SIZE** 2" inches
**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.)
**GROUND WATER LEVELS:**
**DRILLING METHOD** DPT (direct push technology), 1.6" ID MacroCore
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles
**AT END OF DRILLING** ---
**NOTES** Temp SVP @ 15' bgs, removed, grouted boring w/ cement
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0 | | | | | 0.7 Asphalt ~8" thick | |
| | | | SP | | (SP) Brown fine-grained sand, no odor or staining, rock fragments present. Asphalt debris with some sand 0.5-1 (fill) | PID = 0.1 |
| 5 | MC SVP-6@4.5-5 | 100 | SP | | 4.0 (SP) Brown fine-grained sand with very minor rock fragments, does not appear to be any asphalt, no odor or staining | PID = 0.1 |
| 10 | MC SVP-6@9.5-10 | 100 | SP | | 8.0 (SP) Brown fine-grained sand, no rocks present, no odor or staining | PID = 0.1 |
| 15 | MC SVP@14.5-15 | 100 | SP | | 12.0 (SP) Same as above | PID = 0.1 |
| | | | | | 15.5 | |

Bottom of borehole at 15.5 feet.

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# WELL NUMBER SVP-7 A/B



**CLIENT** Police Credit Union **PROJECT NAME** PCU Subsurface

**PROJECT NUMBER** 202006.23 **PROJECT LOCATION** 2550 Irving Street, San Francisco, CA

**DATE STARTED** 5/26/20 **COMPLETED** 5/26/20 **GROUND ELEVATION** **HOLE SIZE** 2" inches

**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.) **GROUND WATER LEVELS:**

**DRILLING METHOD** DPT (direct push technology), 1.6" ID MacroCore **AT TIME OF DRILLING** ---

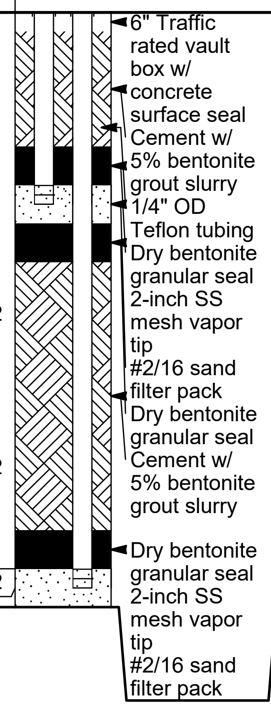
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles **AT END OF DRILLING** ---

**NOTES** 10" dia concrete core, 6" dia vault box. Set perm SVPs @ 5' and 15' bgs **AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0 | | | | | 0.5 4" thick concrete | |
| | | | SP | | (SP) Brown fine-grained sand with minor rock fragments, no odor or staining (fill?) | |
| | | | | | 4.0 | PID = 0 |
| 5 | MC SVP-7@4.5-5 | 100 | SP | | (SP) Same as above (fill?) | |
| | | | | | 8.0 | PID = 0.2 |
| 10 | MC SVP-7@9.5-10 | 100 | SP | | (SP) Brown fine-grained sand, no rocks present, no odor or staining | |
| | | | | | 12.0 | PID = 0.2 |
| 15 | MC SVP-7@14.5-15 | 100 | SP | | (SP) Same as above | PID = 0.2 |

Bottom of borehole at 15.5 feet.

WELL DIAGRAM



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# WELL NUMBER SVP-8 A/B

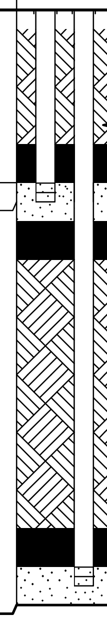


| | | | |
|---|---|---|---|
| **CLIENT** | Police Credit Union | **PROJECT NAME** | PCU Subsurface |
| **PROJECT NUMBER** | 202006.23 | **PROJECT LOCATION** | 2550 Irving Street, San Francisco, CA |
| **DATE STARTED** | 5/24/20 **COMPLETED** 5/24/20 | **GROUND ELEVATION** | **HOLE SIZE** 2" inches |
| **DRILLING CONTRACTOR** | ECA (Environmental Control Associates, Inc.) | **GROUND WATER LEVELS:** | |
| **DRILLING METHOD** | DPT (direct push technology), 1.6" ID MacroCore | **AT TIME OF DRILLING** | --- |
| **LOGGED BY** | Sam Calloway **CHECKED BY** Len Niles | **AT END OF DRILLING** | --- |
| **NOTES** | 10" dia conc core, 6" dia vault box. Set perm SVPs @ 5' and 15' bgs | **AFTER DRILLING** | --- |

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0 – 0.5 | | | | | 4" thick concrete | |
| 0.5 – 5.0 | MC SVP-8@1-1.5 | 100 | SP | | (SP) Tan fine-grained sand with small rock fragments, no odor or staining (fill?) | PID = 0 |
| 5.0 – 9.0 | MC SVP-8@4.5-5 | 100 | SP | | (SP) Tannish brown fine-grained sand, no odor or staining, color darkens with depth | PID = 0 |
| 9.0 – 13.0 | MC SVP-8@9.5-10 | 100 | SP | | (SP) Same as above, tannish brown color consistent throughout | PID = 0 |
| 13.0 – 15.5 | MC SVP-8@14.5-15 | 100 | SP | | (SP) Same as above | PID = 0 |

Bottom of borehole at 15.5 feet.

WELL DIAGRAM




GENERAL BH / TP / WELL - GINT STD US LAB.GDT - 7/9/20 11:41 - K:\BENTLEY\PROJECTS\202006.23 PCU SUBSURFACE.GPJ

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# WELL NUMBER SVP-9 A/B

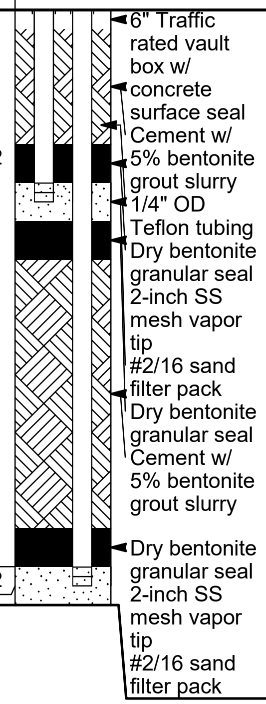


**CLIENT** Police Credit Union
**PROJECT NAME** PCU Subsurface
**PROJECT NUMBER** 202006.23
**PROJECT LOCATION** 2550 Irving Street, San Francisco, CA
**DATE STARTED** 5/23/20 **COMPLETED** 5/23/20
**GROUND ELEVATION** **HOLE SIZE** 2" inches
**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.)
**GROUND WATER LEVELS:**
**DRILLING METHOD** DPT (direct push technology), 1.6" ID MacroCore
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles
**AT END OF DRILLING** ---
**NOTES** 10" dia conc core, 6" dia vault box. Set perm SVPs @ 5' and 15' bgs
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0 | | | | | 0.5 4" concrete | |
| | MC SVP-9@1-1.5 | 100 | SP | | (SP) Tan fine-grained sand, very minor rock fragments present, no odor or staining (fill?) | PID = 0.2 |
| 5 | MC SVP-9@4.5-5 | 100 | SP | | 5.0 (SP) Tan fine-grained sand, no odor or staining | PID = 0.1 |
| 10 | MC SVP-9@9.5-10 | 100 | SP | | 9.0 (SP) Same as above, color becoming tannish brown | PID = 0.1 |
| 15 | MC SVP-9@14.5-15 | 100 | SP | | 13.0 (SP) Same as above. | PID = 0.2 |
| | | | | | 15.5 | |

Bottom of borehole at 15.5 feet.

WELL DIAGRAM



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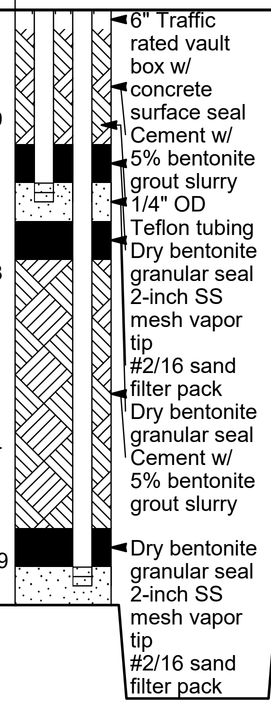
# WELL NUMBER SVP-10 A/B



**CLIENT** Police Credit Union
**PROJECT NAME** PCU Subsurface
**PROJECT NUMBER** 202006.23
**PROJECT LOCATION** 2550 Irving Street, San Francisco, CA
**DATE STARTED** 5/23/20 **COMPLETED** 5/23/20
**GROUND ELEVATION** **HOLE SIZE** 2" inches
**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.)
**GROUND WATER LEVELS:**
**DRILLING METHOD** DPT (direct push technology), 1.6" ID MacroCore
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles
**AT END OF DRILLING** ---
**NOTES** 10" dia conc. core, 6" dia vault box. Set perm SVPs @ 5' and 15' bgs
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0 | | | | | 0.5 4" concrete | |
| | MC SVP-10@1-1.5 | 100 | SP | | (SP) Tan fine-grained sand with small rock fragments present from ~1-3', petroleum odor noticed in soil (fill) | PID = 2.9 |
| 5 | MC SVP-10@4.5-5 | 100 | SP | | 5.0 (SP) Same as above, but no rocks present, petroleum odor becomes stronger as depth increases, staining noticed as color darkens | PID = 6.8 |
| 10 | MC SVP-10@9.5-10 | 100 | SP | | 9.0 (SP) Brownish tan fine-grained sand, stong petroleum odor noticed, soil staining present | PID = 8.4 |
| 15 | MC SVP-10@14.5-15 | 100 | SP | | 13.0 (SP) Same as above | PID = 14.9 |
| | | | | | 15.5 | |

Bottom of borehole at 15.5 feet.



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# WELL NUMBER SVP-11 A/B

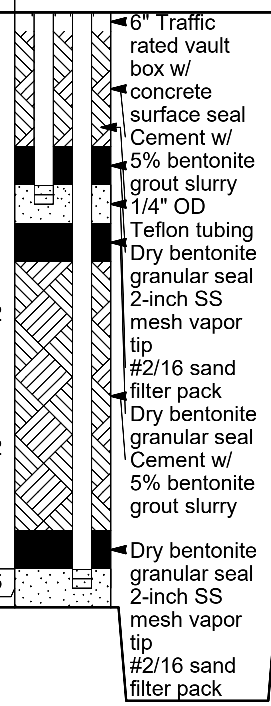


**CLIENT** Police Credit Union
**PROJECT NAME** PCU Subsurface
**PROJECT NUMBER** 202006.23
**PROJECT LOCATION** 2550 Irving Street, San Francisco, CA
**DATE STARTED** 5/26/20 **COMPLETED** 5/26/20
**GROUND ELEVATION** **HOLE SIZE** 2" inches
**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.)
**GROUND WATER LEVELS:**
**DRILLING METHOD** DPT (direct push technology), 1.6" ID MacroCore
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles
**AT END OF DRILLING** ---
**NOTES** 10" dia conc core, 6" dia vault box. Set perm SVPs @ 5' and 15' bgs
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0–0.5 | | | | | 6" thick concrete | |
| 0.5–4.0 | | | SP | | (SP) Brown fine-grained sand with minor rock fragments, no odor or staining (fill?) | PID = 0.1 |
| 4.0–8.0 | MC SVP-11@4.5-5 | 100 | SP | | (SP) Brown fine-grained sand, no odor or staining | |
| 8.0–12.0 | MC SVP-11@9.5-10 | 100 | SP | | (SP) Same as above | PID = 0.2<br>PID = 0.2 |
| 12.0–15.5 | MC SVP-11@14.5-15 | 100 | SP | | (SP) Same as above | PID = 0.5 |

Bottom of borehole at 15.5 feet.

WELL DIAGRAM



# WELL NUMBER SVP-12 A/B



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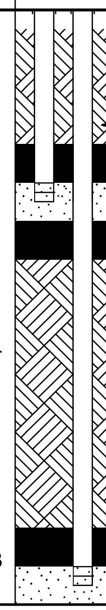
**CLIENT** Police Credit Union
**PROJECT NAME** PCU Subsurface
**PROJECT NUMBER** 202006.23
**PROJECT LOCATION** 2550 Irving Street, San Francisco, CA
**DATE STARTED** 5/23/20 **COMPLETED** 5/23/20
**GROUND ELEVATION** **HOLE SIZE** 2" inches
**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.)
**GROUND WATER LEVELS:**
**DRILLING METHOD** DPT (direct push technology), 1.6" ID MacroCore
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles
**AT END OF DRILLING** ---
**NOTES** 10" dia conc core, 6" dia vault box. Set perm SVPs @ 5' and 15' bgs
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0 – 0.5 | | | | | 4" thick concrete | |
| 0.5 – 5.0 | MC SVP-12@1-1.5 | 100 | SP | | (SP) Tan fine-grained sand, small rock fragments near 4' (fill) | PID = 1 |
| 5.0 – 9.0 | MC SVP-12@4.5-5 | 100 | SP | | (SP) Same as above, slightly turning darker brown (fill) | |
| 9.0 – 13.0 | MC SVP-12@9.5-10 | 100 | SP | | (SP) Same as above, small pieces of rocks and woodis noticed from 12-13' (fill) | PID = 1.4<br>PID = 1 |
| 13.0 – 15.5 | MC SVP-12@14.5-15 | 100 | SP | | (SP) Same as above | PID = 2.3 |

Bottom of borehole at 15.5 feet.

WELL DIAGRAM



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San Francisco, CA 94110
Telephone: 415-391-2510

# WELL NUMBER SVP-13 A/B

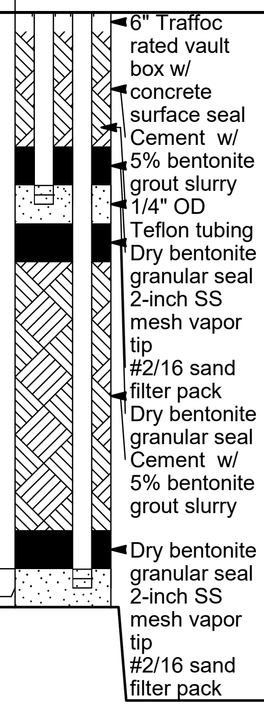


**CLIENT** Police Credit Union
**PROJECT NAME** PCU Subsurface
**PROJECT NUMBER** 202006.23
**PROJECT LOCATION** 2550 Irving Street, San Francisco, CA
**DATE STARTED** 5/23/20 **COMPLETED** 5/23/20
**GROUND ELEVATION** **HOLE SIZE** 2" inches
**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.)
**GROUND WATER LEVELS:**
**DRILLING METHOD** DPT (direct push technology), 1.6" ID MacroCore
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles
**AT END OF DRILLING** ---
**NOTES** 10" dia conc core, 6" dia vault box. Set perm SVPs @ 5' and 15' bgs
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0 | | | | | 0.5 4" thick concrete | |
| | MC SVP-13@1-1.5 | 100 | SP | | (SP) Tannish brown fine-grained sand, no odor or staining | PID = 0 |
| 5 | MC SVP-13@4.5-5 | 100 | SP | | 4.0 (SP) Same as above | |
| | | | SP | | 8.0 (SP) Same as above, color turning darker brown | PID = 0 |
| 10 | MC SVP-13@9.5-10 | 100 | | | | |
| | | | SP | | 12.0 (SP) Same as above, very minor rock pieces | PID = 0 |
| 15 | MC SVP-13@14.5-15 | 100 | | | 15.5 | PID = 0 |

Bottom of borehole at 15.5 feet.

WELL DIAGRAM



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# WELL NUMBER SVP-14 A/B



**CLIENT** Police Credit Union
**PROJECT NAME** PCU Subsurface
**PROJECT NUMBER** 202006.23
**PROJECT LOCATION** 2550 Irving Street, San Francisco, CA
**DATE STARTED** 5/26/20 **COMPLETED** 5/26/20
**GROUND ELEVATION** **HOLE SIZE** 2" inches
**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.)
**GROUND WATER LEVELS:**
**DRILLING METHOD** DPT (direct push technology), 1.6" ID MacroCore
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles
**AT END OF DRILLING** ---
**NOTES** 10" dia conc core, 6" dia vault box. Set perm SVPs @ 5' and 15' bgs
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA | WELL DIAGRAM |
|---|---|---|---|---|---|---|---|
| 0 | | | | 0.5 | 4" thick concrete | | 6" Traffic rated vault box w/ concrete surface seal |
| | | | SP | 4.0 | (SP) Brown fine-grained sand with sparse rock fragments, no odor or staining (fill?) | PID = 0 | Cement w/ 5% bentonite grout slurry; 1/4" OD Teflon tubing |
| 5 | MC SVP-14@4.5-5 | 100 | SP | 8.0 | (SP) Same as above, no rocks present | PID = 0 | Dry bentonite granular seal; 2-inch SS mesh vapor tip; #2/16 sand filter pack; Dry bentonite granular seal |
| 10 | MC SVP-14@9.5-10 | 100 | SP | 12.0 | (SP) Same as above | PID = 0 | Cement w/ 5% bentonite grout slurry |
| 15 | MC SVP-14@14.5-15 | 100 | SP | 15.5 | (SP) Same as above | PID = 0 | Dry bentonite granular seal; 2-inch SS mesh vapor tip; #2/16 sand filter pack |

Bottom of borehole at 15.5 feet.

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# WELL NUMBER SVP-15 A/B

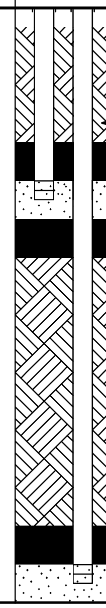


**CLIENT** Police Credit Union
**PROJECT NAME** PCU Subsurface
**PROJECT NUMBER** 202006.23
**PROJECT LOCATION** 2550 Irving Street, San Francisco, CA
**DATE STARTED** 5/23/20 **COMPLETED** 5/23/20
**GROUND ELEVATION** **HOLE SIZE** 2" inches
**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.)
**GROUND WATER LEVELS:**
**DRILLING METHOD** DPT (direct push technology), 1.6" ID MacroCore
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles
**AT END OF DRILLING** ---
**NOTES** 10" dia conc core, 6" dia vault box. Set perm SVPs @ 5' and 15' bgs
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0 | | | | | 0.5 4" thick concrete | |
| | MC SVP-15@1-1.5 | 100 | SP | | (SP) Brown fine-grained sand, no odor or staining, well sorted, poorly graded | PID = 0.1 |
| 5 | MC SVP-15@4.5-5 | 100 | SP | | 5.0 (SP) Same as above | |
| 10 | MC SVP-15@9.5-10 | 100 | SP | | 9.0 (SP) Same as above | PID = 0 |
| 15 | MC SVP-15@14.5-15 | 100 | SP | | 13.0 (SP) Same as above | PID = 0 |
| | | | | | 15.5 | |

Bottom of borehole at 15.5 feet.

WELL DIAGRAM



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# WELL NUMBER SVP-16 A/B

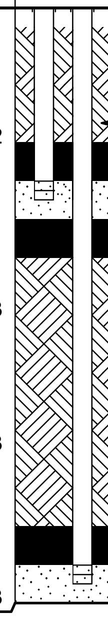


**CLIENT** Police Credit Union
**PROJECT NAME** PCU Subsurface
**PROJECT NUMBER** 202006.23
**PROJECT LOCATION** 2550 Irving Street, San Francisco, CA
**DATE STARTED** 5/26/20 **COMPLETED** 5/26/20
**GROUND ELEVATION** **HOLE SIZE** 2" inches
**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.)
**GROUND WATER LEVELS:**
**DRILLING METHOD** DPT (direct push technology), 1.6" ID MacroCore
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles
**AT END OF DRILLING** ---
**NOTES** 10" dia conc core, 6" dia vault box. Set perm SVPs @ 5' and 15' bgs
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0 | | | | | 0.5 4" thick concrete | |
| | | | SP | | (SP) Brown fine-grained sand, very minor rocks present, no odor or staining (fill?) | PID = 0.2 |
| 5 | MC SVP-16@4.5-5 | 100 | SP | | 4.0 (SP) Same as above, no rocks present | |
| | | | SP | | 8.0 (SP) Same as above, color becomes slightly lighter brown towards 9-12' bgs | PID = 0.3 |
| 10 | MC SVP-16@9.5-10 | 100 | | | | PID = 0.3 |
| 15 | MC SVP-16@14.5-15 | 100 | SP | | 12.0 (SP) Same as above, color is brown | PID = 0.3 |
| | | | | | 15.5 | |

Bottom of borehole at 15.5 feet.

WELL DIAGRAM



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# BORING NUMBER SVP-17



**CLIENT** Police Credit Union
**PROJECT NAME** PCU Subsurface
**PROJECT NUMBER** 202006.23
**PROJECT LOCATION** 2550 Irving Street, San Francisco, CA
**DATE STARTED** 5/28/20 **COMPLETED** 5/28/20
**GROUND ELEVATION** **HOLE SIZE** 2" inches
**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.)
**GROUND WATER LEVELS:**
**DRILLING METHOD** DPT (direct push technology), 1.6" ID MacroCore
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles
**AT END OF DRILLING** ---
**NOTES** Temporary SVP @ 15' bgs, removed, grouted boring w/ cement
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0 | | | | | 0.7 Asphalt ~8" thick. | |
| | | | SP | | (SP) Brown fine-grained sand with minor asphalt debris toward 1-2.5, asphalt debris is not present from ~2.5-4' (fill) | PID = 0.2 |
| 5 | MC SVP-17@4.5-5 | 100 | SP | | 4.0 (SP) Brown fine-grained sand, no odor or staining, very minor rock fragments present and does not appear to be asphalt (fill?) | PID = 0.2 |
| 10 | MC SVP-17@9.5-10 | 100 | SP | | 8.0 (SP) Brown fine-grained sand, no odor or staining | PID = 0.3 |
| 15 | MC SVP-17@14.5-15 | 100 | SP | | 12.0 (SP) Same as above | PID = 0.3 |
| | | | | | 15.5 | |

Bottom of borehole at 15.5 feet.

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Telephone: 415-391-2510

# WELL NUMBER SVP-18 A/B

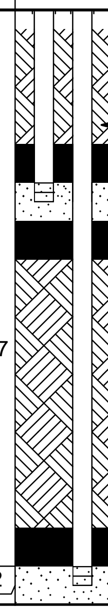


**CLIENT** Police Credit Union
**PROJECT NAME** PCU Subsurface
**PROJECT NUMBER** 202006.23
**PROJECT LOCATION** 2550 Irving Street, San Francisco, CA
**DATE STARTED** 5/23/20 **COMPLETED** 5/23/20
**GROUND ELEVATION** **HOLE SIZE** 2" inches
**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.)
**GROUND WATER LEVELS:**
**DRILLING METHOD** DPT (direct push technology), 1.6" ID MacroCore
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles
**AT END OF DRILLING** ---
**NOTES** 10" dia conc core, 6" dia vault box. Set perm SVPs @ 5' and 15' bgs
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0 | | | | | 0.5 4" thick concrete | |
| | MC SVP-18@1-1.5 | 100 | SP | | (SP) Tan fine-grained sand with small rock fragments, no odor or staining (fill?) | PID = 1 |
| 5 | MC SVP-18@4.5-5 | 100 | SP | | 5.0 (SP) Tan fine-grained sand, no odor or staining | |
| 10 | MC SVP-18@9.5-10 | 100 | SP | | 9.0 (SP) Same as above | PID = 0.07 |
| 15 | MC SVP-18@14.5-15 | 100 | SP | | 13.0 (SP) Same as above | PID = 0.2 |
| | | | | | 15.5 | |

Bottom of borehole at 15.5 feet.

WELL DIAGRAM



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# BORING NUMBER SVP-19 A/B



**CLIENT** Police Credit Union
**PROJECT NAME** PCU Subsurface
**PROJECT NUMBER** 202006.23
**PROJECT LOCATION** 2525 Irving Street, San Francisco, CA
**DATE STARTED** 5/27/20 **COMPLETED** 5/27/20
**GROUND ELEVATION** **HOLE SIZE** 2" inches
**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.)
**GROUND WATER LEVELS:**
**DRILLING METHOD** DPT (direct push technology), 1.6" ID MacroCore
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles
**AT END OF DRILLING** ---
**NOTES** Temporary SVPs @ 5' and 15" bgs, removed, grouted boring w/ cement
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0 | | | | | 0.5 Asphalt ~2" thick. | |
| | | | SP | | (SP) Brown fine-grained sand, no odor or staining, small asphalt pieces mix in from ~0.5-3' bgs(fill) | PID = 0.3 |
| 5 | MC SVP-19@4.5-5 | 100 | SP | | 4.0 (SP) Brown fine-grained sand, no odor or staining | PID = 0.3 |
| 10 | MC SVP-19 @9.5-10 | 100 | SP | | 8.0 (SP) Same as above | PID = 0.4 |
| 15 | MC SVP-19@14.5-15 | 100 | SP | | 12.0 (SP) Same as above<br>15.5 | PID = 0.4 |

Bottom of borehole at 15.5 feet.

GENERAL BH / TP / WELL - GINT STD US LAB.GDT - 7/6/20 16:02 - K:\BENTLEY\PROJECTS\202006.23 PCU SUBSURFACE.GPJ

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# BORING NUMBER SVP-20 A/B



**CLIENT** Police Credit Union
**PROJECT NAME** PCU Subsurface
**PROJECT NUMBER** 202006.23
**PROJECT LOCATION** 2525 Irving Street, San Francisco, CA
**DATE STARTED** 5/27/20 **COMPLETED** 5/27/20
**GROUND ELEVATION** **HOLE SIZE** 2" inches
**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.)
**GROUND WATER LEVELS:**
**DRILLING METHOD** DPT (direct push technology), 1.6" ID MacroCore
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles
**AT END OF DRILLING** ---
**NOTES** SVP at 5' and 15', removed SVPs, grouted boring w/ cement
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0 | | | | 0.5 | Asphalt ~2" thick | |
| | | | SP | 4.0 | (SP) Brown fine-grained sand, no odor or staining. Asphalt debris at 0-0.5' bgs (fill?) | PID = 0 |
| 5 | MC SVP-20@4.5-5 | 100 | SP | 8.0 | (SP) Brown fine-grained sand, very minor rock fragments near 7-8' | PID = 0 |
| 10 | MC SVP-20@9.5-10 | 100 | SP | 12.0 | (SP) Same as above, no rocks present | PID = 0 |
| 15 | MC SVP-20@14.5-15 | 100 | SP | 15.5 | (SP) Same as above | PID = 0 |

Bottom of borehole at 15.5 feet.

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# BORING NUMBER SVP-21 A/B



**CLIENT** Police Credit Union
**PROJECT NAME** PCU Subsurface
**PROJECT NUMBER** 202006.23
**PROJECT LOCATION** 2525 Irving Street, San Francisco, CA
**DATE STARTED** 5/27/20 **COMPLETED** 2/27/20
**GROUND ELEVATION** **HOLE SIZE** 2" inches
**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.)
**GROUND WATER LEVELS:**
**DRILLING METHOD** DPT (direct push technology), 1.6" ID MacroCore
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles
**AT END OF DRILLING** ---
**NOTES** Temporary SVPs @ 5' and 15' bgs, removed, grouted w/ cement
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0 | | | | 0.5 | Asphalt ~ 2" thick | |
| | | | SP | 4.0 | (SP) Brown fine-grained sand with asphalt bebris from approximately 1-2', asphalt debris is minor compared to other borings, soil is mixed throughout, no odor or staining (fill) | PID = 0 |
| 5 | MC SVP-21@4.5-5 | 100 | SP | 8.0 | (SP) Brown fine-grained sand, no odor or staining, small rock fragments present, but not asphalt (fill?) | PID = 0.1 |
| 10 | MC SVP-21@9.5-10 | 100 | SP | 12.0 | (SP) Brown fine-grained sand, no odor or staining | PID = 0.1 |
| 15 | MC SVP-21@14.5-15 | 100 | SP | 15.5 | (SP) Same as above | PID = 0.1 |

Bottom of borehole at 15.5 feet.

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# BORING NUMBER SVP-22 A/B



**CLIENT** Police Credit Union
**PROJECT NAME** PCU Subsurface
**PROJECT NUMBER** 202006.23
**PROJECT LOCATION** 2525 Irving Street, San Francisco, CA
**DATE STARTED** 5/27/20 **COMPLETED** 5/27/20
**GROUND ELEVATION** **HOLE SIZE** 2" inches
**DRILLING CONTRACTOR** ECA (Environmental Control Associates, Inc.)
**GROUND WATER LEVELS:**
**DRILLING METHOD** DPT (direct push technology), 1.6"MacroCore
**AT TIME OF DRILLING** ---
**LOGGED BY** Sam Calloway **CHECKED BY** Len Niles
**AT END OF DRILLING** ---
**NOTES** Temp SVPs set at 5' and 15' bgs, removed, grouted boring w/ cement
**AFTER DRILLING** ---

| DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % | U.S.C.S. | GRAPHIC LOG | MATERIAL DESCRIPTION | ENVIRONMENTAL DATA |
|---|---|---|---|---|---|---|
| 0 | | | | | 0.5 Asphalt ~2" thick | |
| | | | SP | | (SP) Brown fine-grained sand, no odor or staining; asphalt debris from 0-0.5 feet (fill) | PID = 0 |
| 5 | MC SVP-22@4.5-5 | 100 | SP | | 4.0 (SP) Brown fine-grained sand, no odor or staining | PID = 0.2 |
| 10 | MC SVP-22@9.5-10 | 100 | SP | | 8.0 (SP) Same as above | PID = 0.2 |
| 15 | MC SVP-22@14.5-15 | 100 | SP | | 12.0 (SP) Same as above | PID = 0.3 |
| | | | | | 15.5 | |

Bottom of borehole at 15.5 feet.

# APPENDIX E
# Geotechnical Laboratory Testing Data

A3GEO

A3GEO, Inc.
821 Bancroft Way
Berkeley, CA 94710, (510) 705-1664

DRY DENSITY
ASTM 7263

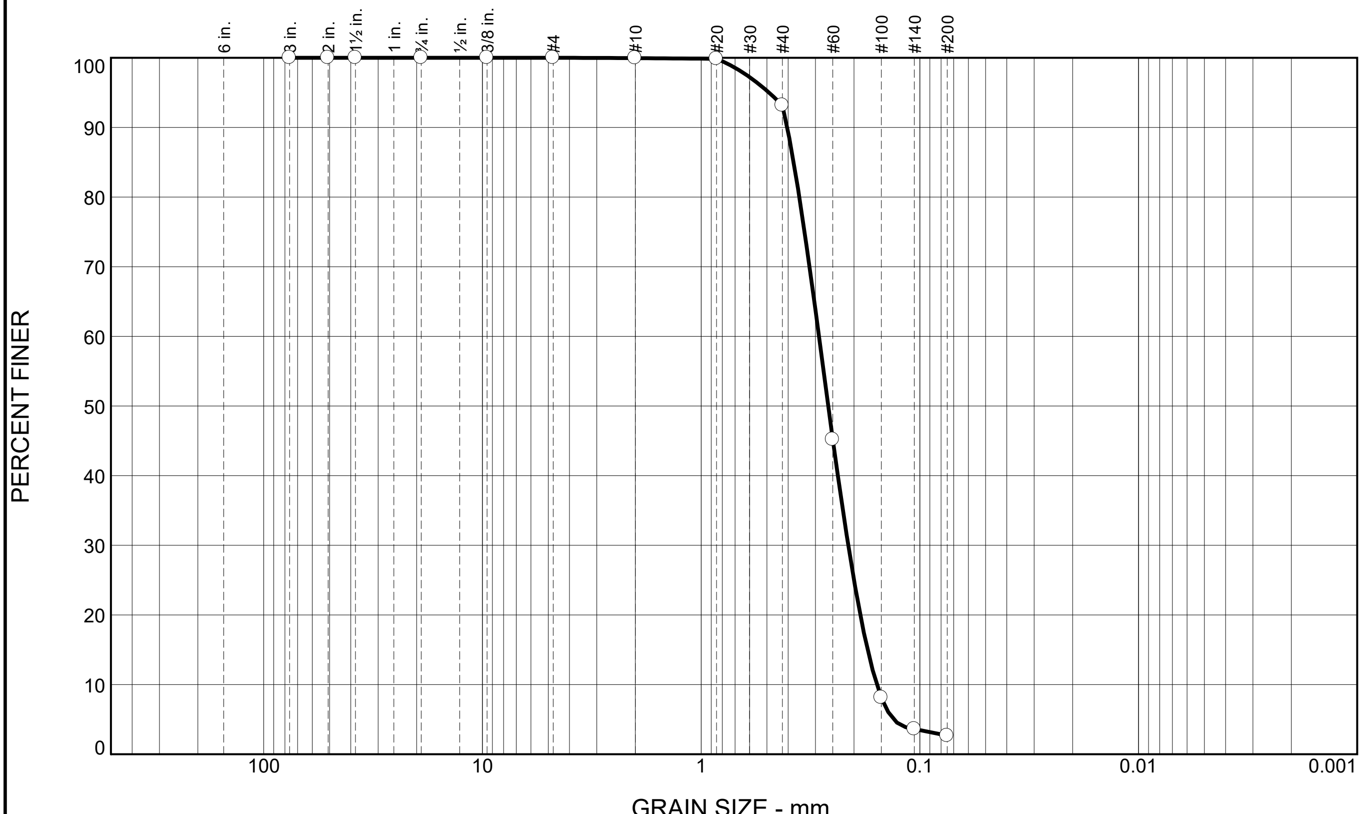
| | | | | | |
|---|---|---|---|---|---|
| Project No. : | 1146-4B | Project Name : | 2550 Irving St, Design Phase | Date Received : | 02/17/22 |
| Client : | Tenderloin Neighborhood Development Corporation | | | Date Tested : | 02/24/22 |
| Sample Notes: | | | | Tested By: | RES |
| | | | | Checked By: | DKM |
| | | | | Revision Date: | 3/12/2020 |

| | | | | | | |
|---|---|---|---|---|---|---|
| Sample Number | # | 1B | 4B | 3B | 6B | 4B |
| Boring Number | # | GB-1 | GB-1 | GB-2 | GB-2 | GB-3 |
| Sample Depth | (ft) | 3.0-3.5 | 10.5-11.0 | 5.5-6.0 | 20.5-21.0 | 8.0-8.5 |
| Length Sample (Lo) | (in) | 5.684 | 6.000 | 6.000 | 6.000 | 5.857 |
| Diameter of Sample | (in) | 2.430 | 2.410 | 2.418 | 2.415 | 2.415 |
| Weight of Sample+Liner | (gms.) | 978.38 | 1034.04 | 998.03 | 1056.24 | 988.32 |
| Weight of Liner | (gms.) | 253.82 | 276.82 | 262.81 | 266.74 | 264.63 |
| Weight of Sample | (gms.) | 724.6 | 757.2 | 735.2 | 789.5 | 723.7 |
| Dry Weight of Sample | (gms.) | 716.0 | 734.2 | 716.8 | 768.4 | 708.0 |
| Volume of Sample | ($in^3$) | 26.4 | 27.4 | 27.6 | 27.5 | 26.8 |
| Dry Density of Sample | ($gm/in^3$) | 27.2 | 26.8 | 26.0 | 28.0 | 26.4 |
| **Dry Density of Sample** | **($lb/ft^3$)** | **103.5** | **102.2** | **99.1** | **106.5** | **100.5** |
| | | | | | | |
| Container Number | # | #204 | #205 | #202 | #200 | #300 |
| Container Weight | (gms.) | 270.98 | 271.69 | 273.30 | 271.62 | 190.50 |
| Wet Soil + Container | (gms.) | 995.22 | 1028.13 | 1008.01 | 1059.87 | 913.86 |
| Dry Soil + Container | (gms.) | 986.65 | 1005.17 | 989.58 | 1038.79 | 898.14 |
| Weight of Water | (gms.) | 8.57 | 22.96 | 18.43 | 21.08 | 15.72 |
| Weight of Dry Soil | (gms.) | 715.67 | 733.48 | 716.28 | 767.17 | 707.64 |
| **Moisture Content** | (%) | **1.2** | **3.1** | **2.6** | **2.7** | **2.2** |

| | | | | | | |
|---|---|---|---|---|---|---|
| Sample Number | # | 6B | 8B | | | |
| Boring Number | # | GB-3 | GB-3 | | | |
| Sample Depth | (ft) | 15.5-16.0 | 25.5-26.0 | | | |
| Length Sample (Lo) | (in) | 5.920 | 5.940 | | | |
| Diameter of Sample | (in) | 2.412 | 2.412 | | | |
| Weight of Sample+Liner | (gms.) | 1060.24 | 1085.96 | | | |
| Weight of Liner | (gms.) | 266.48 | 292.74 | | | |
| Weight of Sample | (gms.) | 793.76 | 793.22 | | | |
| Dry Weight of Sample | (gms.) | 775.07 | 775.64 | | | |
| Volume of Sample | ($in^3$) | 27.05 | 27.14 | | | |
| Dry Density of Sample | ($gm/in^3$) | 28.65 | 28.58 | | | |
| **Dry Density of Sample** | **($lb/ft^3$)** | **109.2** | **108.9** | | | |
| | | | | | | |
| Container Number | # | #301 | #303 | | | |
| Container Weight | (gms.) | 279.59 | 306.92 | | | |
| Wet Soil + Container | (gms.) | 1086.91 | 1099.73 | | | |
| Dry Soil + Container | (gms.) | 1067.90 | 1082.16 | | | |
| Weight of Water | (gms.) | 19.01 | 17.57 | | | |
| Weight of Dry Soil | (gms.) | 788.31 | 775.24 | | | |
| **Moisture Content** | (%) | **2.4** | **2.3** | | | |

Additional Laboratory Notes:

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 0.0 | 0.0 | 6.8 | 90.5 | 2.7 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3.00 | 100.0 | | |
| 2.00 | 100.0 | | |
| 1.5 | 100.0 | | |
| 0.75 | 100.0 | | |
| 0.375 | 100.0 | | |
| #4 | 100.0 | | |
| #10 | 100.0 | | |
| #20 | 99.9 | | |
| #40 | 93.2 | | |
| #60 | 45.2 | | |
| #100 | 8.1 | | |
| #140 | 3.6 | | |
| #200 | 2.7 | | |

* (no specification provided)

**Material Description**

Poorly-Graded Sand, light brown, fine-medium sand, trace fines, non-plastic, moist

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{90}$= 0.4039 $D_{85}$= 0.3773 $D_{60}$= 0.2892
$D_{50}$= 0.2623 $D_{30}$= 0.2124 $D_{15}$= 0.1733
$D_{10}$= 0.1574 $C_u$= 1.84 $C_c$= 0.99

**Classification**

USCS= SP AASHTO=

**Remarks**

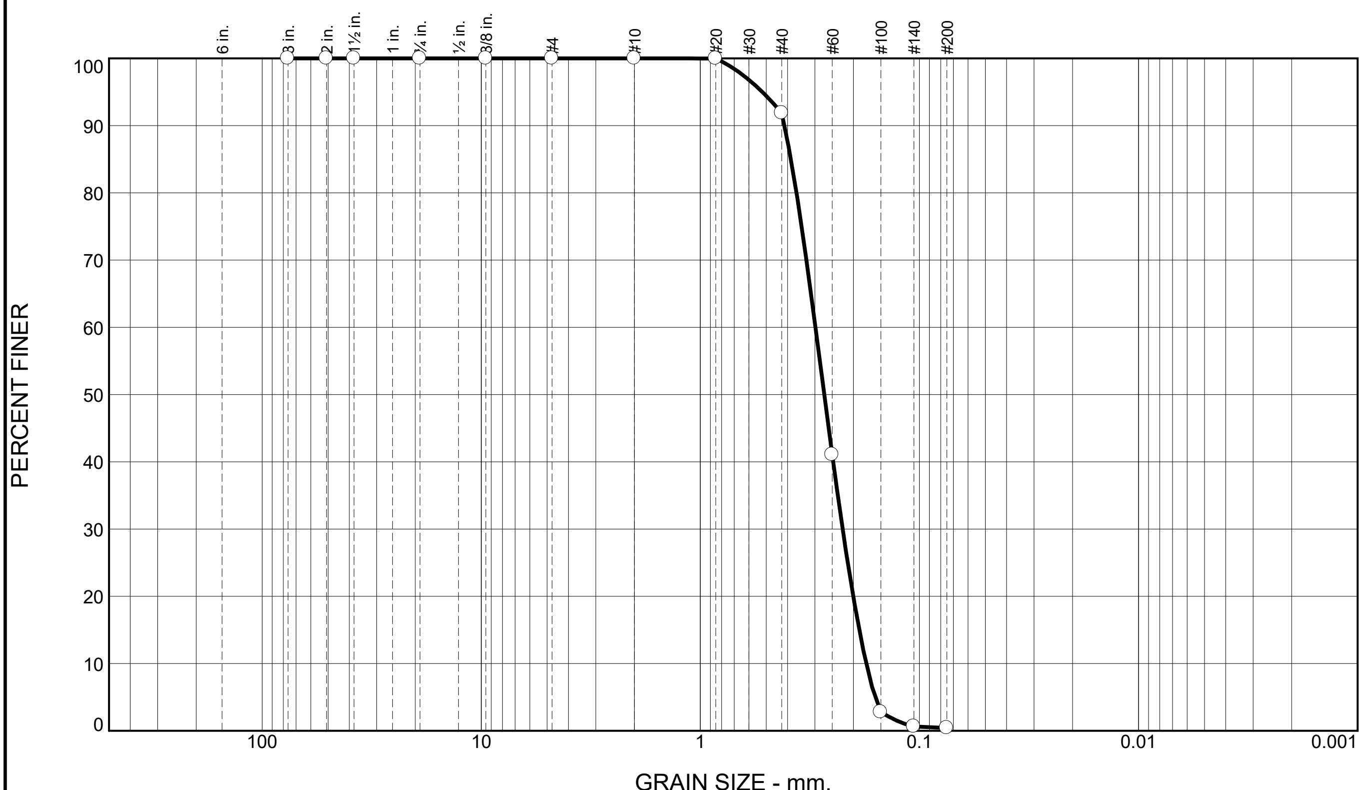
**Location:** GB-1
**Sample Number:** 1B **Depth:** 3.0' **Date:** 2/24/2022

A3GEO

**Client:** Tenderloin Neighborhood Development Corporation
**Project:** 2550 Irving St, Design Phase

**Project No:** 1146-4B **Figure**

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 0.0 | 0.0 | 8.1 | 91.5 | 0.4 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3.00 | 100.0 | | |
| 2.00 | 100.0 | | |
| 1.5 | 100.0 | | |
| 0.75 | 100.0 | | |
| 0.375 | 100.0 | | |
| #4 | 100.0 | | |
| #10 | 100.0 | | |
| #20 | 100.0 | | |
| #40 | 91.9 | | |
| #60 | 41.0 | | |
| #100 | 2.8 | | |
| #140 | 0.6 | | |
| #200 | 0.4 | | |

* (no specification provided)

**Material Description**

Poorly-Graded Sand, light brown, fine-medium sand, trace fines, non-plastic, moist

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{90}$= 0.4129 $D_{85}$= 0.3859 $D_{60}$= 0.2982
$D_{50}$= 0.2718 $D_{30}$= 0.2241 $D_{15}$= 0.1882
$D_{10}$= 0.1749 $C_u$= 1.70 $C_c$= 0.96

**Classification**

USCS= SP AASHTO=

**Remarks**

**Location:** GB-1
**Sample Number:** 4B **Depth:** 10.5' **Date:** 2/24/2022

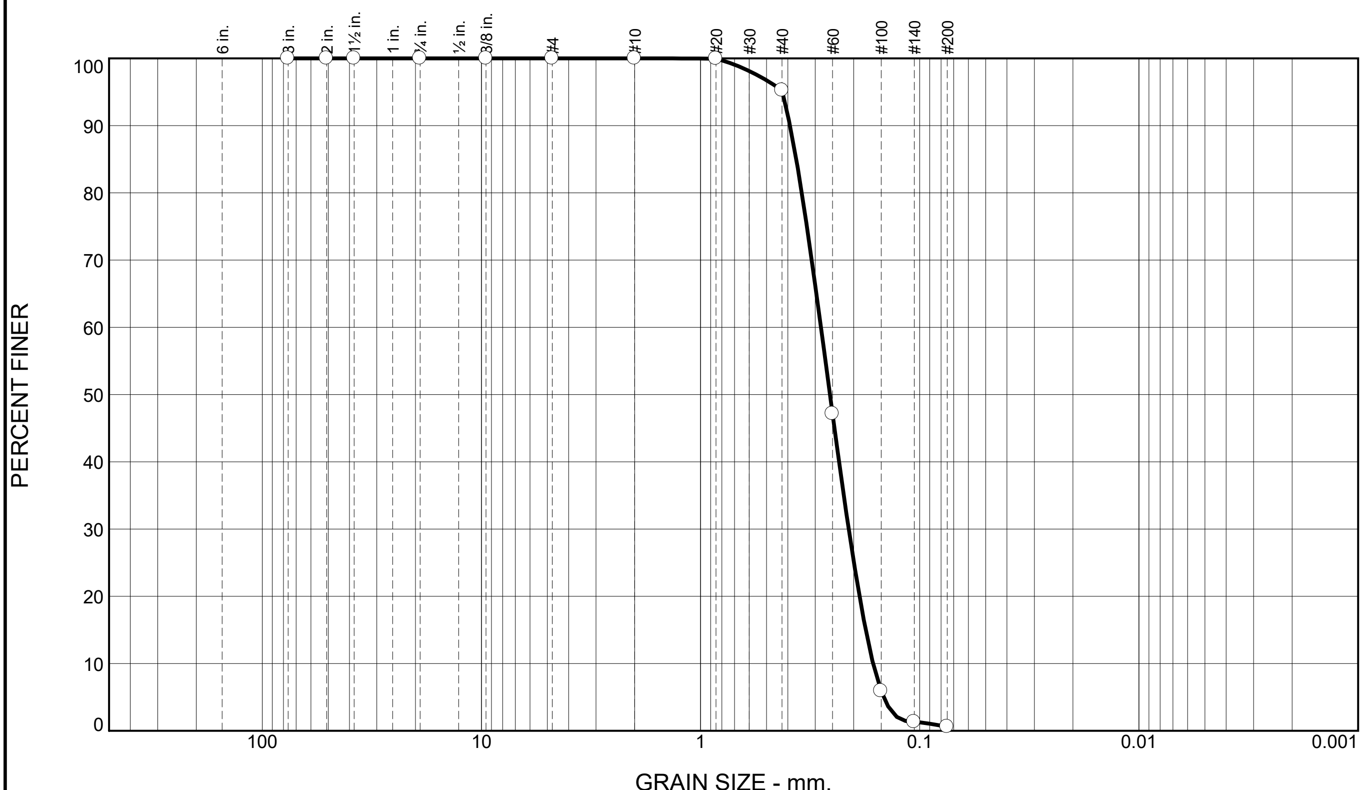
A3GEO

**Client:** Tenderloin Neighborhood Development Corporation
**Project:** 2550 Irving St, Design Phase

**Project No:** 1146-4B **Figure**

**Tested By:** RES **Checked By:** DKM

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 0.0 | 0.0 | 4.8 | 94.6 | 0.6 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3.00 | 100.0 | | |
| 2.00 | 100.0 | | |
| 1.5 | 100.0 | | |
| 0.75 | 100.0 | | |
| 0.375 | 100.0 | | |
| #4 | 100.0 | | |
| #10 | 100.0 | | |
| #20 | 100.0 | | |
| #40 | 95.2 | | |
| #60 | 47.1 | | |
| #100 | 5.9 | | |
| #140 | 1.3 | | |
| #200 | 0.6 | | |

* (no specification provided)

**Material Description**

Poorly-Graded Sand, light brown, fine-medium sand, trace fines, non-plastic, moist

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{90}$= 0.3908 $D_{85}$= 0.3661 $D_{60}$= 0.2823
$D_{50}$= 0.2569 $D_{30}$= 0.2109 $D_{15}$= 0.1764
$D_{10}$= 0.1632 $C_u$= 1.73 $C_c$= 0.97

**Classification**

USCS= SP AASHTO=

**Remarks**

**Location:** GB-2
**Sample Number:** 3B **Depth:** 5.5' **Date:** 2/24/2022

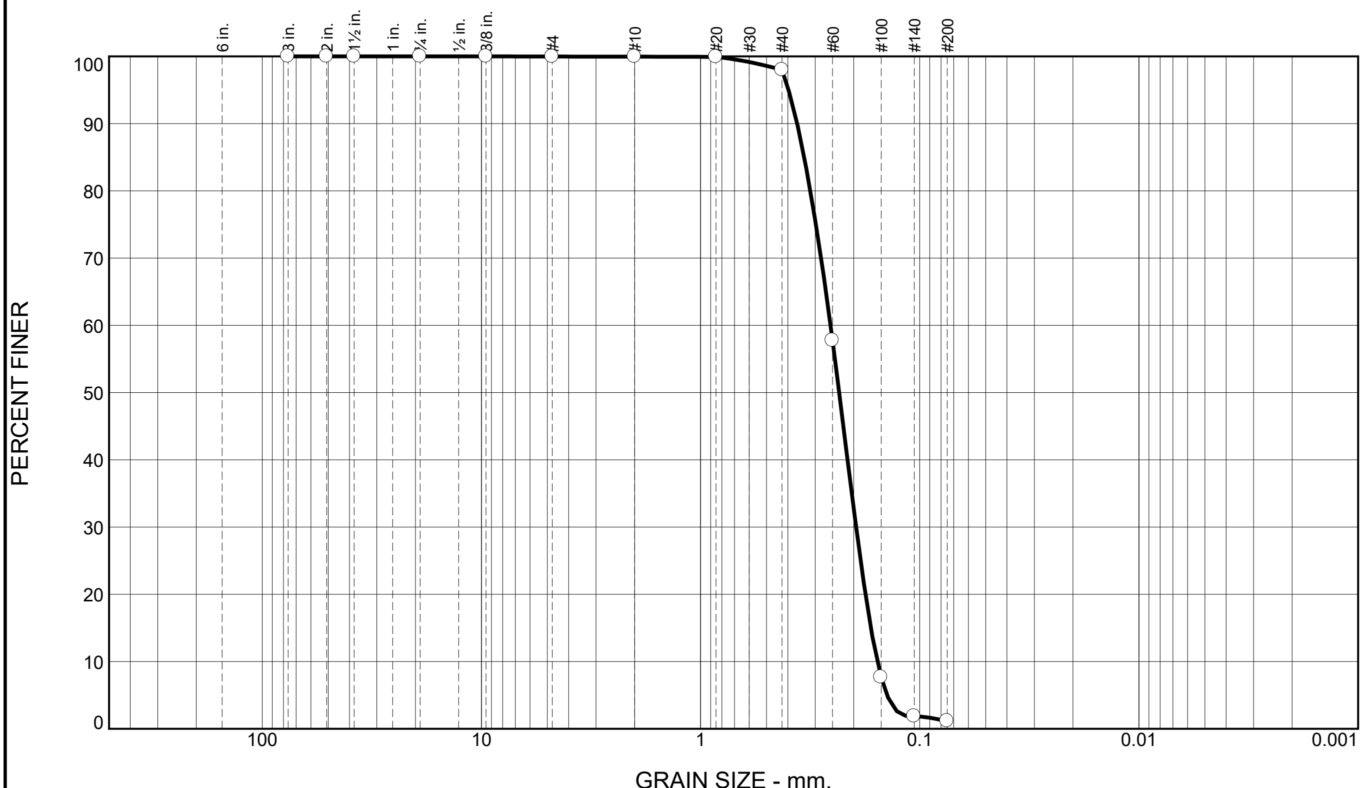
A3GEO

**Client:** Tenderloin Neighborhood Development Corporation
**Project:** 2550 Irving St, Design Phase

**Project No:** 1146-4B **Figure**

**Tested By:** RES **Checked By:** DKM

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 0.0 | 0.1 | 1.9 | 96.9 | 1.1 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3.00 | 100.0 | | |
| 2.00 | 100.0 | | |
| 1.5 | 100.0 | | |
| 0.75 | 100.0 | | |
| 0.375 | 100.0 | | |
| #4 | 100.0 | | |
| #10 | 99.9 | | |
| #20 | 99.9 | | |
| #40 | 98.0 | | |
| #60 | 57.7 | | |
| #100 | 7.7 | | |
| #140 | 1.9 | | |
| #200 | 1.1 | | |

* (no specification provided)

**Material Description**

Poorly-Graded Sand, light brown, fine-medium sand, trace fines, non-plastic, moist

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{90}$= 0.3626 $D_{85}$= 0.3368 $D_{60}$= 0.2553
$D_{50}$= 0.2331 $D_{30}$= 0.1952 $D_{15}$= 0.1671
$D_{10}$= 0.1561 $C_u$= 1.64 $C_c$= 0.96

**Classification**

USCS= SP AASHTO=

**Remarks**

**Location:** GB-2
**Sample Number:** 6B **Depth:** 20.5' **Date:** 2/24/2022

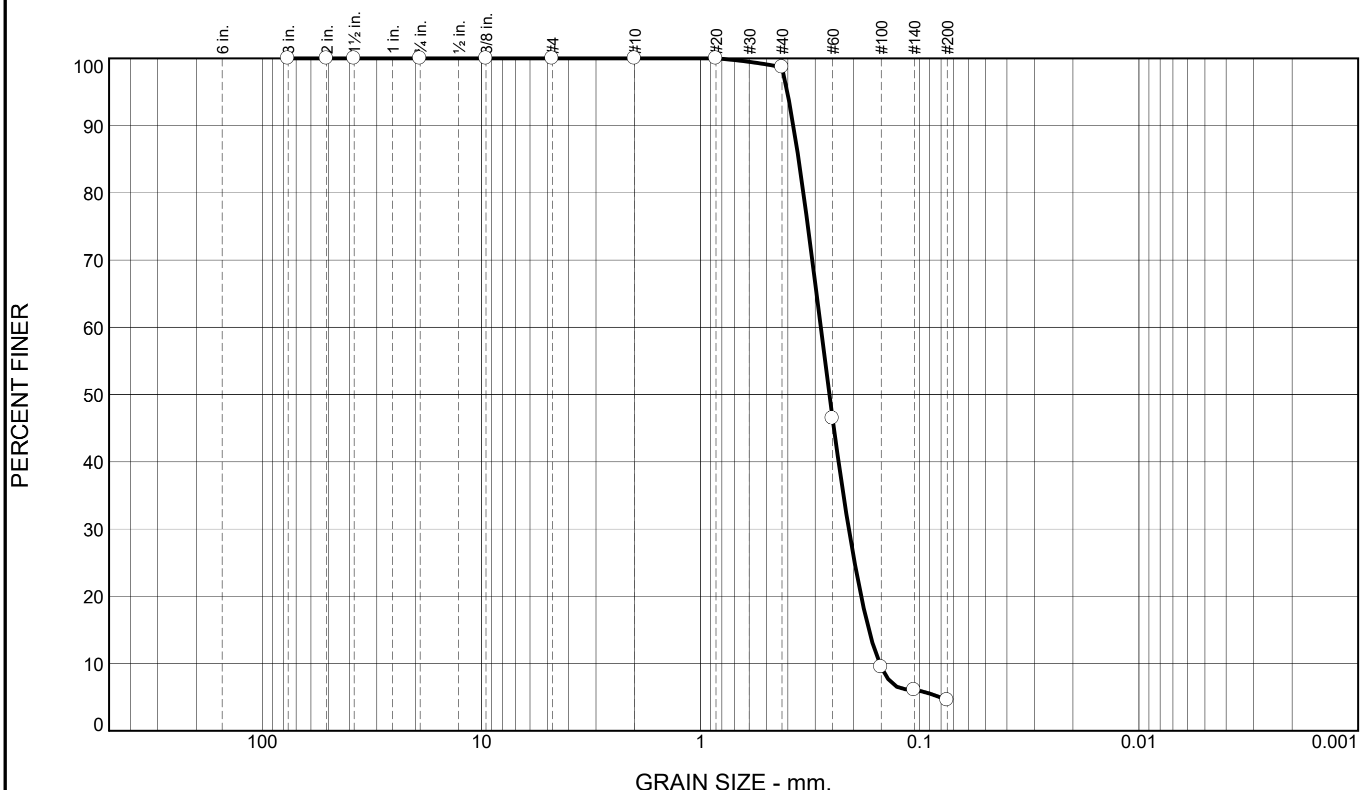
A3GEO

**Client:** Tenderloin Neighborhood Development Corporation
**Project:** 2550 Irving St, Design Phase

**Project No:** 1146-4B **Figure**

**Tested By:** RES **Checked By:** DKM

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 0.0 | 0.0 | 1.3 | 94.1 | 4.6 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3.00 | 100.0 | | |
| 2.00 | 100.0 | | |
| 1.5 | 100.0 | | |
| 0.75 | 100.0 | | |
| 0.375 | 100.0 | | |
| #4 | 100.0 | | |
| #10 | 100.0 | | |
| #20 | 100.0 | | |
| #40 | 98.7 | | |
| #60 | 46.5 | | |
| #100 | 9.5 | | |
| #140 | 6.1 | | |
| #200 | 4.6 | | |

* (no specification provided)

**Material Description**

Poorly-Graded Sand, light brown, fine-medium sand, trace fines, non-plastic, moist

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{90}$= 0.3774 $D_{85}$= 0.3572 $D_{60}$= 0.2828
$D_{50}$= 0.2584 $D_{30}$= 0.2107 $D_{15}$= 0.1706
$D_{10}$= 0.1525 $C_u$= 1.85 $C_c$= 1.03

**Classification**

USCS= SP AASHTO=

**Remarks**

**Location:** GB-2
**Sample Number:** 9 **Depth:** 35.0' **Date:** 2/24/2022

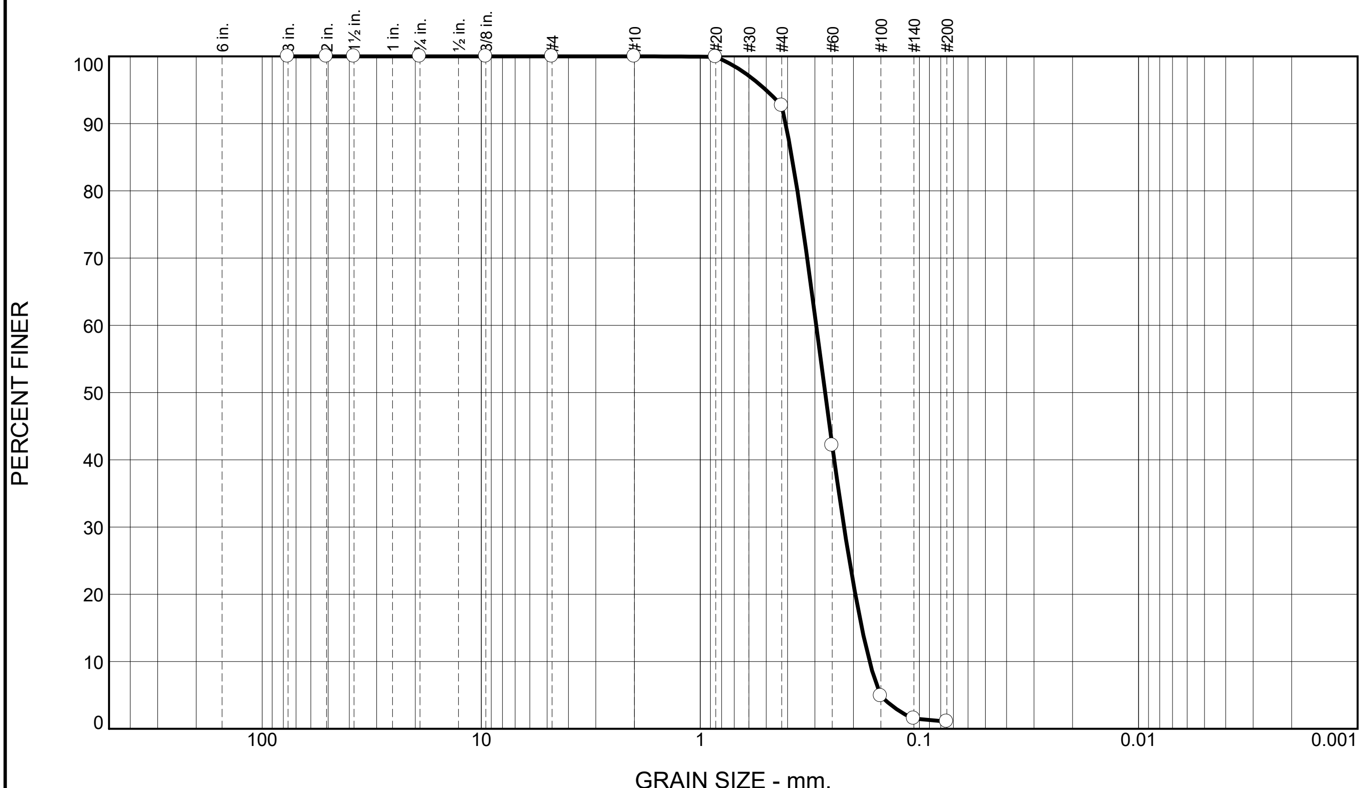
A3GEO

**Client:** Tenderloin Neighborhood Development Corporation
**Project:** 2550 Irving St, Design Phase

**Project No:** 1146-4B **Figure**

**Tested By:** RES **Checked By:** DKM

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 0.0 | 0.0 | 7.3 | 91.6 | 1.1 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3.00 | 100.0 | | |
| 2.00 | 100.0 | | |
| 1.5 | 100.0 | | |
| 0.75 | 100.0 | | |
| 0.375 | 100.0 | | |
| #4 | 100.0 | | |
| #10 | 100.0 | | |
| #20 | 99.9 | | |
| #40 | 92.7 | | |
| #60 | 42.1 | | |
| #100 | 4.9 | | |
| #140 | 1.5 | | |
| #200 | 1.1 | | |

* (no specification provided)

**Material Description**

Poorly-Graded Sand, light brown, fine-medium sand, trace fines, non-plastic, moist

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{90}$= 0.4079 $D_{85}$= 0.3819 $D_{60}$= 0.2958
$D_{50}$= 0.2694 $D_{30}$= 0.2208 $D_{15}$= 0.1832
$D_{10}$= 0.1688 $C_u$= 1.75 $C_c$= 0.98

**Classification**

USCS= SP AASHTO=

**Remarks**

**Location:** GB-3
**Sample Number:** 4B **Depth:** 8.0' **Date:** 2/24/2022

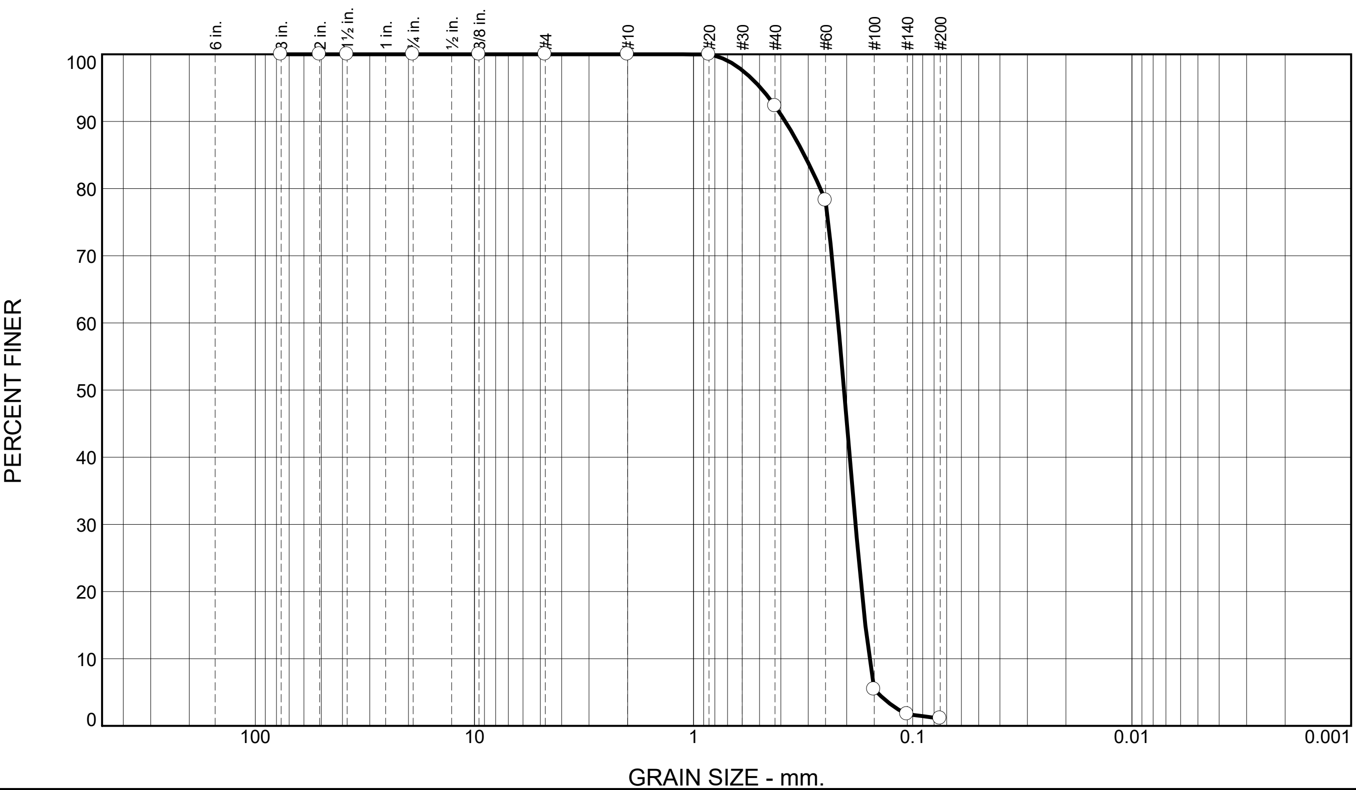
A3GEO

**Client:** Tenderloin Neighborhood Development Corporation
**Project:** 2550 Irving St, Design Phase

**Project No:** 1146-4B **Figure**

**Tested By:** RES **Checked By:** DKM

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 0.0 | 0.0 | 7.7 | 91.2 | 1.1 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3.00 | 100.0 | | |
| 2.00 | 100.0 | | |
| 1.5 | 100.0 | | |
| 0.75 | 100.0 | | |
| 0.375 | 100.0 | | |
| #4 | 100.0 | | |
| #10 | 100.0 | | |
| #20 | 100.0 | | |
| #40 | 92.3 | | |
| #60 | 78.2 | | |
| #100 | 5.4 | | |
| #140 | 1.7 | | |
| #200 | 1.1 | | |

* (no specification provided)

**Material Description**

Poorly-Graded Sand, light brown, fine-medium sand, trace fines, non-plastic, moist

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{90}$= 0.3815 $D_{85}$= 0.3124 $D_{60}$= 0.2189
$D_{50}$= 0.2058 $D_{30}$= 0.1824 $D_{15}$= 0.1645
$D_{10}$= 0.1576 $C_u$= 1.39 $C_c$= 0.96

**Classification**

USCS= SP AASHTO=

**Remarks**

**Location:** GB-3
**Sample Number:** 6B **Depth:** 15.5' **Date:** 2/24/2022

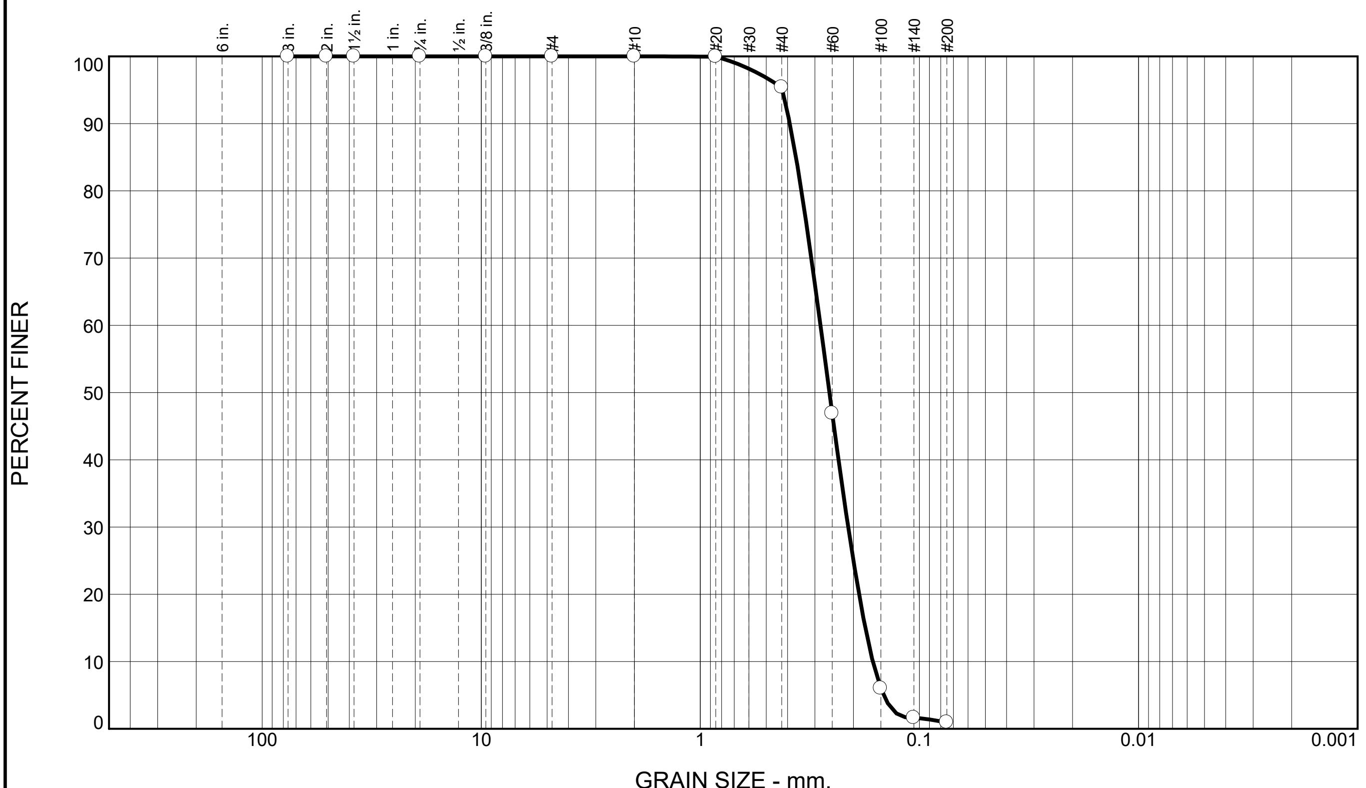
A3GEO

**Client:** Tenderloin Neighborhood Development Corporation
**Project:** 2550 Irving St, Design Phase

**Project No:** 1146-4B **Figure**

**Tested By:** RES **Checked By:** DKM

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 0.0 | 0.0 | 4.6 | 94.5 | 0.9 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3.00 | 100.0 | | |
| 2.00 | 100.0 | | |
| 1.5 | 100.0 | | |
| 0.75 | 100.0 | | |
| 0.375 | 100.0 | | |
| #4 | 100.0 | | |
| #10 | 100.0 | | |
| #20 | 99.9 | | |
| #40 | 95.4 | | |
| #60 | 46.9 | | |
| #100 | 6.0 | | |
| #140 | 1.6 | | |
| #200 | 0.9 | | |

* (no specification provided)

**Material Description**

Poorly-Graded Sand, light brown, fine-medium sand, trace fines, non-plastic, moist

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{90}$= 0.3904 $D_{85}$= 0.3660 $D_{60}$= 0.2829
$D_{50}$= 0.2575 $D_{30}$= 0.2114 $D_{15}$= 0.1765
$D_{10}$= 0.1632 $C_u$= 1.73 $C_c$= 0.97

**Classification**

USCS= SP AASHTO=

**Remarks**

**Location:** GB-3
**Sample Number:** 8B **Depth:** 25.5' **Date:** 2/24/2022

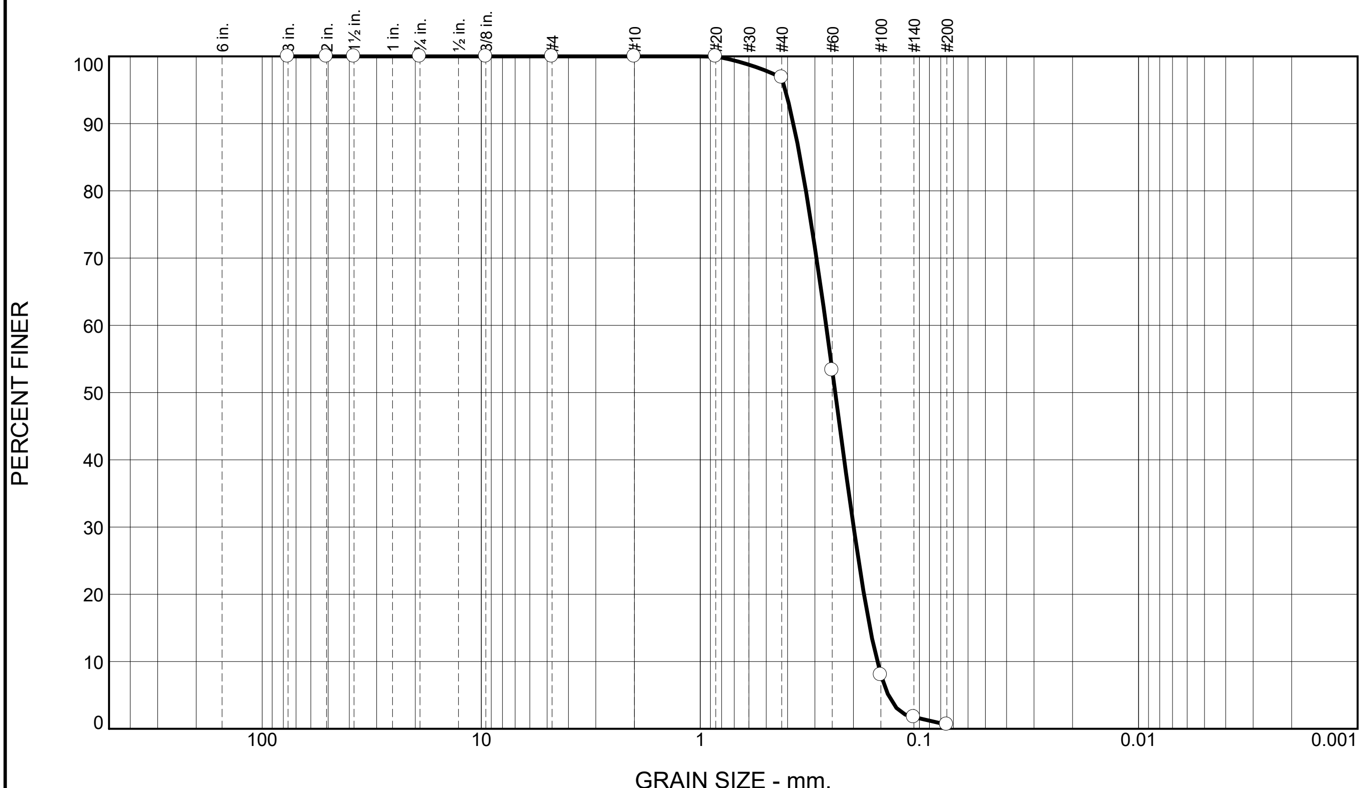
A3GEO

**Client:** Tenderloin Neighborhood Development Corporation
**Project:** 2550 Irving St, Design Phase

**Project No:** 1146-4B **Figure**

**Tested By:** RES **Checked By:** DKM

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 0.0 | 0.0 | 3.2 | 96.2 | 0.6 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3.00 | 100.0 | | |
| 2.00 | 100.0 | | |
| 1.5 | 100.0 | | |
| 0.75 | 100.0 | | |
| 0.375 | 100.0 | | |
| #4 | 100.0 | | |
| #10 | 100.0 | | |
| #20 | 100.0 | | |
| #40 | 96.8 | | |
| #60 | 53.3 | | |
| #100 | 8.0 | | |
| #140 | 1.7 | | |
| #200 | 0.6 | | |

* (no specification provided)

**Material Description**

Poorly-Graded Sand, light brown, fine-medium sand, trace fines, non-plastic, moist

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{90}$= 0.3761 $D_{85}$= 0.3507 $D_{60}$= 0.2666
$D_{50}$= 0.2422 $D_{30}$= 0.1997 $D_{15}$= 0.1681
$D_{10}$= 0.1558 $C_u$= 1.71 $C_c$= 0.96

**Classification**

USCS= SP AASHTO=

**Remarks**

**Location:** GHA-1
**Sample Number:** 3 **Depth:** 2.5' **Date:** 2/24/20222

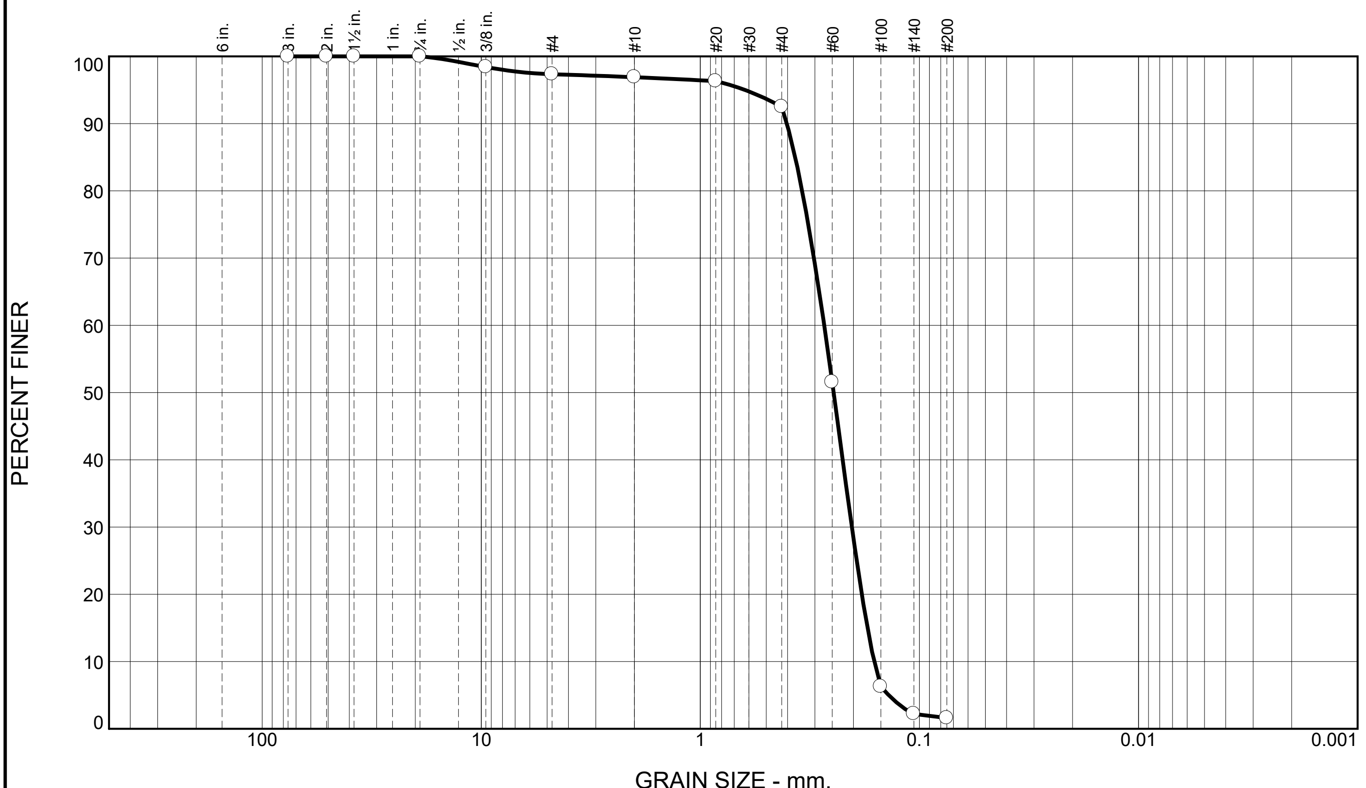
A3GEO

**Client:** Tenderloin Neighborhood Development Corporation
**Project:** 2550 Irving St, Design Phase

**Project No:** 1146-4B **Figure**

**Tested By:** RES **Checked By:** DKM

# Particle Size Distribution Report




| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 2.7 | 0.4 | 4.4 | 90.9 | 1.6 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3.00 | 100.0 | | |
| 2.00 | 100.0 | | |
| 1.5 | 100.0 | | |
| 0.75 | 100.0 | | |
| 0.375 | 98.4 | | |
| #4 | 97.3 | | |
| #10 | 96.9 | | |
| #20 | 96.3 | | |
| #40 | 92.5 | | |
| #60 | 51.5 | | |
| #100 | 6.3 | | |
| #140 | 2.2 | | |
| #200 | 1.6 | | |

* (no specification provided)

**Material Description**

Poorly-Graded Sand, light brown, fine-medium sand, trace fines, non-plastic, moist

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{90}$= 0.4026 $D_{85}$= 0.3689 $D_{60}$= 0.2720
$D_{50}$= 0.2463 $D_{30}$= 0.2035 $D_{15}$= 0.1726
$D_{10}$= 0.1608 $C_u$= 1.69 $C_c$= 0.95

**Classification**

USCS= SP AASHTO=

**Remarks**

1-in. gravel size concrete debris particle (24.11g) removed from sample and not included in test mass

**Location:** GHA-2
**Sample Number:** 4 **Depth:** 4.5' **Date:** 2/24/2022

A3GEO

**Client:** Tenderloin Neighborhood Development Corporation
**Project:** 2550 Irving St, Design Phase

**Project No:** 1146-4B **Figure**

**Tested By:** RES **Checked By:** DKM

# Corrosivity Test Summary

COOPER TESTING LABORATORY

**CTL #** 748-053 **Date:** 3/4/2022 **Tested By:** PJ **Checked:** PJ
**Client:** A3GEO **Project:** 2550 Irving St - Design Phase **Proj. No:** 1146-4B
**Remarks:**

| Sample Location or ID | | | Resistivity @ 15.5 °C (Ohm-cm) | | | Chloride | Sulfate | | pH | ORP | Moisture | Soil Visual Description |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Boring | Sample, No. | Depth, ft. | As Rec. | Minimum | Saturated | mg/kg | mg/kg | % | | (Redox) | At Test | |
| | | | | | | Dry Wt. | Dry Wt. | Dry Wt. | | mv | % | |
| | | | ASTM G57 | Cal 643 | ASTM G57 | Cal 422-mod. | Cal 417-mod. | Cal 417-mod. | Cal 643 | SM 2580B | ASTM D2216 | |
| GB-2 | 2 | 2.5 | - | 5720 | - | 3 | 397 | 0.0397 | 7.0 | - | 2.6 | Brown SAND w/ Silt |

# APPENDIX F
# Drill Spoils Analytical Laboratory Testing Data

Enthalpy Analytical
931 West Barkley Ave
Orange, CA 92868
(714) 771-6900

enthalpy.com

| | |
|---|---|
| Lab Job Number: | 459353 |
| Report Level: | II |
| Report Date: | 03/15/2022 |

**Analytical Report** *prepared for:*

Dillon Braud
A3GEO Inc.
821 Bancroft Way
Berkeley, CA 94710

Project: 1146-4B - 2550 Irving St.

*Authorized for release by:*

Miguel Gamboa, Project Coordinator
miguel.gamboa@enthalpy.com

This data package has been reviewed for technical correctness and completeness. Release of this data has been authorized by the Laboratory Manager or the Manager's designee, as verified by the above signature which applies to this PDF file as well as any associated electronic data deliverable files. The results contained in this report meet all requirements of NELAP and pertain only to those samples which were submitted for analysis. This report may be reproduced only in its entirety.

CA ELAP# 1338, NELAP# 4038, SCAQMD LAP# 18LA0518, LACSD ID# 10105

ENTHALPY ANALYTICAL

# Sample Summary

| | | | |
|---|---|---|---|
| Dillon Braud | | Lab Job #: | 459353 |
| A3GEO Inc. | | Project No: | 1146-4B |
| 821 Bancroft Way | | Location: | 2550 Irving St. |
| Berkeley, CA 94710 | | Date Received: | 03/07/22 |

| Sample ID | Lab ID | Collected | Matrix |
|---|---|---|---|
| S #1 | 459353-001 | 03/07/22 12:00 | Soil |

# Case Narrative

| | | | |
|---|---|---|---|
| A3GEO Inc. | | Lab Job Number: | 459353 |
| 821 Bancroft Way | | Project No: | 1146-4B |
| Berkeley, CA 94710 | | Location: | 2550 Irving St. |
| Dillon Braud | | Date Received: | 03/07/22 |

This data package contains sample and QC results for one soil sample, requested for the above referenced project on 03/07/22. The sample was received cold and intact.

**TPH-Extractables by GC (EPA 8015M):**

No analytical problems were encountered.

**Volatile Organics by GC/MS (EPA 8260B):**

No analytical problems were encountered.

**Metals (EPA 6020 and EPA 7471A):**

- High responses were observed for silver in the CCV analyzed 03/11/22 20:36 and the CCV analyzed 03/11/22 21:39; affected data was qualified with "b".
- Low recoveries were observed for antimony in the MS/MSD of S #1 (lab # 459353-001); the LCS was within limits. High RPD was also observed for antimony; this analyte was not detected at or above the RL in the associated sample.
- No other analytical problems were encountered.

# Analysis Results for 459353

| | |
|---|---|
| Dillon Braud | Lab Job #: 459353 |
| A3GEO Inc. | Project No: 1146-4B |
| 821 Bancroft Way | Location: 2550 Irving St. |
| Berkeley, CA 94710 | Date Received: 03/07/22 |

**Sample ID: S #1** **Lab ID: 459353-001** **Collected: 03/07/22 12:00**
**Matrix: Soil**

| 459353-001 Analyte | Result | Qual | Units | RL | DF | Batch | Prepared | Analyzed | Chemist |
|---|---|---|---|---|---|---|---|---|---|
| Method: EPA 6020<br>Prep Method: EPA 3050B | | | | | | | | | |
| Antimony | ND | | mg/Kg | 0.91 | 0.91 | 285235 | 03/09/22 | 03/11/22 | CMQ |
| Arsenic | **3.2** | | mg/Kg | 0.91 | 0.91 | 285235 | 03/09/22 | 03/11/22 | CMQ |
| Barium | **8.0** | | mg/Kg | 0.91 | 0.91 | 285235 | 03/09/22 | 03/11/22 | CMQ |
| Beryllium | ND | | mg/Kg | 0.91 | 0.91 | 285235 | 03/09/22 | 03/11/22 | CMQ |
| Cadmium | ND | | mg/Kg | 0.45 | 0.91 | 285235 | 03/09/22 | 03/11/22 | CMQ |
| Chromium | **31** | | mg/Kg | 0.91 | 0.91 | 285235 | 03/09/22 | 03/11/22 | CMQ |
| Cobalt | **4.8** | | mg/Kg | 0.91 | 0.91 | 285235 | 03/09/22 | 03/11/22 | CMQ |
| Copper | **2.9** | | mg/Kg | 0.91 | 0.91 | 285235 | 03/09/22 | 03/11/22 | CMQ |
| Lead | **1.7** | | mg/Kg | 0.45 | 0.91 | 285235 | 03/09/22 | 03/11/22 | CMQ |
| Molybdenum | ND | | mg/Kg | 0.91 | 0.91 | 285235 | 03/09/22 | 03/11/22 | CMQ |
| Nickel | **24** | | mg/Kg | 0.91 | 0.91 | 285235 | 03/09/22 | 03/11/22 | CMQ |
| Selenium | ND | | mg/Kg | 1.8 | 0.91 | 285235 | 03/09/22 | 03/11/22 | CMQ |
| Silver | ND | | mg/Kg | 0.45 | 0.91 | 285235 | 03/09/22 | 03/11/22 | CMQ |
| Thallium | ND | | mg/Kg | 0.91 | 0.91 | 285235 | 03/09/22 | 03/11/22 | CMQ |
| Vanadium | **20** | | mg/Kg | 1.8 | 0.91 | 285235 | 03/09/22 | 03/11/22 | CMQ |
| Zinc | **15** | | mg/Kg | 4.5 | 0.91 | 285235 | 03/09/22 | 03/11/22 | CMQ |
| Method: EPA 7471A<br>Prep Method: METHOD | | | | | | | | | |
| Mercury | ND | | mg/Kg | 0.14 | 1 | 285241 | 03/09/22 | 03/10/22 | SBW |
| Method: EPA 8015M<br>Prep Method: EPA 3580 | | | | | | | | | |
| GRO C8-C10 | ND | | mg/Kg | 10 | 1 | 285182 | 03/09/22 | 03/11/22 | MES |
| DRO C10-C28 | ND | | mg/Kg | 10 | 1 | 285182 | 03/09/22 | 03/11/22 | MES |
| ORO C28-C44 | ND | | mg/Kg | 20 | 1 | 285182 | 03/09/22 | 03/11/22 | MES |
| **Surrogates** | | | | **Limits** | | | | | |
| n-Triacontane | 122% | | %REC | 70-130 | 1 | 285182 | 03/09/22 | 03/11/22 | MES |
| Method: EPA 8260B<br>Prep Method: EPA 5030B | | | | | | | | | |
| 3-Chloropropene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Freon 12 | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Chloromethane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Vinyl Chloride | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Bromomethane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Chloroethane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Trichlorofluoromethane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |



Results for any subcontracted analyses are not included in this section.

ENTHALPY ANALYTICAL

# Analysis Results for 459353

| 459353-001 Analyte | Result | Qual | Units | RL | DF | Batch | Prepared | Analyzed | Chemist |
|---|---|---|---|---|---|---|---|---|---|
| Acetone | ND | | ug/Kg | 100 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Freon 113 | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,1-Dichloroethene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Methylene Chloride | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| MTBE | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| trans-1,2-Dichloroethene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,1-Dichloroethane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 2-Butanone | ND | | ug/Kg | 100 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| cis-1,2-Dichloroethene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 2,2-Dichloropropane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Chloroform | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Bromochloromethane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,1,1-Trichloroethane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,1-Dichloropropene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Carbon Tetrachloride | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,2-Dichloroethane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Benzene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Trichloroethene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,2-Dichloropropane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Bromodichloromethane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Dibromomethane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 4-Methyl-2-Pentanone | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| cis-1,3-Dichloropropene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Toluene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| trans-1,3-Dichloropropene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,1,2-Trichloroethane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,3-Dichloropropane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Tetrachloroethene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Dibromochloromethane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,2-Dibromoethane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Chlorobenzene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,1,1,2-Tetrachloroethane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Ethylbenzene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| m,p-Xylenes | ND | | ug/Kg | 10 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| o-Xylene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Styrene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Bromoform | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Isopropylbenzene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,1,2,2-Tetrachloroethane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,2,3-Trichloropropane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Propylbenzene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Bromobenzene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,3,5-Trimethylbenzene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 2-Chlorotoluene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 4-Chlorotoluene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| tert-Butylbenzene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |



Results for any subcontracted analyses are not included in this section.

## Analysis Results for 459353

| 459353-001 Analyte | Result | Qual | Units | RL | DF | Batch | Prepared | Analyzed | Chemist |
|---|---|---|---|---|---|---|---|---|---|
| 1,2,4-Trimethylbenzene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| sec-Butylbenzene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| para-Isopropyl Toluene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,3-Dichlorobenzene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,4-Dichlorobenzene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| n-Butylbenzene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,2-Dichlorobenzene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,2-Dibromo-3-Chloropropane | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,2,4-Trichlorobenzene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Hexachlorobutadiene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Naphthalene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,2,3-Trichlorobenzene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| cis-1,4-Dichloro-2-butene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| trans-1,4-Dichloro-2-butene | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Xylene (total) | ND | | ug/Kg | 5.0 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| **Surrogates** | | | | **Limits** | | | | | |
| Dibromofluoromethane | 100% | | %REC | 70-145 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| 1,2-Dichloroethane-d4 | 97% | | %REC | 70-145 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Toluene-d8 | 103% | | %REC | 70-145 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |
| Bromofluorobenzene | 95% | | %REC | 70-145 | 1 | 285111 | 03/08/22 | 03/08/22 | RAO |

ND Not Detected



Results for any subcontracted analyses are not included in this section.

# Batch QC

| Type: Blank | Lab ID: QC976564 | Batch: 285235 |
|---|---|---|
| Matrix: Soil | Method: EPA 6020 | Prep Method: EPA 3050B |

| QC976564 Analyte | Result | Qual | Units | RL | Prepared | Analyzed |
|---|---|---|---|---|---|---|
| Antimony | ND | | mg/Kg | 1.0 | 03/09/22 | 03/11/22 |
| Arsenic | ND | | mg/Kg | 1.0 | 03/09/22 | 03/11/22 |
| Barium | ND | | mg/Kg | 1.0 | 03/09/22 | 03/11/22 |
| Beryllium | ND | | mg/Kg | 1.0 | 03/09/22 | 03/11/22 |
| Cadmium | ND | | mg/Kg | 0.50 | 03/09/22 | 03/11/22 |
| Chromium | ND | | mg/Kg | 1.0 | 03/09/22 | 03/11/22 |
| Cobalt | ND | | mg/Kg | 1.0 | 03/09/22 | 03/11/22 |
| Copper | ND | | mg/Kg | 1.0 | 03/09/22 | 03/11/22 |
| Lead | ND | | mg/Kg | 0.50 | 03/09/22 | 03/11/22 |
| Molybdenum | ND | | mg/Kg | 1.0 | 03/09/22 | 03/11/22 |
| Nickel | ND | | mg/Kg | 1.0 | 03/09/22 | 03/11/22 |
| Selenium | ND | | mg/Kg | 2.0 | 03/09/22 | 03/11/22 |
| Silver | ND | | mg/Kg | 0.50 | 03/09/22 | 03/11/22 |
| Thallium | ND | | mg/Kg | 1.0 | 03/09/22 | 03/11/22 |
| Vanadium | ND | | mg/Kg | 2.0 | 03/09/22 | 03/11/22 |
| Zinc | ND | | mg/Kg | 5.0 | 03/09/22 | 03/11/22 |

| Type: Lab Control Sample | Lab ID: QC976565 | Batch: 285235 |
|---|---|---|
| Matrix: Soil | Method: EPA 6020 | Prep Method: EPA 3050B |

| QC976565 Analyte | Result | Spiked | Units | Recovery | Qual | Limits |
|---|---|---|---|---|---|---|
| Antimony | 53.74 | 50.00 | mg/Kg | 107% | | 80-120 |
| Arsenic | 50.97 | 50.00 | mg/Kg | 102% | | 80-120 |
| Barium | 50.63 | 50.00 | mg/Kg | 101% | | 80-120 |
| Beryllium | 50.82 | 50.00 | mg/Kg | 102% | | 80-120 |
| Cadmium | 51.94 | 50.00 | mg/Kg | 104% | | 80-120 |
| Chromium | 51.07 | 50.00 | mg/Kg | 102% | | 80-120 |
| Cobalt | 53.11 | 50.00 | mg/Kg | 106% | | 80-120 |
| Copper | 51.60 | 50.00 | mg/Kg | 103% | | 80-120 |
| Lead | 51.95 | 50.00 | mg/Kg | 104% | | 80-120 |
| Molybdenum | 57.23 | 50.00 | mg/Kg | 114% | | 80-120 |
| Nickel | 52.38 | 50.00 | mg/Kg | 105% | | 80-120 |
| Selenium | 50.13 | 50.00 | mg/Kg | 100% | | 80-120 |
| Silver | 28.69 | 25.00 | mg/Kg | 115% | b | 80-120 |
| Thallium | 51.81 | 50.00 | mg/Kg | 104% | | 80-120 |
| Vanadium | 51.62 | 50.00 | mg/Kg | 103% | | 80-120 |
| Zinc | 51.85 | 50.00 | mg/Kg | 104% | | 80-120 |

# Batch QC

| Type: | Matrix Spike | Lab ID: | QC976566 | Batch: | 285235 |
|---|---|---|---|---|---|
| Matrix (Source ID): | Soil (459353-001) | Method: | EPA 6020 | Prep Method: | EPA 3050B |

| QC976566 Analyte | Result | Source Sample Result | Spiked | Units | Recovery | Qual | Limits | DF |
|---|---|---|---|---|---|---|---|---|
| Antimony | 14.24 | 0.07176 | 45.45 | mg/Kg | 31% | * | 75-125 | 0.91 |
| Arsenic | 49.56 | 3.237 | 45.45 | mg/Kg | 102% | | 75-125 | 0.91 |
| Barium | 57.21 | 7.971 | 45.45 | mg/Kg | 108% | | 75-125 | 0.91 |
| Beryllium | 47.44 | 0.1443 | 45.45 | mg/Kg | 104% | | 75-125 | 0.91 |
| Cadmium | 47.66 | 0.03850 | 45.45 | mg/Kg | 105% | | 75-125 | 0.91 |
| Chromium | 71.80 | 31.16 | 45.45 | mg/Kg | 89% | | 75-125 | 0.91 |
| Cobalt | 53.09 | 4.835 | 45.45 | mg/Kg | 106% | | 75-125 | 0.91 |
| Copper | 50.89 | 2.883 | 45.45 | mg/Kg | 106% | | 75-125 | 0.91 |
| Lead | 48.71 | 1.659 | 45.45 | mg/Kg | 104% | | 75-125 | 0.91 |
| Molybdenum | 47.03 | 0.08136 | 45.45 | mg/Kg | 103% | | 75-125 | 0.91 |
| Nickel | 71.81 | 23.72 | 45.45 | mg/Kg | 106% | | 75-125 | 0.91 |
| Selenium | 46.32 | 0.1442 | 45.45 | mg/Kg | 102% | | 75-125 | 0.91 |
| Silver | 25.38 | 0.009918 | 22.73 | mg/Kg | 112% | b | 75-125 | 0.91 |
| Thallium | 47.27 | 0.03941 | 45.45 | mg/Kg | 104% | | 75-125 | 0.91 |
| Vanadium | 69.33 | 20.46 | 45.45 | mg/Kg | 108% | | 75-125 | 0.91 |
| Zinc | 63.24 | 14.79 | 45.45 | mg/Kg | 107% | | 75-125 | 0.91 |

| Type: | Matrix Spike Duplicate | Lab ID: | QC976567 | Batch: | 285235 |
|---|---|---|---|---|---|
| Matrix (Source ID): | Soil (459353-001) | Method: | EPA 6020 | Prep Method: | EPA 3050B |

| QC976567 Analyte | Result | Source Sample Result | Spiked | Units | Recovery | Qual | Limits | RPD | RPD Lim | DF |
|---|---|---|---|---|---|---|---|---|---|---|
| Antimony | 18.91 | 0.07176 | 47.62 | mg/Kg | 40% | * | 75-125 | 24* | 20 | 0.95 |
| Arsenic | 51.84 | 3.237 | 47.62 | mg/Kg | 102% | | 75-125 | 0 | 20 | 0.95 |
| Barium | 60.53 | 7.971 | 47.62 | mg/Kg | 110% | | 75-125 | 2 | 20 | 0.95 |
| Beryllium | 49.83 | 0.1443 | 47.62 | mg/Kg | 104% | | 75-125 | 0 | 20 | 0.95 |
| Cadmium | 50.68 | 0.03850 | 47.62 | mg/Kg | 106% | | 75-125 | 2 | 20 | 0.95 |
| Chromium | 77.06 | 31.16 | 47.62 | mg/Kg | 96% | | 75-125 | 4 | 20 | 0.95 |
| Cobalt | 56.16 | 4.835 | 47.62 | mg/Kg | 108% | | 75-125 | 1 | 20 | 0.95 |
| Copper | 52.75 | 2.883 | 47.62 | mg/Kg | 105% | | 75-125 | 1 | 20 | 0.95 |
| Lead | 51.85 | 1.659 | 47.62 | mg/Kg | 105% | | 75-125 | 2 | 20 | 0.95 |
| Molybdenum | 50.07 | 0.08136 | 47.62 | mg/Kg | 105% | | 75-125 | 2 | 20 | 0.95 |
| Nickel | 74.31 | 23.72 | 47.62 | mg/Kg | 106% | | 75-125 | 0 | 20 | 0.95 |
| Selenium | 48.53 | 0.1442 | 47.62 | mg/Kg | 102% | | 75-125 | 0 | 20 | 0.95 |
| Silver | 26.61 | 0.009918 | 23.81 | mg/Kg | 112% | b | 75-125 | 0 | 20 | 0.95 |
| Thallium | 50.28 | 0.03941 | 47.62 | mg/Kg | 106% | | 75-125 | 2 | 20 | 0.95 |
| Vanadium | 71.94 | 20.46 | 47.62 | mg/Kg | 108% | | 75-125 | 0 | 20 | 0.95 |
| Zinc | 64.27 | 14.79 | 47.62 | mg/Kg | 104% | | 75-125 | 2 | 20 | 0.95 |

# Batch QC

| | | | | | |
|---|---|---|---|---|---|
| Type: | Blank | Lab ID: | QC976591 | Batch: | 285241 |
| Matrix: | Miscell. | Method: | EPA 7471A | Prep Method: | METHOD |

| QC976591 Analyte | Result | Qual | Units | RL | Prepared | Analyzed |
|---|---|---|---|---|---|---|
| Mercury | ND | | mg/Kg | 0.14 | 03/09/22 | 03/10/22 |

| | | | | | |
|---|---|---|---|---|---|
| Type: | Lab Control Sample | Lab ID: | QC976592 | Batch: | 285241 |
| Matrix: | Miscell. | Method: | EPA 7471A | Prep Method: | METHOD |

| QC976592 Analyte | Result | Spiked | Units | Recovery | Qual | Limits |
|---|---|---|---|---|---|---|
| Mercury | 0.8996 | 0.8333 | mg/Kg | 108% | | 80-120 |

| | | | | | |
|---|---|---|---|---|---|
| Type: | Matrix Spike | Lab ID: | QC976593 | Batch: | 285241 |
| Matrix (Source ID): | Soil (459332-005) | Method: | EPA 7471A | Prep Method: | METHOD |

| QC976593 Analyte | Result | Source Sample Result | Spiked | Units | Recovery | Qual | Limits | DF |
|---|---|---|---|---|---|---|---|---|
| Mercury | 1.002 | ND | 0.9615 | mg/Kg | 104% | | 75-125 | 1.2 |

| | | | | | |
|---|---|---|---|---|---|
| Type: | Matrix Spike Duplicate | Lab ID: | QC976594 | Batch: | 285241 |
| Matrix (Source ID): | Soil (459332-005) | Method: | EPA 7471A | Prep Method: | METHOD |

| QC976594 Analyte | Result | Source Sample Result | Spiked | Units | Recovery | Qual | Limits | RPD | RPD Lim | DF |
|---|---|---|---|---|---|---|---|---|---|---|
| Mercury | 0.9415 | ND | 0.8929 | mg/Kg | 105% | | 75-125 | 1 | 20 | 1.1 |

| | | | | | |
|---|---|---|---|---|---|
| Type: | Blank | Lab ID: | QC976392 | Batch: | 285182 |
| Matrix: | Soil | Method: | EPA 8015M | Prep Method: | EPA 3580 |

| QC976392 Analyte | Result | Qual | Units | RL | Prepared | Analyzed |
|---|---|---|---|---|---|---|
| GRO C8-C10 | ND | | mg/Kg | 10 | 03/08/22 | 03/09/22 |
| DRO C10-C28 | ND | | mg/Kg | 10 | 03/08/22 | 03/09/22 |
| ORO C28-C44 | ND | | mg/Kg | 20 | 03/08/22 | 03/09/22 |
| **Surrogates** | | | | **Limits** | | |
| n-Triacontane | 110% | | %REC | 70-130 | 03/08/22 | 03/09/22 |

| | | | | | |
|---|---|---|---|---|---|
| Type: | Lab Control Sample | Lab ID: | QC976393 | Batch: | 285182 |
| Matrix: | Soil | Method: | EPA 8015M | Prep Method: | EPA 3580 |

| QC976393 Analyte | Result | Spiked | Units | Recovery | Qual | Limits |
|---|---|---|---|---|---|---|
| Diesel C10-C28 | 264.0 | 250.0 | mg/Kg | 106% | | 76-122 |
| **Surrogates** | | | | | | |
| n-Triacontane | 11.28 | 10.00 | mg/Kg | 113% | | 70-130 |

# Batch QC

| | | | | | |
|---|---|---|---|---|---|
| **Type:** | **Matrix Spike** | **Lab ID:** | **QC976394** | **Batch:** | **285182** |
| **Matrix (Source ID):** | **Soil (459259-008)** | **Method:** | **EPA 8015M** | **Prep Method:** | **EPA 3580** |

| QC976394 Analyte | Result | Source Sample Result | Spiked | Units | Recovery | Qual | Limits | DF |
|---|---|---|---|---|---|---|---|---|
| Diesel C10-C28 | 212.6 | 16.79 | 250.0 | mg/Kg | 78% | | 62-126 | 1 |
| **Surrogates** | | | | | | | | |
| n-Triacontane | 11.71 | | 10.00 | mg/Kg | 117% | | 70-130 | 1 |

| | | | | | |
|---|---|---|---|---|---|
| **Type:** | **Matrix Spike Duplicate** | **Lab ID:** | **QC976395** | **Batch:** | **285182** |
| **Matrix (Source ID):** | **Soil (459259-008)** | **Method:** | **EPA 8015M** | **Prep Method:** | **EPA 3580** |

| QC976395 Analyte | Result | Source Sample Result | Spiked | Units | Recovery | Qual | Limits | RPD | RPD Lim | DF |
|---|---|---|---|---|---|---|---|---|---|---|
| Diesel C10-C28 | 239.9 | 16.79 | 250.0 | mg/Kg | 89% | | 62-126 | 12 | 35 | 1 |
| **Surrogates** | | | | | | | | | | |
| n-Triacontane | 11.81 | | 10.00 | mg/Kg | 118% | | 70-130 | | | 1 |

# Batch QC

| Type: | Blank | Lab ID: | QC976201 | Batch: | 285111 |
|---|---|---|---|---|---|
| Matrix: | Soil | Method: | EPA 8260B | Prep Method: | EPA 5030B |

| QC976201 Analyte | Result | Qual | Units | RL | Prepared | Analyzed |
|---|---|---|---|---|---|---|
| 3-Chloropropene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Freon 12 | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Chloromethane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Vinyl Chloride | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Bromomethane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Chloroethane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Trichlorofluoromethane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Acetone | ND | | ug/Kg | 100 | 03/08/22 | 03/08/22 |
| Freon 113 | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,1-Dichloroethene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Methylene Chloride | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| MTBE | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| trans-1,2-Dichloroethene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,1-Dichloroethane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 2-Butanone | ND | | ug/Kg | 100 | 03/08/22 | 03/08/22 |
| cis-1,2-Dichloroethene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 2,2-Dichloropropane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Chloroform | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Bromochloromethane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,1,1-Trichloroethane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,1-Dichloropropene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Carbon Tetrachloride | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,2-Dichloroethane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Benzene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Trichloroethene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,2-Dichloropropane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Bromodichloromethane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Dibromomethane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 4-Methyl-2-Pentanone | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| cis-1,3-Dichloropropene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Toluene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| trans-1,3-Dichloropropene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,1,2-Trichloroethane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,3-Dichloropropane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Tetrachloroethene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Dibromochloromethane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,2-Dibromoethane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Chlorobenzene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,1,1,2-Tetrachloroethane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Ethylbenzene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| m,p-Xylenes | ND | | ug/Kg | 10 | 03/08/22 | 03/08/22 |
| o-Xylene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |

ENTHALPY ANALYTICAL

## Batch QC

| QC976201 Analyte | Result | Qual | Units | RL | Prepared | Analyzed |
|---|---|---|---|---|---|---|
| Styrene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Bromoform | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Isopropylbenzene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,1,2,2-Tetrachloroethane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,2,3-Trichloropropane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Propylbenzene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Bromobenzene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,3,5-Trimethylbenzene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 2-Chlorotoluene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 4-Chlorotoluene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| tert-Butylbenzene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,2,4-Trimethylbenzene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| sec-Butylbenzene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| para-Isopropyl Toluene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,3-Dichlorobenzene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,4-Dichlorobenzene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| n-Butylbenzene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,2-Dichlorobenzene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,2-Dibromo-3-Chloropropane | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,2,4-Trichlorobenzene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Hexachlorobutadiene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Naphthalene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| 1,2,3-Trichlorobenzene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| cis-1,4-Dichloro-2-butene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| trans-1,4-Dichloro-2-butene | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| Xylene (total) | ND | | ug/Kg | 5.0 | 03/08/22 | 03/08/22 |
| **Surrogates** | | | | **Limits** | | |
| Dibromofluoromethane | 98% | | %REC | 70-130 | 03/08/22 | 03/08/22 |
| 1,2-Dichloroethane-d4 | 95% | | %REC | 70-145 | 03/08/22 | 03/08/22 |
| Toluene-d8 | 109% | | %REC | 70-145 | 03/08/22 | 03/08/22 |
| Bromofluorobenzene | 100% | | %REC | 70-145 | 03/08/22 | 03/08/22 |

ENTHALPY ANALYTICAL

# Batch QC

| Type: | Lab Control Sample | Lab ID: | QC976202 | Batch: | 285111 |
|---|---|---|---|---|---|
| Matrix: | Soil | Method: | EPA 8260B | Prep Method: | EPA 5030B |

| QC976202 Analyte | Result | Spiked | Units | Recovery | Qual | Limits |
|---|---|---|---|---|---|---|
| 1,1-Dichloroethene | 52.37 | 50.00 | ug/Kg | 105% | | 70-131 |
| MTBE | 47.15 | 50.00 | ug/Kg | 94% | | 69-130 |
| Benzene | 51.91 | 50.00 | ug/Kg | 104% | | 70-130 |
| Trichloroethene | 56.38 | 50.00 | ug/Kg | 113% | | 70-130 |
| Toluene | 56.17 | 50.00 | ug/Kg | 112% | | 70-130 |
| Chlorobenzene | 56.33 | 50.00 | ug/Kg | 113% | | 70-130 |
| **Surrogates** | | | | | | |
| Dibromofluoromethane | 51.41 | 50.00 | ug/Kg | 103% | | 70-130 |
| 1,2-Dichloroethane-d4 | 47.44 | 50.00 | ug/Kg | 95% | | 70-145 |
| Toluene-d8 | 54.90 | 50.00 | ug/Kg | 110% | | 70-145 |
| Bromofluorobenzene | 49.03 | 50.00 | ug/Kg | 98% | | 70-145 |

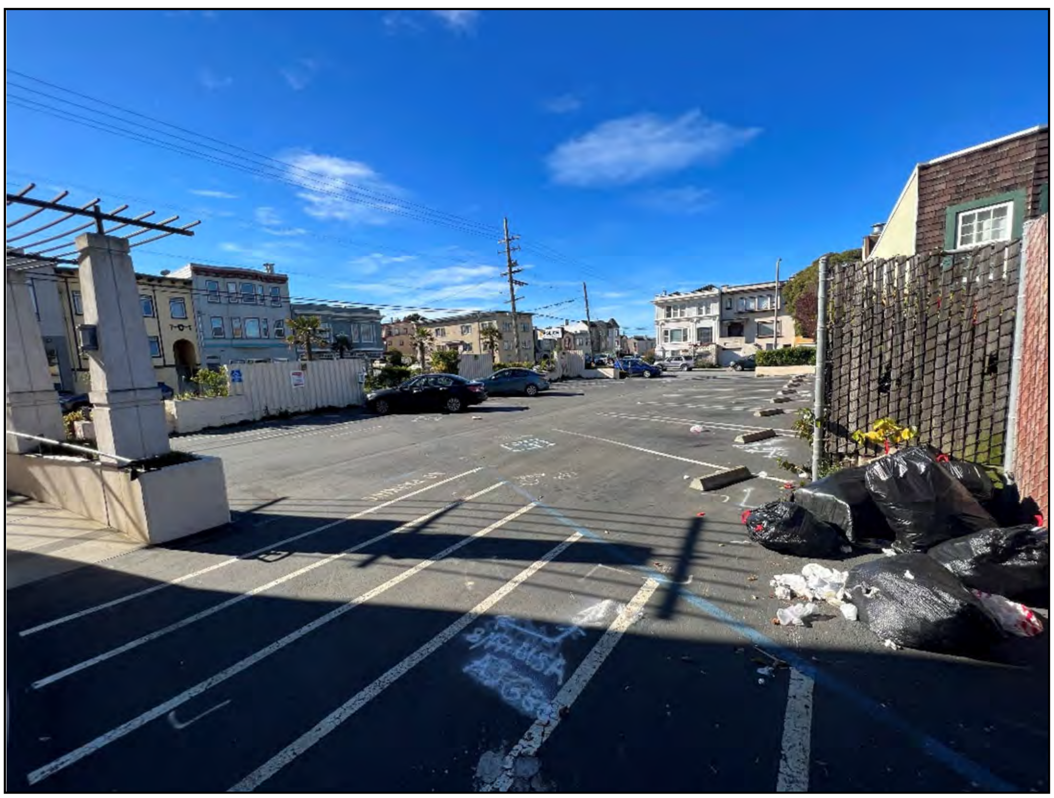
| Type: | Lab Control Sample Duplicate | Lab ID: | QC976203 | Batch: | 285111 |
|---|---|---|---|---|---|
| Matrix: | Soil | Method: | EPA 8260B | Prep Method: | EPA 5030B |

| QC976203 Analyte | Result | Spiked | Units | Recovery | Qual | Limits | RPD | RPD Lim |
|---|---|---|---|---|---|---|---|---|
| 1,1-Dichloroethene | 47.98 | 50.00 | ug/Kg | 96% | | 70-131 | 9 | 33 |
| MTBE | 45.66 | 50.00 | ug/Kg | 91% | | 69-130 | 3 | 30 |
| Benzene | 47.21 | 50.00 | ug/Kg | 94% | | 70-130 | 9 | 30 |
| Trichloroethene | 50.50 | 50.00 | ug/Kg | 101% | | 70-130 | 11 | 30 |
| Toluene | 51.29 | 50.00 | ug/Kg | 103% | | 70-130 | 9 | 30 |
| Chlorobenzene | 50.44 | 50.00 | ug/Kg | 101% | | 70-130 | 11 | 30 |
| **Surrogates** | | | | | | | | |
| Dibromofluoromethane | 49.96 | 50.00 | ug/Kg | 100% | | 70-130 | | |
| 1,2-Dichloroethane-d4 | 46.82 | 50.00 | ug/Kg | 94% | | 70-145 | | |
| Toluene-d8 | 54.27 | 50.00 | ug/Kg | 109% | | 70-145 | | |
| Bromofluorobenzene | 49.91 | 50.00 | ug/Kg | 100% | | 70-145 | | |

| | |
|---|---|
| * | Value is outside QC limits |
| ND | Not Detected |
| b | See narrative |

# APPENDIX G
# Site Photographs

Photograph of the existing parking lot
(taken from the southwest corner of property, looking northeast; 1-22-22)



Photograph of the existing parking lot
(taken from the northeast side of parking lot, looking southwest; 1-22-22)

Photograph of the existing fenced patio area
(taken from the patio area, looking east; 2-17-22)

Photograph of the existing building
(taken from the parking lot, looking east; 2-17-22)


Photograph of the existing building
(taken from the sidewalk at Irving St looking north; 2-17-22)

Photograph of the existing building
(taken the sidewalk at Irving St, looking north; 2-17-22)

Photograph of the existing building
(taken from 26th Ave, looking west; 2-17-22)

Photograph of the existing building
(taken from the sidewalk at 26th Ave, looking southwest; 2-17-22)


Photograph of the existing building
(taken from 26th Ave, looking west; 2-17-22)

Photograph of the existing building
(taken from the sidewalk at 26th Ave, looking southwest; 2-17-22)

Photograph of some cracking at column of entryway (2-17-22)

Photograph showing some cracking at column of entryway (2-17-22)

Photograph of some cracking within concrete entryway (2-17-22)

Photograph of some cracking of exterior column (2-17-22)




**Site Photographs**
**1938 Historical Photo**


GEOTECHNICAL INVESTIGATION REPORT
2550 IRVING STREET, SAN FRANCISCO, CA

**Site Photographs**
**1946 Historical Photo**


GEOTECHNICAL INVESTIGATION REPORT
2550 IRVING STREET, SAN FRANCISCO, CA

**Site Photographs**
**1965 Historical Photo**

# APPENDIX H
# Site-Specific Ground Motion Hazard Analysis Results

## Ground Motion Hazard Analysis

2550 Irving St, San Francisco, CA
TNDC

A3GEO, Inc.
Project # 1146-4B

### Input Parameters

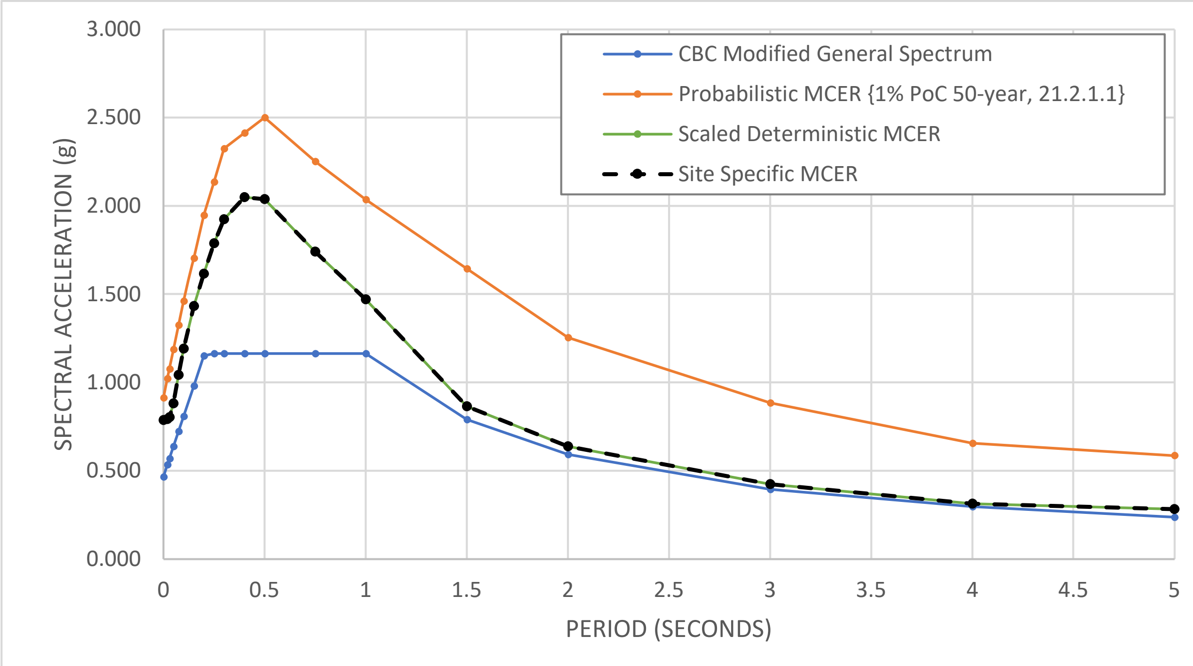
| Parameter | Value |
|---|---|
| Shear Wave Velocity, Vs100 (m/s) | 288 |
| Site Latitude | 37.763464 |
| Site Longitude | -122.485029 |
| Site Class | D |
| Risk Category | II |

### CBC Seismic Parameters

| Parameter | Value |
|---|---|
| SS (mapped short-period spectral acceleration) | 1.745 |
| S1 (mapped one second period spectral acceleration) | 0.711 |
| SMS (site-modified short-period spectral acceleration) | 1.745 |
| SM1 (site-modified one second period spectral acceleration) | 1.7775 |
| SDS (design short-period spectral acceleration) | 1.164 |
| SD1 (design one second period spectral acceleration) | 1.185 |
| Fa (site amplification factor at 0.2 second) | 1 |
| Fv (site amplification factor at 1.0 second) | 2.5 |
| PGA (peak ground acceleration) | 0.752 |
| FPGA (site amplificaiton factor at PGA) | 1.1 |
| PGAM (site modified PGA) | 0.828 |
| TL (long period transition period in seconds) | 12 |
| CRS (mapped value of risk coefficient at short periods) | 0.898 |
| CR1 (mapped value of risk coefficient at period of 1 second) | 0.883 |
| Period at 0.2*Ts | 0.203608247 |
| Period at SD1/SDS | 1.018041237 |

### Calculations:

| Period (Seconds) | CBC Modified General Spectrum | Probabilistic 2% PoE 50-year | Risk Coefficient | Probabilistic MCER {1% PoC 50-year, 21.2.1.1} | Deterministic 84th % | Scaled Deterministic MCER | Site Specific MCER | 2/3 Site Specific MCER | 80% CBC Modified General Spectrum | Design Response Spectrum |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 0.47 | 1.02 | 0.90 | 0.91 | 0.79 | 0.79 | 0.79 | 0.52 | 0.37 | 0.52 |
| 0.02 | 0.53 | 1.14 | 0.90 | 1.02 | 0.79 | 0.79 | 0.79 | 0.53 | 0.43 | 0.53 |
| 0.03 | 0.57 | 1.20 | 0.90 | 1.08 | 0.80 | 0.80 | 0.80 | 0.54 | 0.45 | 0.54 |
| 0.05 | 0.64 | 1.32 | 0.90 | 1.19 | 0.88 | 0.88 | 0.88 | 0.59 | 0.51 | 0.59 |
| 0.075 | 0.72 | 1.47 | 0.90 | 1.32 | 1.04 | 1.04 | 1.04 | 0.69 | 0.58 | 0.69 |
| 0.1 | 0.81 | 1.63 | 0.90 | 1.46 | 1.19 | 1.19 | 1.19 | 0.79 | 0.65 | 0.79 |
| 0.15 | 0.98 | 1.90 | 0.90 | 1.70 | 1.43 | 1.43 | 1.43 | 0.95 | 0.78 | 0.95 |
| 0.2 | 1.15 | 2.17 | 0.90 | 1.95 | 1.62 | 1.62 | 1.62 | 1.08 | 0.92 | 1.08 |
| 0.25 | 1.16 | 2.38 | 0.90 | 2.14 | 1.79 | 1.79 | 1.79 | 1.19 | 0.93 | 1.19 |
| 0.3 | 1.16 | 2.59 | 0.90 | 2.32 | 1.92 | 1.92 | 1.92 | 1.28 | 0.93 | 1.28 |
| 0.4 | 1.16 | 2.70 | 0.89 | 2.41 | 2.05 | 2.05 | 2.05 | 1.37 | 0.93 | 1.37 |
| 0.5 | 1.16 | 2.80 | 0.89 | 2.50 | 2.04 | 2.04 | 2.04 | 1.36 | 0.93 | 1.36 |
| 0.75 | 1.16 | 2.54 | 0.89 | 2.25 | 1.74 | 1.74 | 1.74 | 1.16 | 0.93 | 1.16 |
| 1 | 1.16 | 2.30 | 0.88 | 2.04 | 1.47 | 1.47 | 1.47 | 0.98 | 0.93 | 0.98 |
| 1.5 | 0.79 | 1.86 | 0.88 | 1.64 | 0.86 | 0.86 | 0.86 | 0.58 | 0.63 | 0.63 |
| 2 | 0.59 | 1.42 | 0.88 | 1.25 | 0.64 | 0.64 | 0.64 | 0.43 | 0.47 | 0.47 |
| 3 | 0.40 | 1.00 | 0.88 | 0.88 | 0.42 | 0.42 | 0.42 | 0.28 | 0.32 | 0.32 |
| 4 | 0.30 | 0.74 | 0.88 | 0.66 | 0.31 | 0.31 | 0.31 | 0.21 | 0.24 | 0.24 |
| 5 | 0.24 | 0.66 | 0.88 | 0.59 | 0.28 | 0.28 | 0.28 | 0.19 | 0.19 | 0.19 |




| Period (Seconds) | CBC Modified General Spectrum | Probabilistic $MCE_R$ {1% PoC 50-year, 21.2.1.1} | Scaled Deterministic $MCE_R$ | Site Specific $MCE_R$ |
|---|---|---|---|---|
| 0 | 0.466 | 0.912 | 0.785 | 0.785 |
| 0.02 | 0.534 | 1.022 | 0.792 | 0.792 |
| 0.03 | 0.569 | 1.077 | 0.803 | 0.803 |
| 0.05 | 0.637 | 1.187 | 0.881 | 0.881 |
| 0.075 | 0.723 | 1.324 | 1.042 | 1.042 |
| 0.1 | 0.809 | 1.461 | 1.191 | 1.191 |
| 0.15 | 0.980 | 1.704 | 1.432 | 1.432 |
| 0.2 | 1.152 | 1.946 | 1.615 | 1.615 |
| 0.25 | 1.164 | 2.136 | 1.788 | 1.788 |
| 0.3 | 1.164 | 2.325 | 1.923 | 1.923 |
| 0.4 | 1.164 | 2.412 | 2.049 | 2.049 |
| 0.5 | 1.164 | 2.500 | 2.037 | 2.037 |
| 0.75 | 1.164 | 2.252 | 1.741 | 1.741 |
| 1 | 1.164 | 2.035 | 1.469 | 1.469 |
| 1.5 | 0.790 | 1.644 | 0.864 | 0.864 |
| 2 | 0.593 | 1.254 | 0.638 | 0.638 |
| 3 | 0.395 | 0.885 | 0.423 | 0.423 |
| 4 | 0.296 | 0.655 | 0.312 | 0.312 |
| 5 | 0.237 | 0.586 | 0.282 | 0.282 |

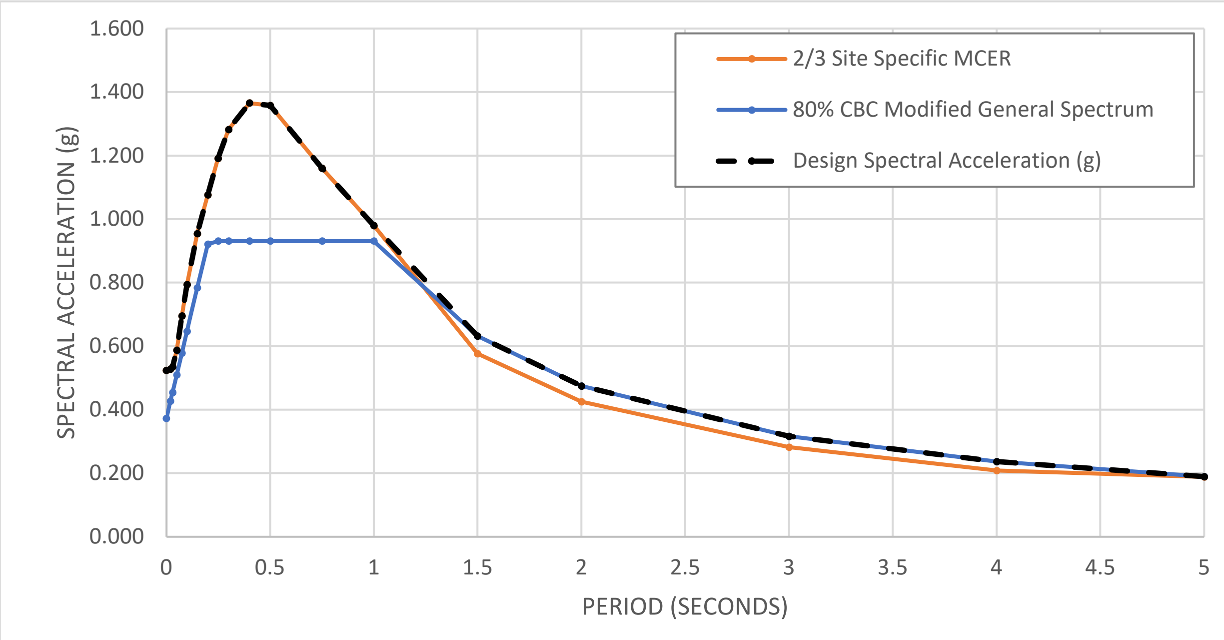
2550 IRVING STREET
TNDC

PROJECT NO. 1146-4B

**SITE SPECIFIC RESPONSE SPECTRUM**

A3GEO

FIGURE H-1




| Design Response Spectrum | |
|---|---|
| Period (Seconds) | Design Spectral Acceleration (g) |
| 0 | 0.524 |
| 0.02 | 0.528 |
| 0.03 | 0.535 |
| 0.05 | 0.588 |
| 0.075 | 0.695 |
| 0.1 | 0.794 |
| 0.15 | 0.955 |
| 0.2 | 1.077 |
| 0.25 | 1.192 |
| 0.3 | 1.282 |
| 0.4 | 1.366 |
| 0.5 | 1.358 |
| 0.75 | 1.160 |
| 1 | 0.980 |
| 1.5 | 0.632 |
| 2 | 0.474 |
| 3 | 0.316 |
| 4 | 0.237 |
| 5 | 0.190 |

| Design Acceleration Parameters | |
|---|---|
| $S_{DS}$ | 1.229 |
| $S_{D1}$ | 0.980 |
| $S_{MS}$ | 1.844 |
| $S_{M1}$ | 1.469 |

| Site Specific $MCE_G$ Peak Ground Acceleration ($PGA_M$) |
|---|
| 0.71 |

2550 IRVING STREET
TNDC

PROJECT NO. 1146-4B

**DESIGN RESPONSE SPECTRUM AND ACCELERATION PARAMETERS**

A3GEO

FIGURE H-2

# APPENDIX I
# Liquefaction and Dynamic Settlement Analysis Results

## TABLE OF CONTENTS

**A3GEO, Inc.**
821 Bancroft Way
Berkeley, CA 94710
(510) 705-1664

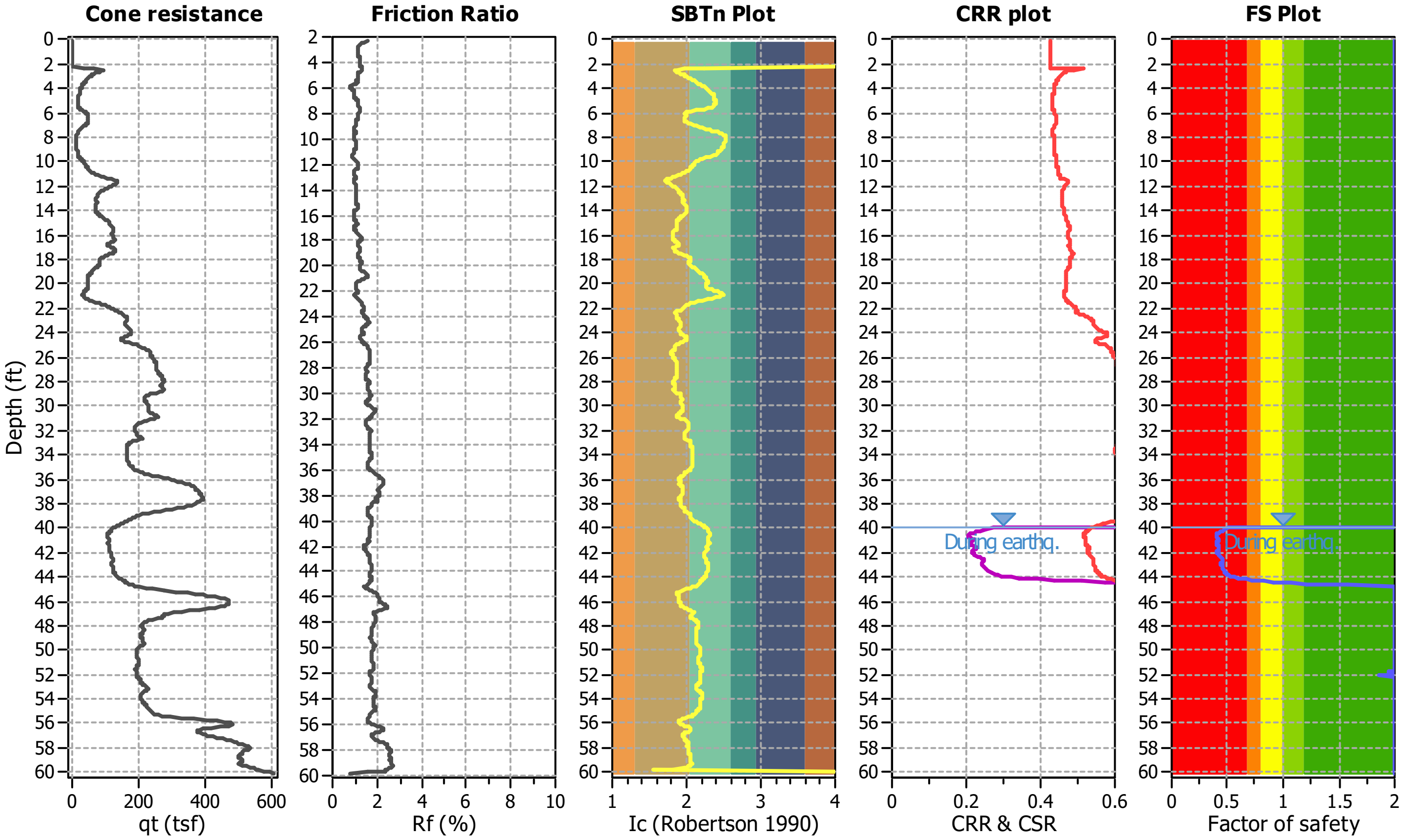
# LIQUEFACTION ANALYSIS REPORT

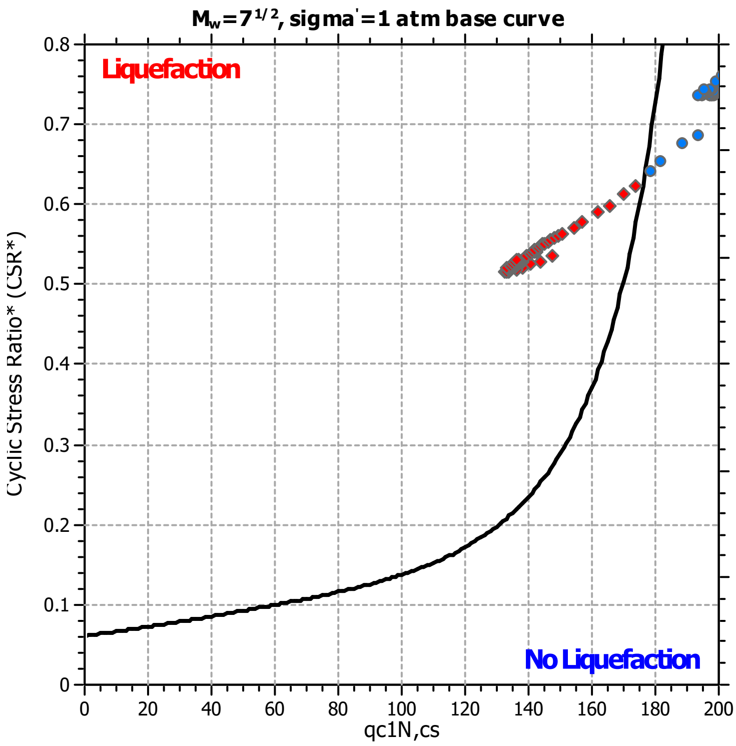
**Project title : 2550 Irving Street** **Location :**

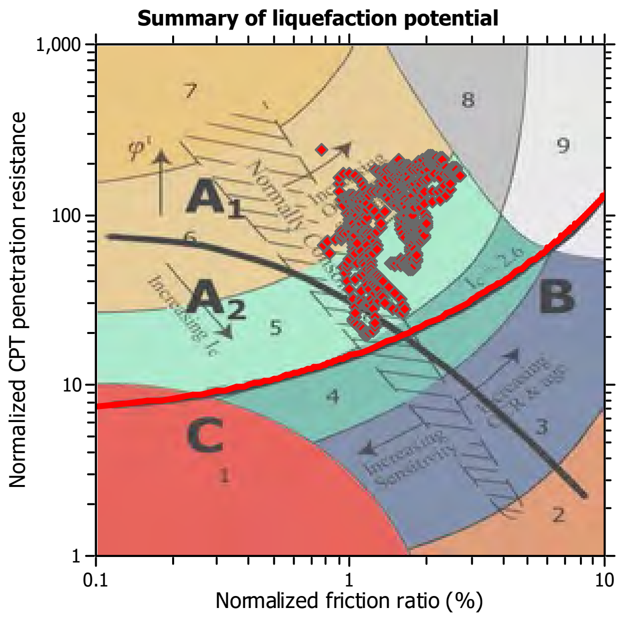
**CPT file : CPT-1**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 60.00 ft | Use fill: | No | Clay like behavior applied: | Sands only |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 40.00 ft | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 8.00 | Ic cut-off value: | 2.60 | Trans. detect. applied: | No | MSF method: | Method based |
| Peak ground acceleration: | 0.71 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |

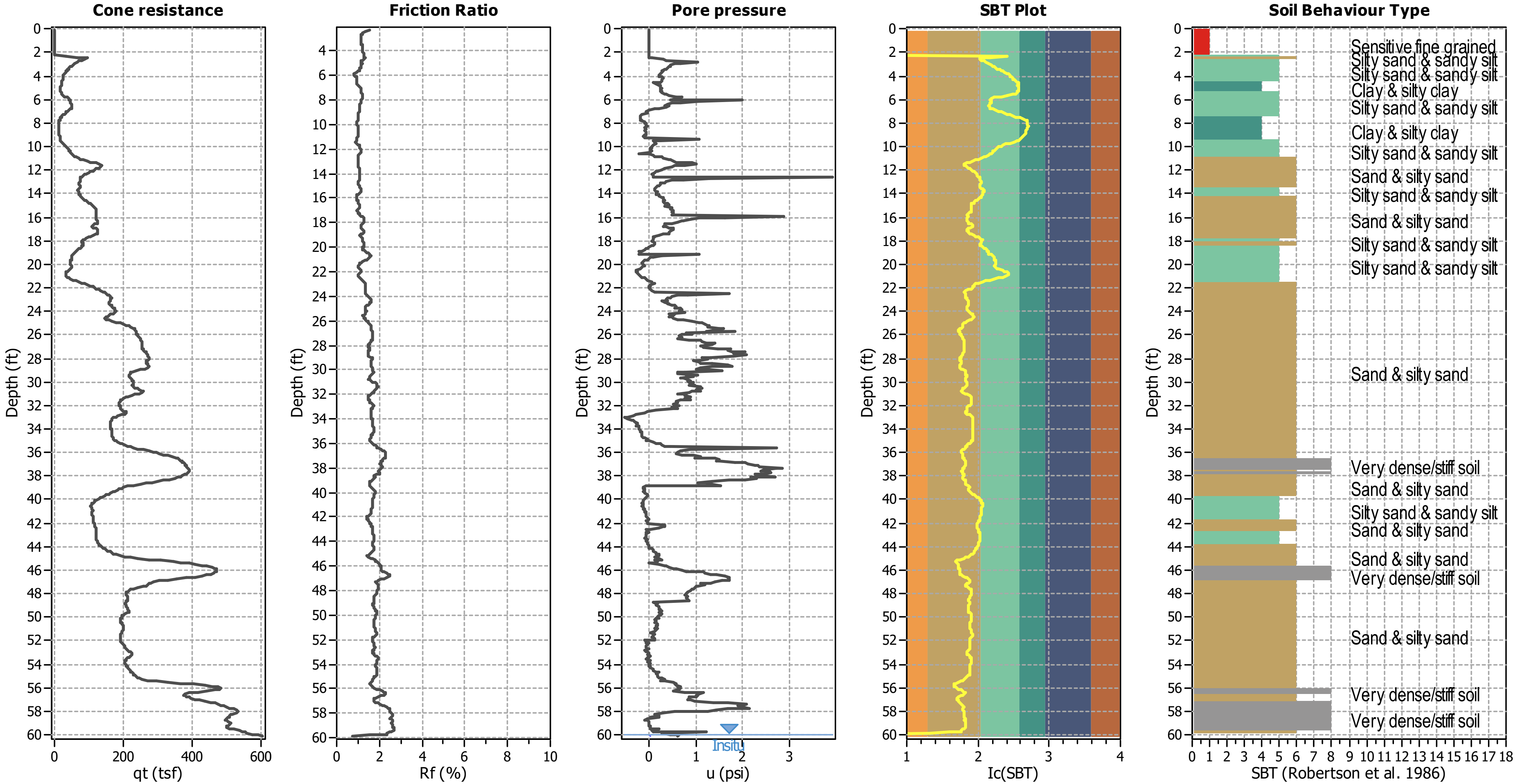



Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

## CPT basic interpretation plots



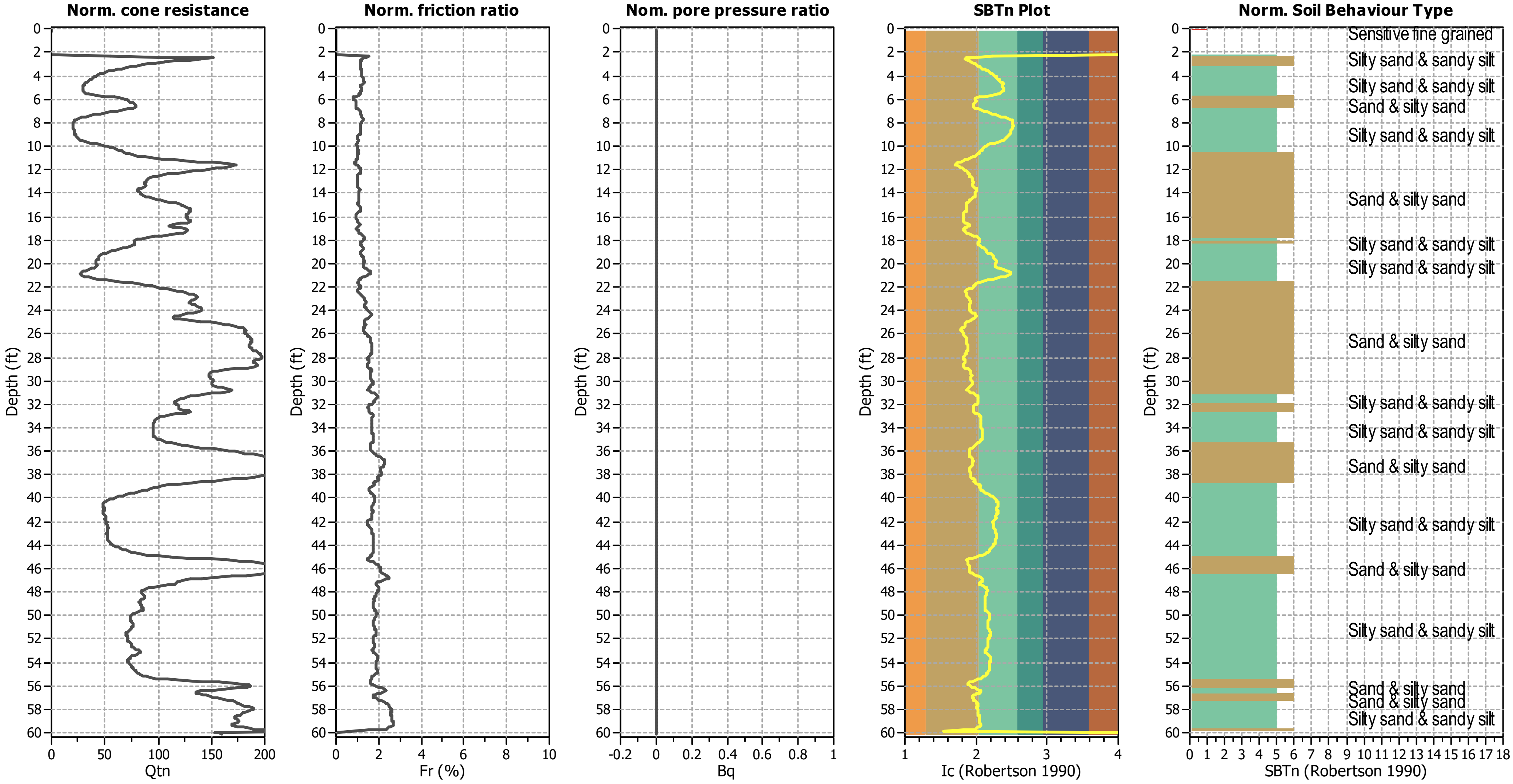

**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | Depth to GWT (erthq.): | 40.00 ft | Fill weight: | N/A |
| Fines correction method: | B&I (2014) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 8.00 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.71 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 60.00 ft | Fill height: | N/A | Limit depth: | N/A |

**SBT legend**

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

# CPT basic interpretation plots (normalized)



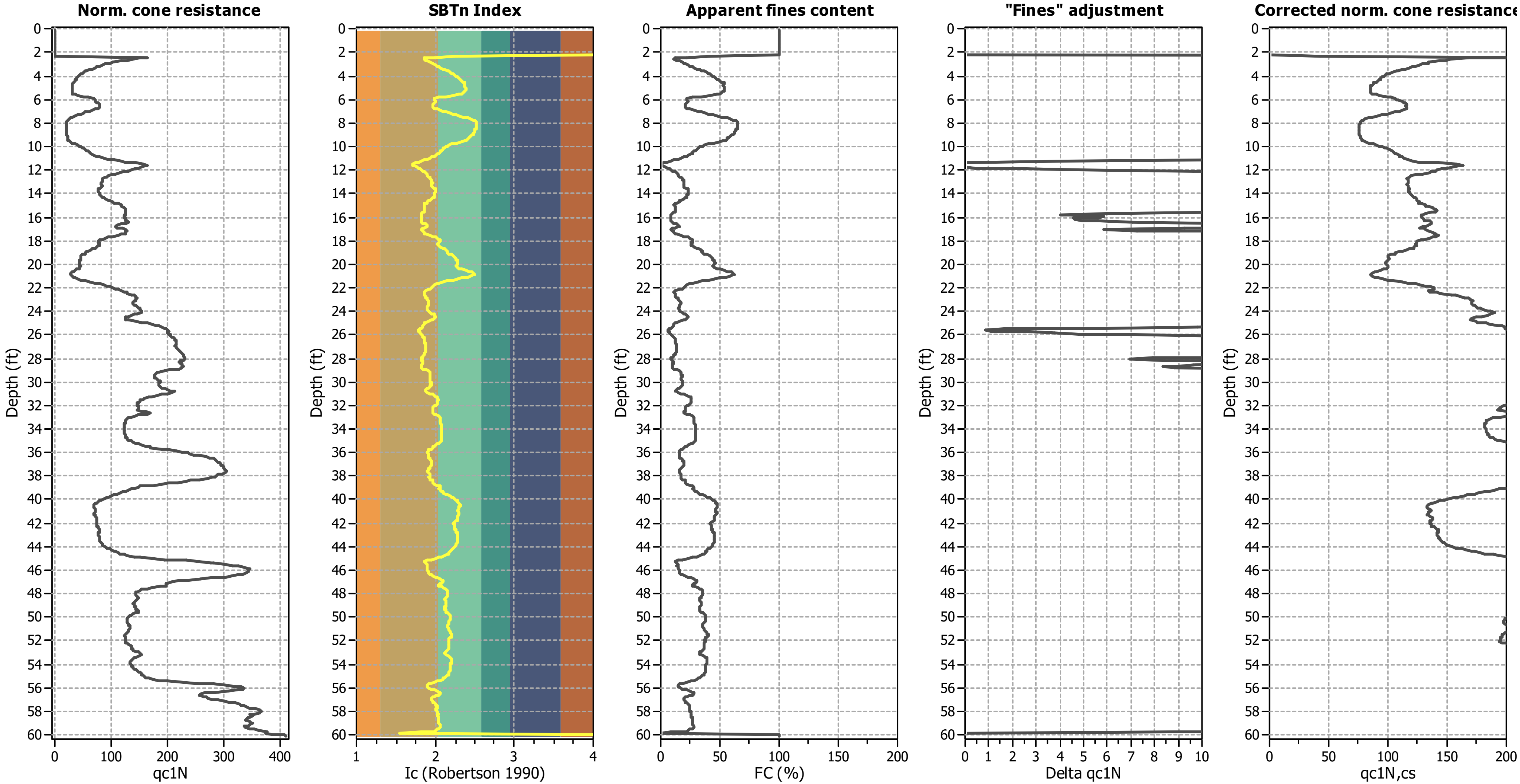

**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | Depth to GWT (erthq.): | 40.00 ft | Fill weight: | N/A |
| Fines correction method: | B&I (2014) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 8.00 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.71 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 60.00 ft | Fill height: | N/A | Limit depth: | N/A |

**SBTn legend**

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

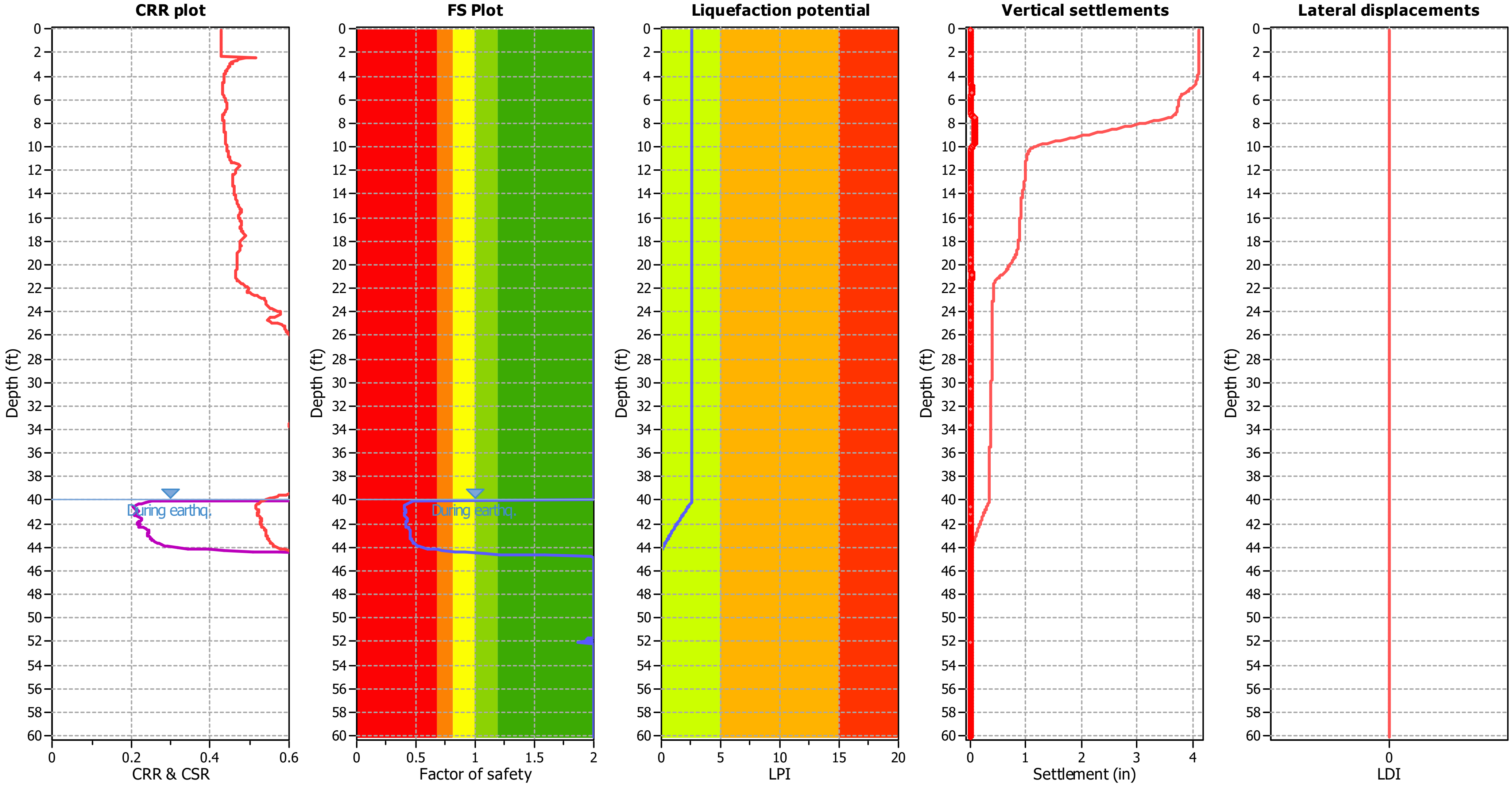
## Liquefaction analysis overall plots (intermediate results)




### Input parameters and analysis data

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | Depth to GWT (erthq.): | 40.00 ft | Fill weight: | N/A |
| Fines correction method: | B&I (2014) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 8.00 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.71 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 60.00 ft | Fill height: | N/A | Limit depth: | N/A |

## Liquefaction analysis overall plots




**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | Depth to GWT (erthq.): | 40.00 ft | Fill weight: | N/A |
| Fines correction method: | B&I (2014) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 8.00 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.71 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 60.00 ft | Fill height: | N/A | Limit depth: | N/A |

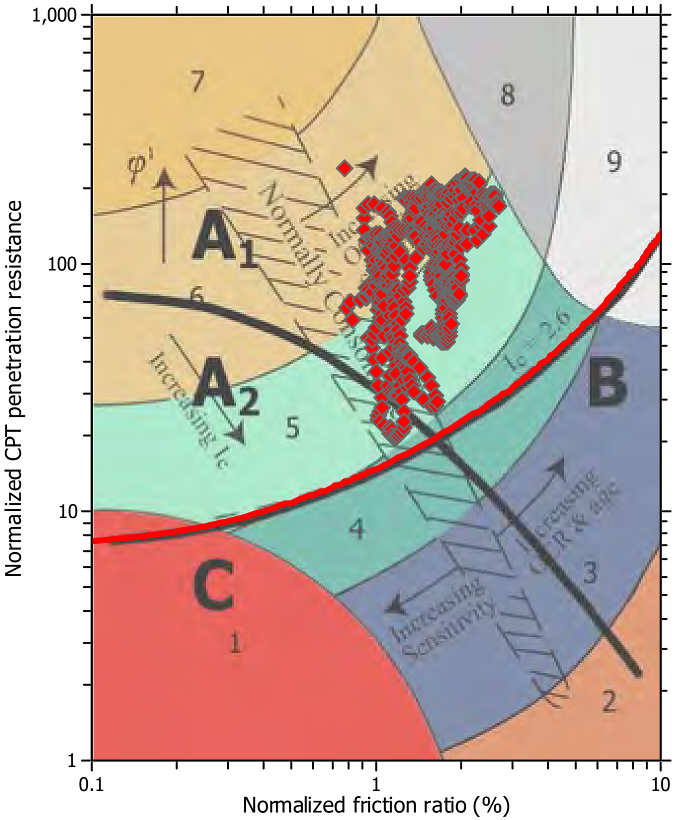
**F.S. color scheme**

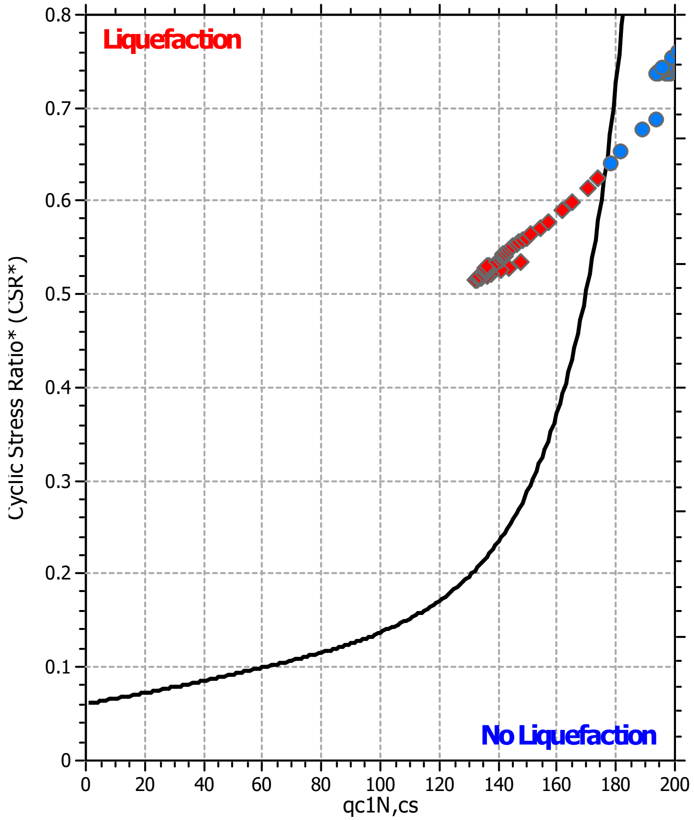
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

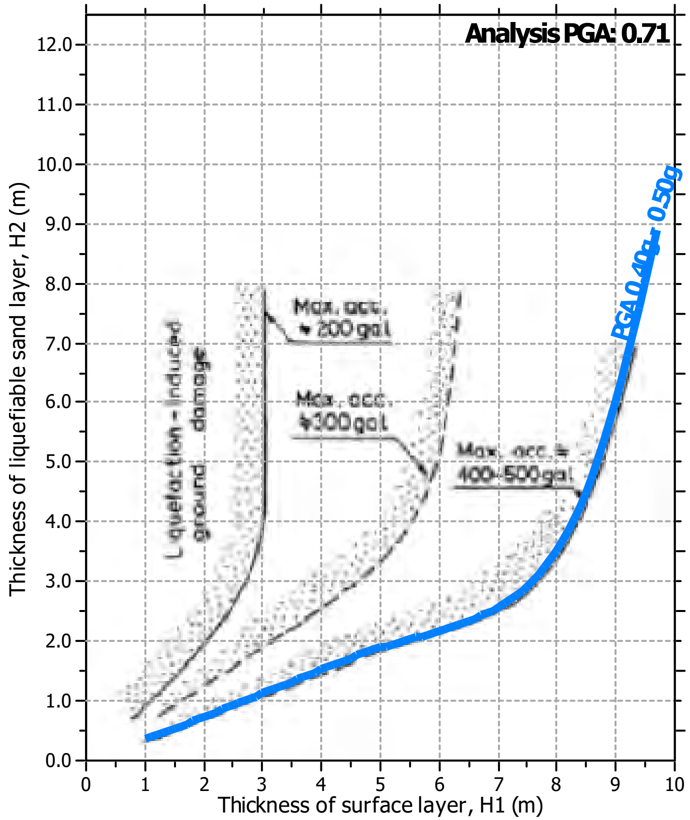
**LPI color scheme**

- Very high risk
- High risk
- Low risk

## Liquefaction analysis summary plots



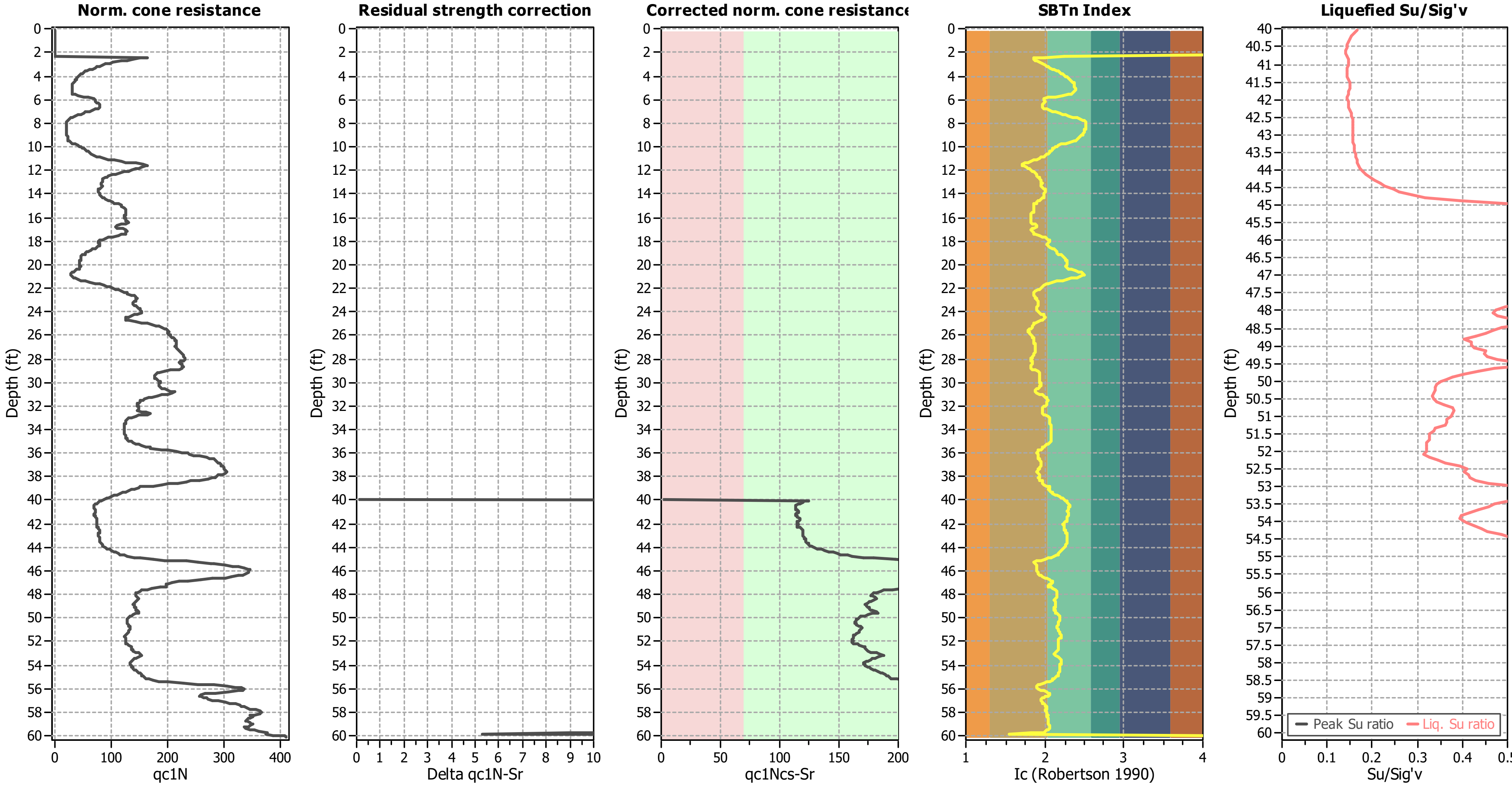




**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | Depth to GWT (erthq.): | 40.00 ft | Fill weight: | N/A |
| Fines correction method: | B&I (2014) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 8.00 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.71 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 60.00 ft | Fill height: | N/A | Limit depth: | N/A |

## Check for strength loss plots (Idriss & Boulanger (2008))




### Input parameters and analysis data

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | Depth to GWT (erthq.): | 40.00 ft | Fill weight: | N/A |
| Fines correction method: | B&I (2014) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 8.00 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.71 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 60.00 ft | Fill height: | N/A | Limit depth: | N/A |

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(510) 705-1664

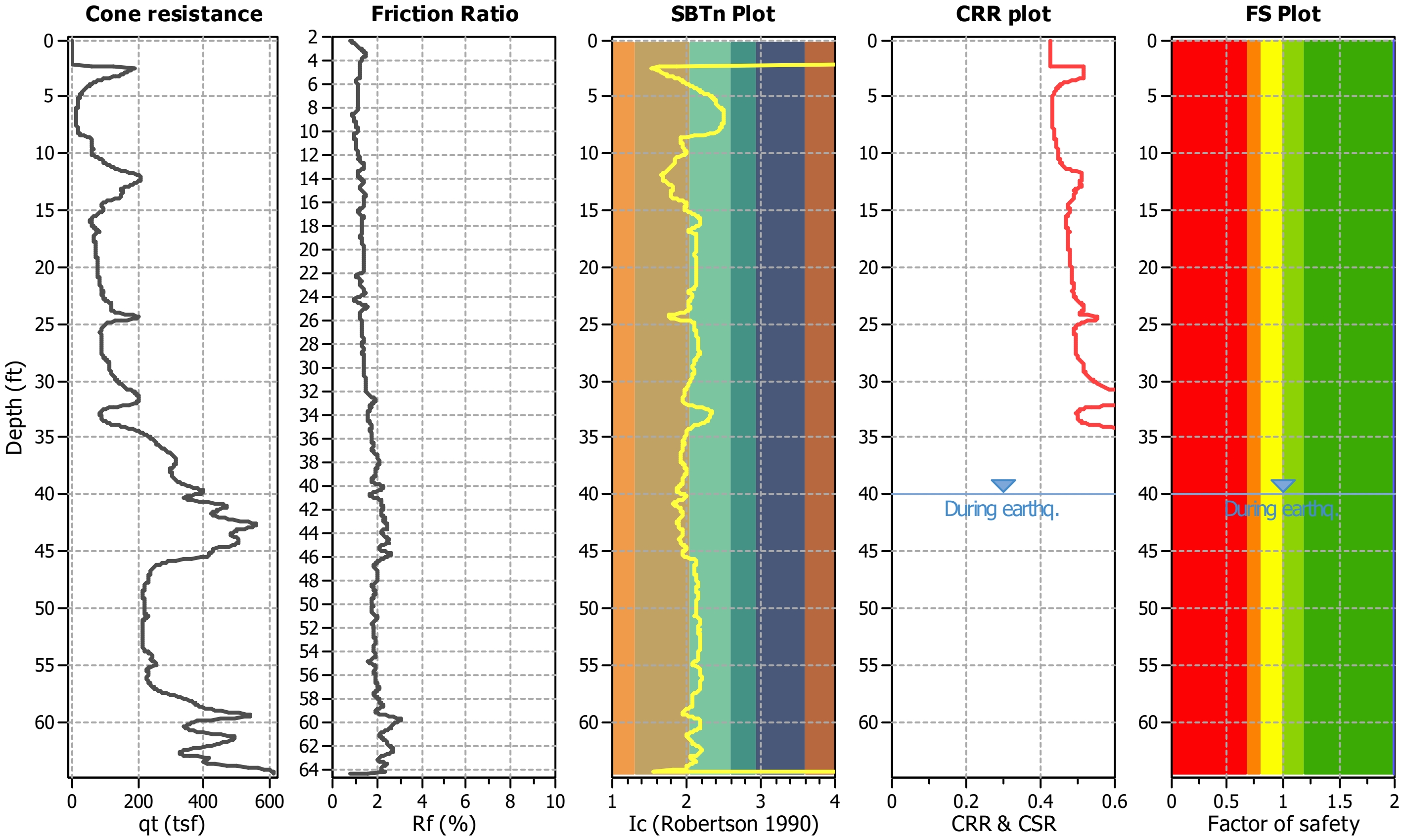
# LIQUEFACTION ANALYSIS REPORT

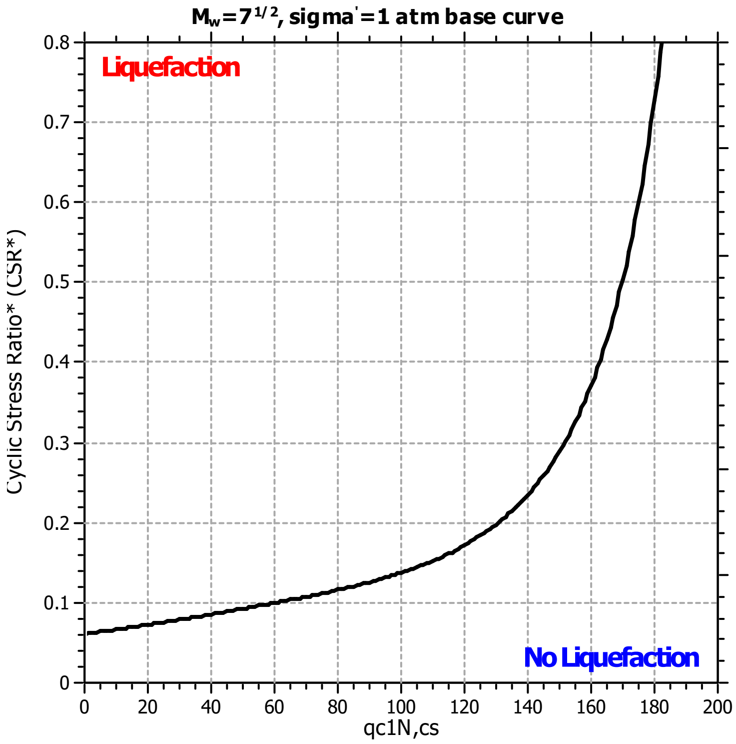
**Project title : 2550 Irving Street** **Location :**

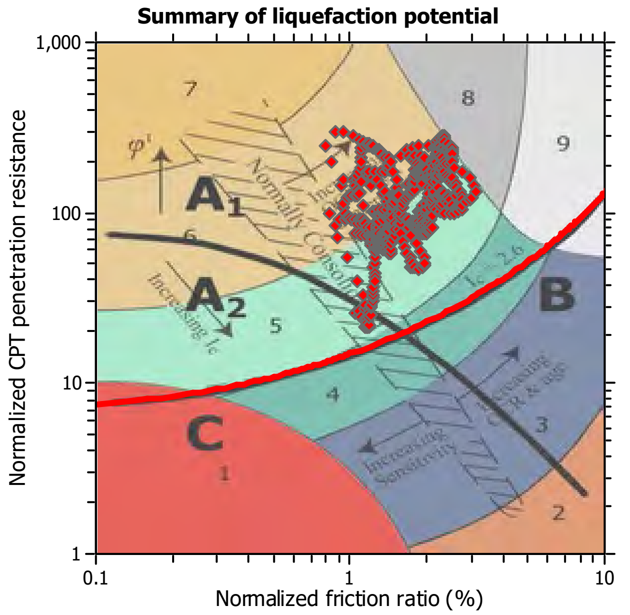
**CPT file : CPT-2**

**Input parameters and analysis data**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 60.00 ft | Use fill: | No | Clay like behavior applied: | Sands only |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 40.00 ft | Fill height: | N/A | Limit depth applied: | No |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth: | N/A |
| Earthquake magnitude $M_w$: | 8.00 | Ic cut-off value: | 2.60 | Trans. detect. applied: | No | MSF method: | Method based |
| Peak ground acceleration: | 0.71 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | | |

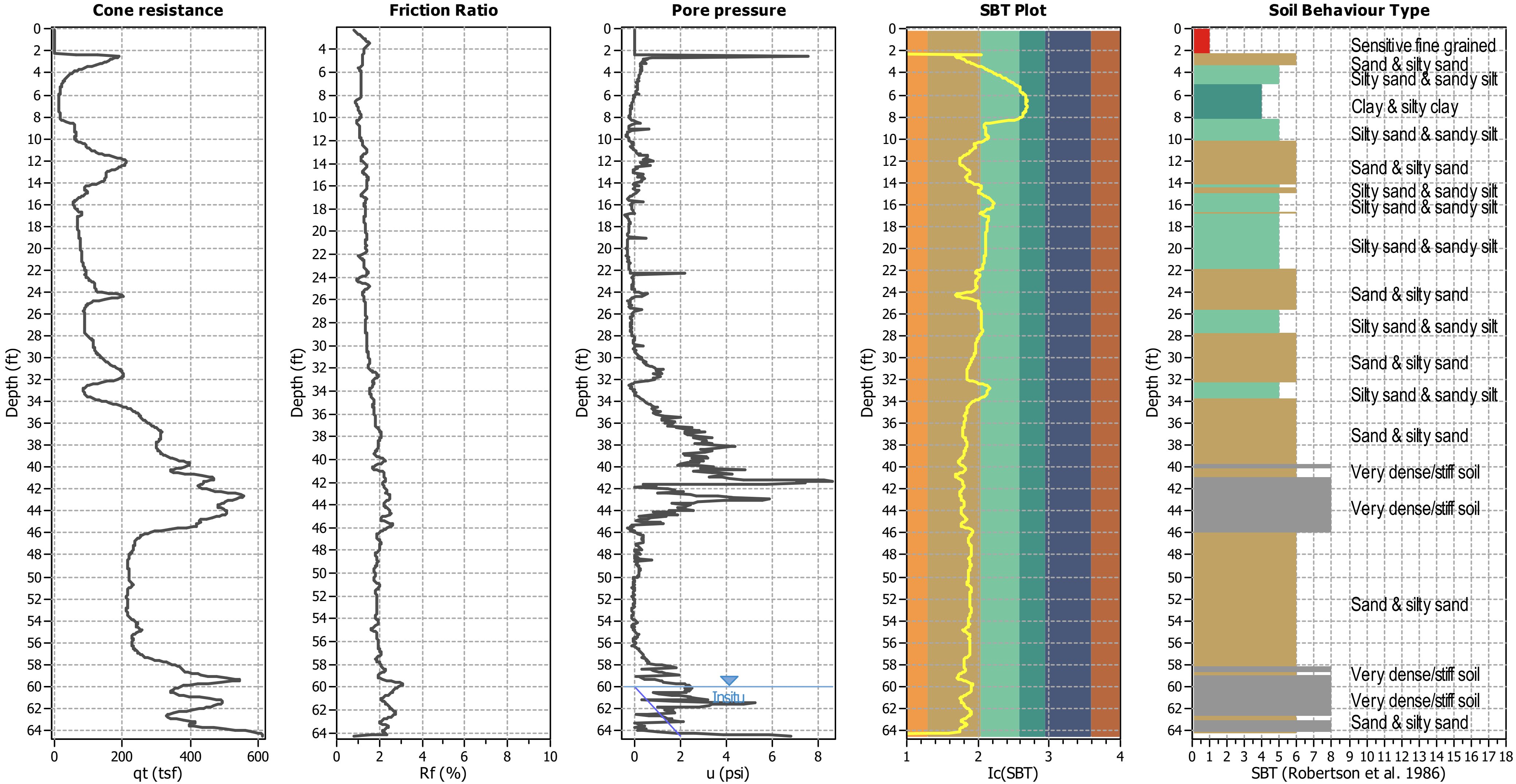



Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

## CPT basic interpretation plots



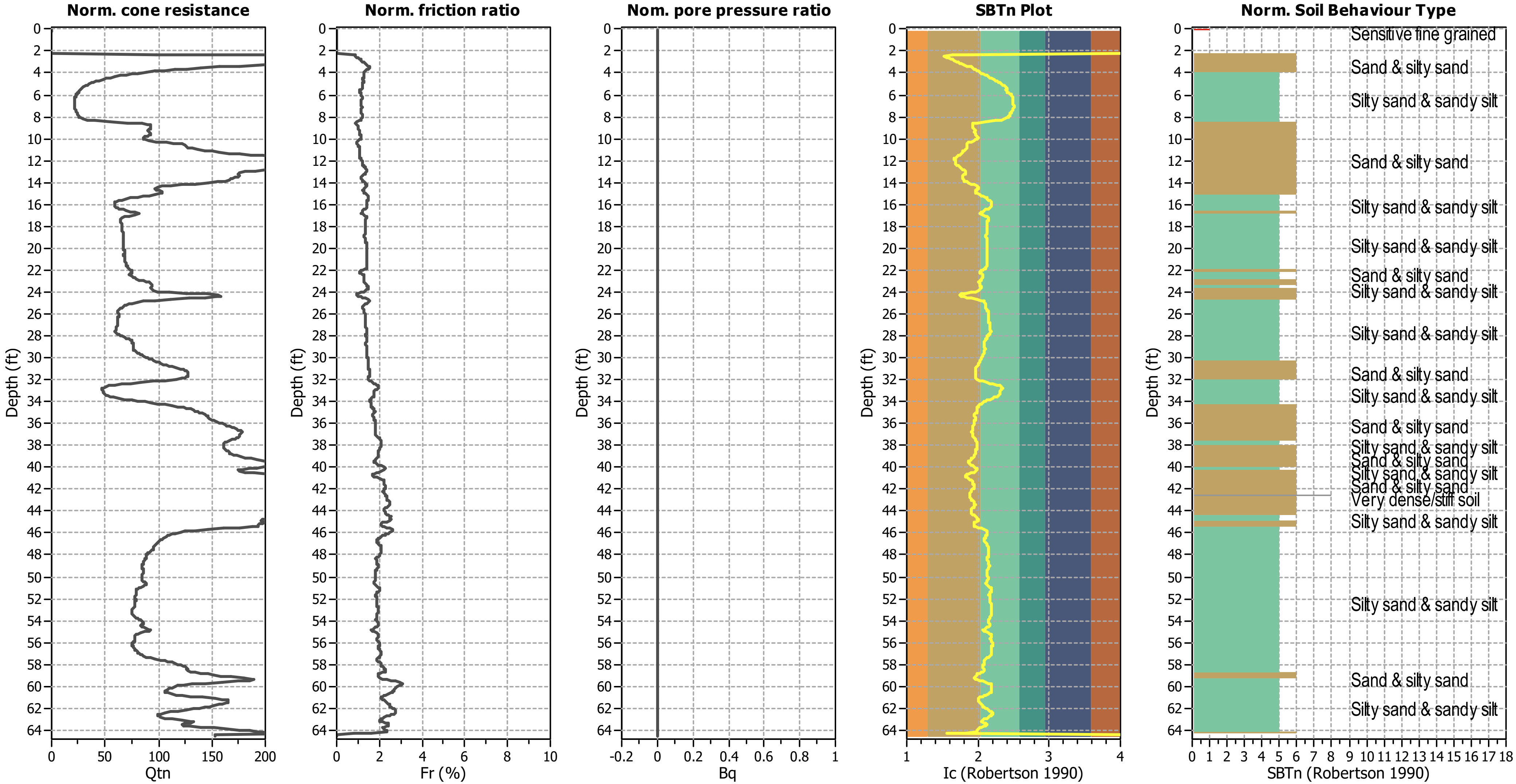

### Input parameters and analysis data

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | Depth to GWT (erthq.): | 40.00 ft | Fill weight: | N/A |
| Fines correction method: | B&I (2014) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 8.00 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.71 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 60.00 ft | Fill height: | N/A | Limit depth: | N/A |

### SBT legend

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

## CPT basic interpretation plots (normalized)



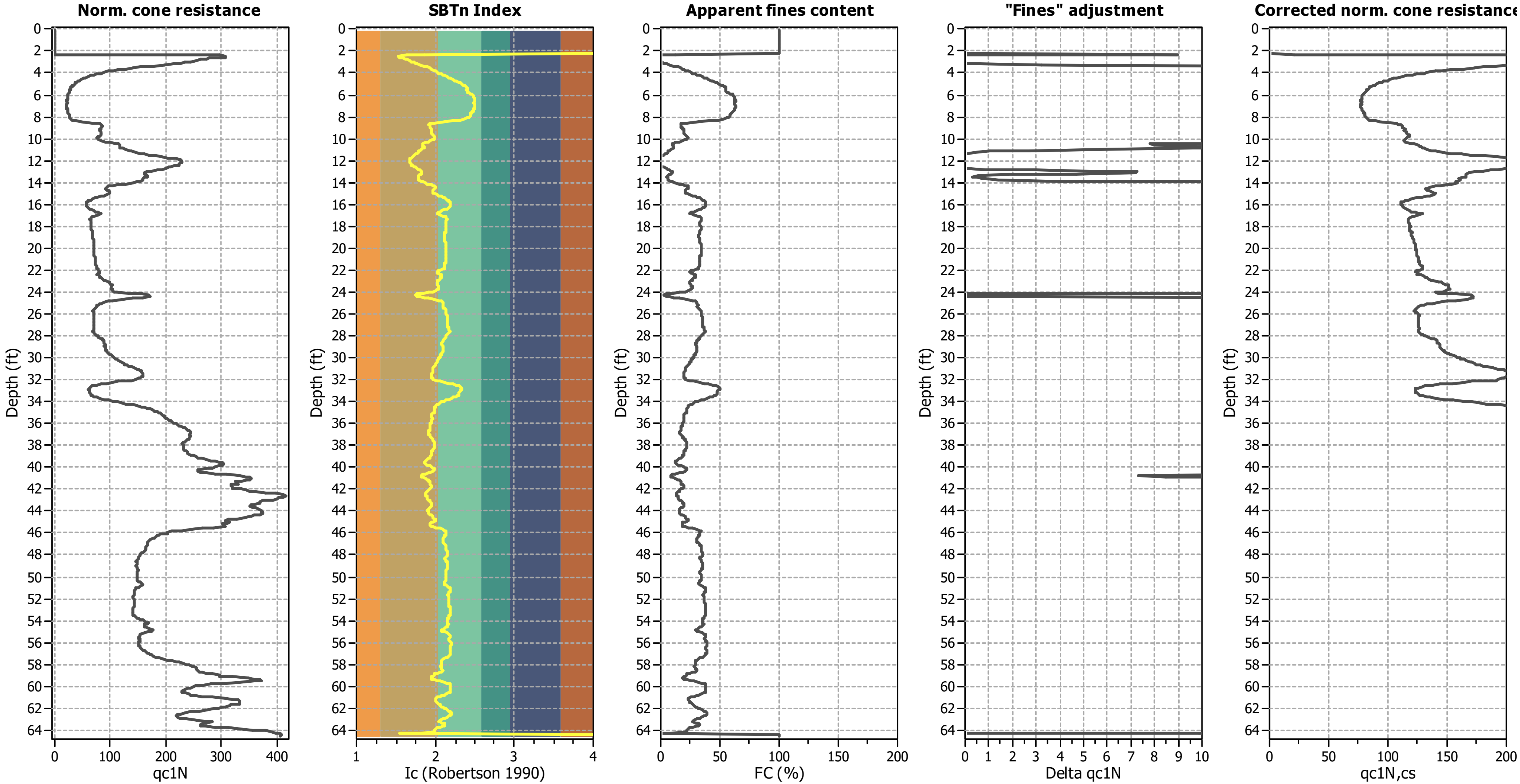

### Input parameters and analysis data

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | Depth to GWT (erthq.): | 40.00 ft | Fill weight: | N/A |
| Fines correction method: | B&I (2014) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 8.00 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.71 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 60.00 ft | Fill height: | N/A | Limit depth: | N/A |

### SBTn legend

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

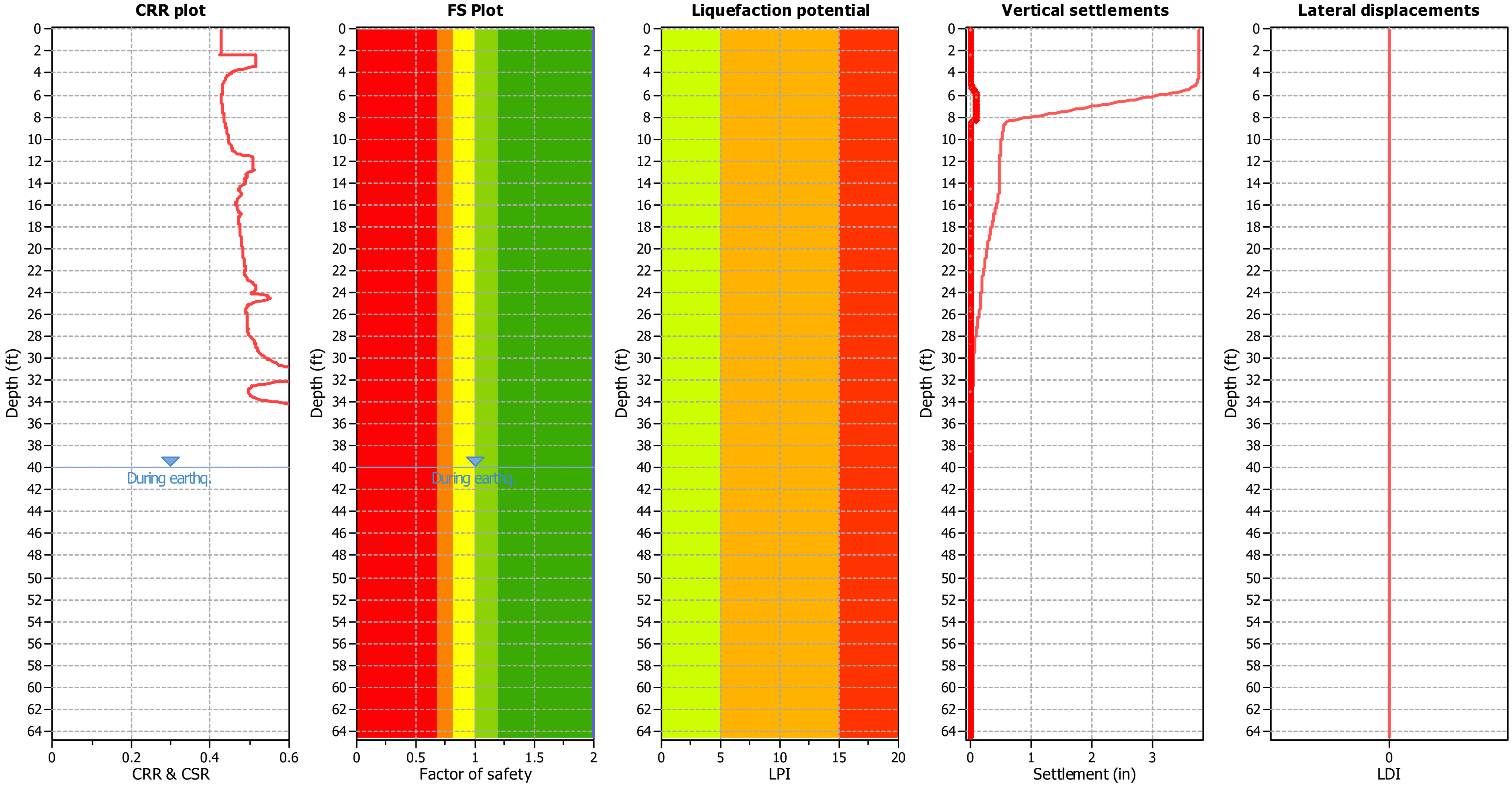
# Liquefaction analysis overall plots (intermediate results)




**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | Depth to GWT (erthq.): | 40.00 ft | Fill weight: | N/A |
| Fines correction method: | B&I (2014) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 8.00 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.71 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 60.00 ft | Fill height: | N/A | Limit depth: | N/A |

# Liquefaction analysis overall plots




**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | Depth to GWT (erthq.): | 40.00 ft | Fill weight: | N/A |
| Fines correction method: | B&I (2014) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 8.00 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.71 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 60.00 ft | Fill height: | N/A | Limit depth: | N/A |

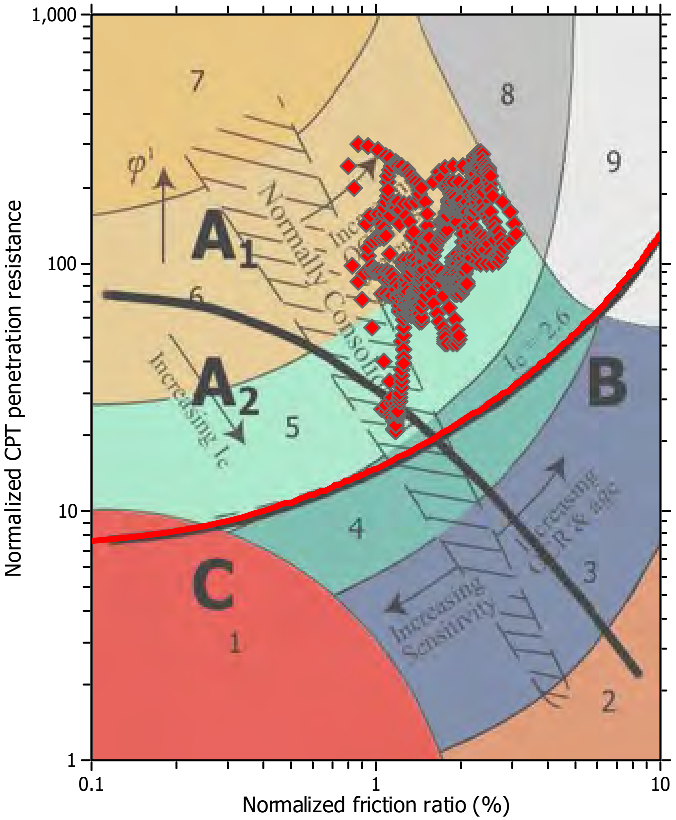
**F.S. color scheme**

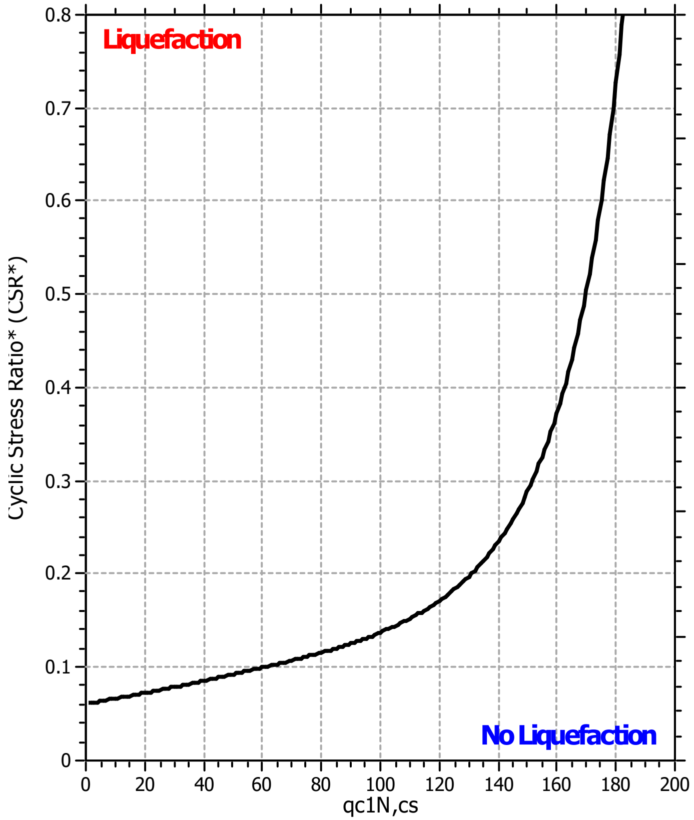
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

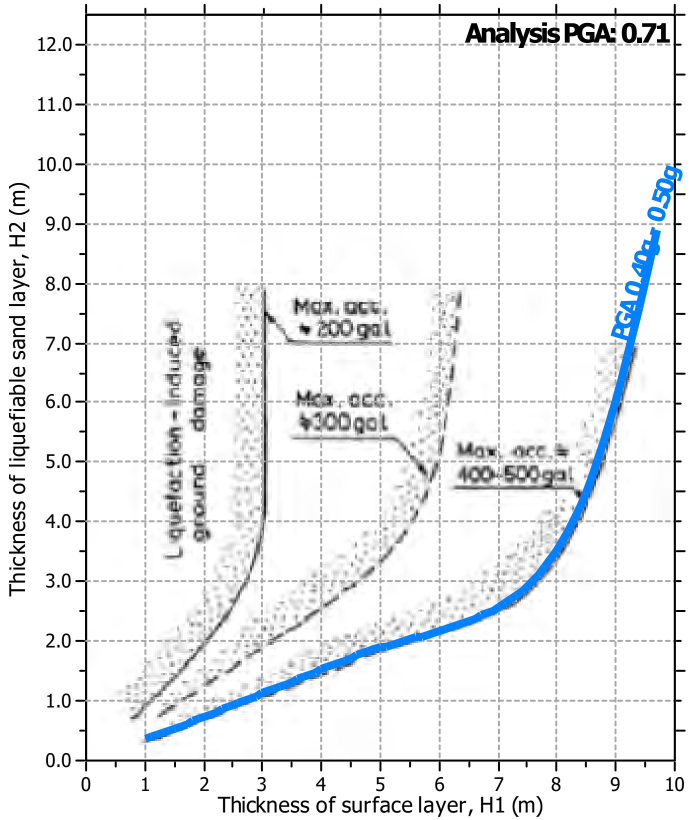
**LPI color scheme**

- Very high risk
- High risk
- Low risk

## Liquefaction analysis summary plots



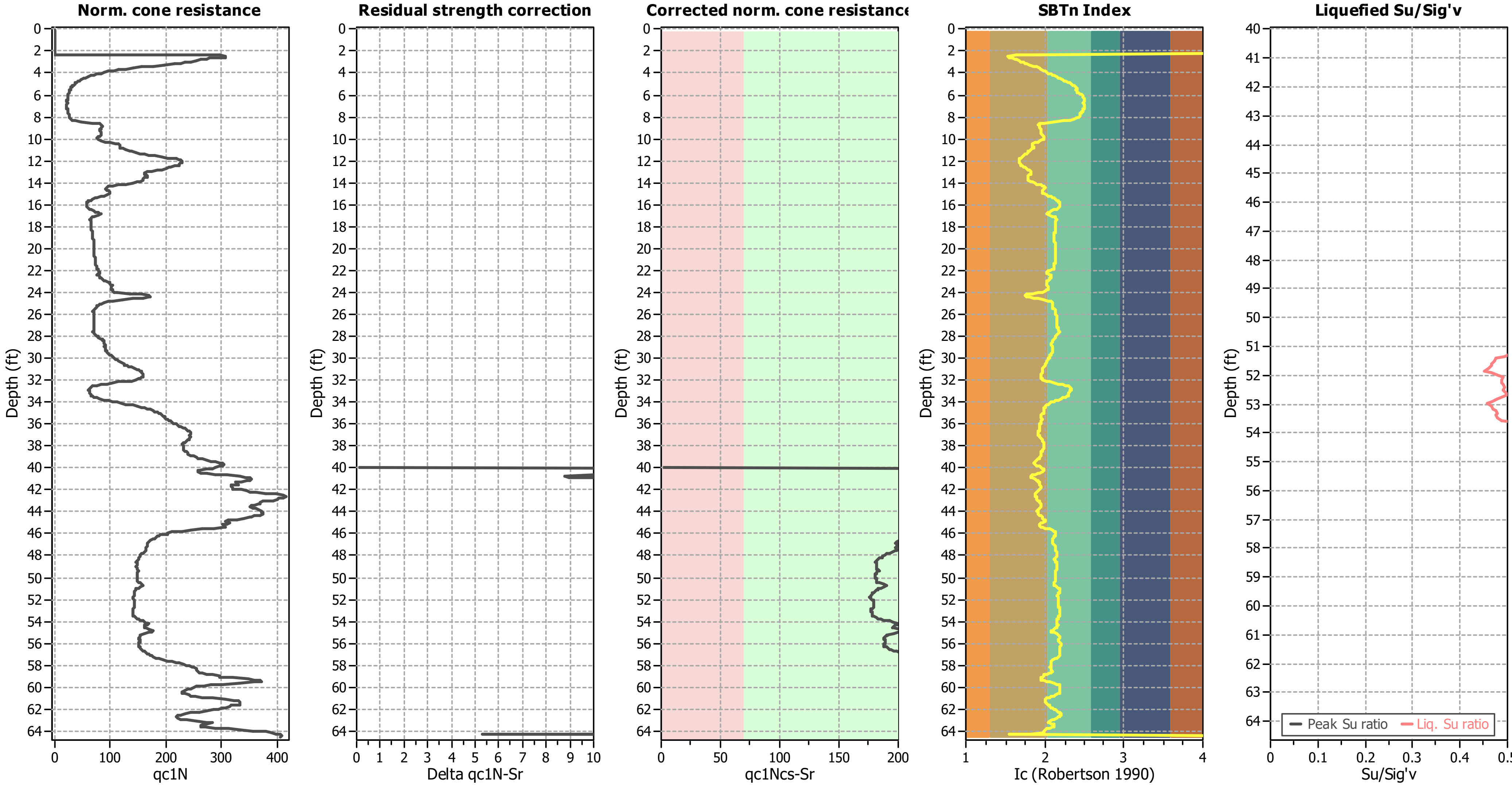




**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | Depth to GWT (erthq.): | 40.00 ft | Fill weight: | N/A |
| Fines correction method: | B&I (2014) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 8.00 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.71 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 60.00 ft | Fill height: | N/A | Limit depth: | N/A |

## Check for strength loss plots (Idriss & Boulanger (2008))




### Input parameters and analysis data

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | Depth to GWT (erthq.): | 40.00 ft | Fill weight: | N/A |
| Fines correction method: | B&I (2014) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 8.00 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.71 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 60.00 ft | Fill height: | N/A | Limit depth: | N/A |

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Berkeley, CA 94710
(510) 705-1664

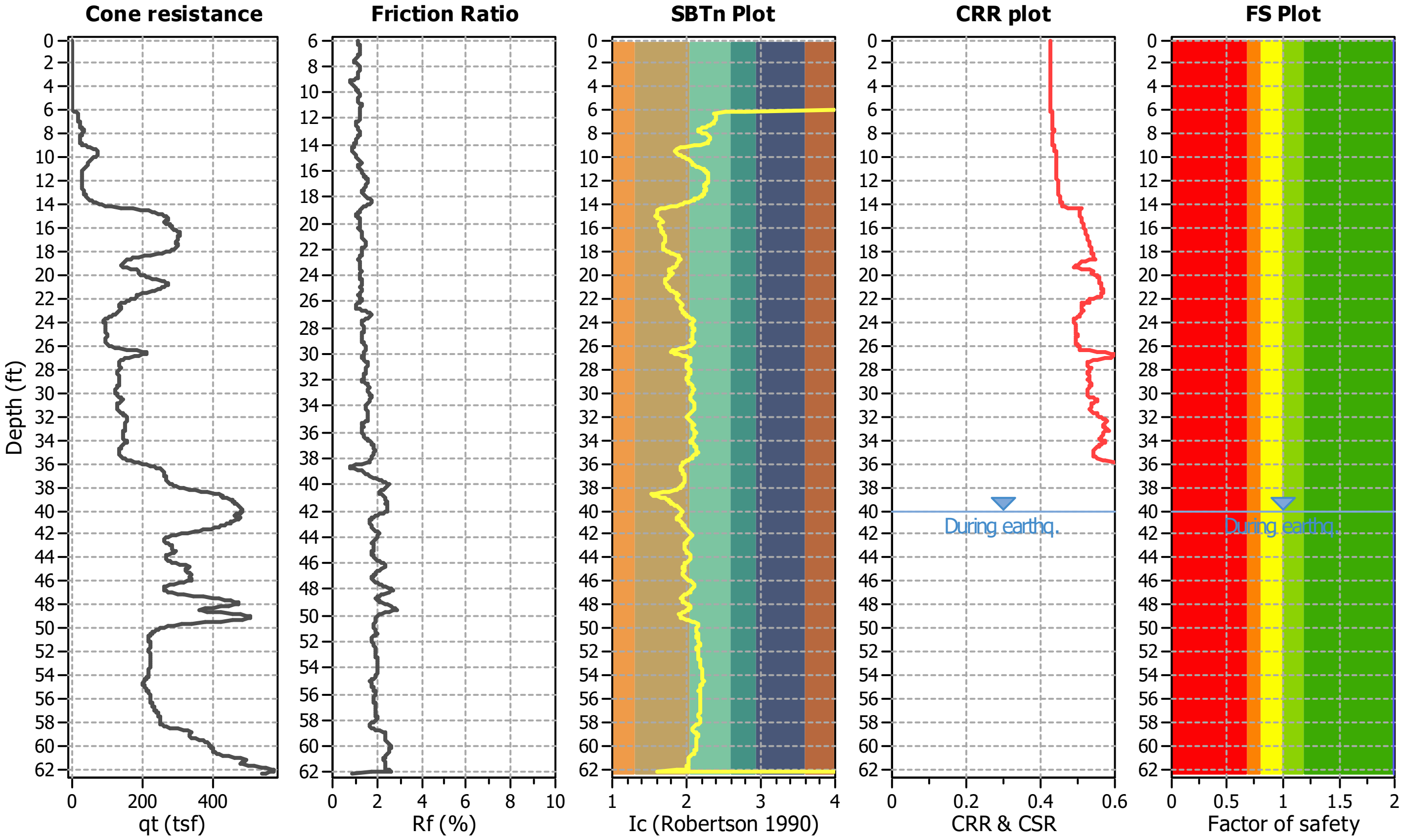
## LIQUEFACTION ANALYSIS REPORT

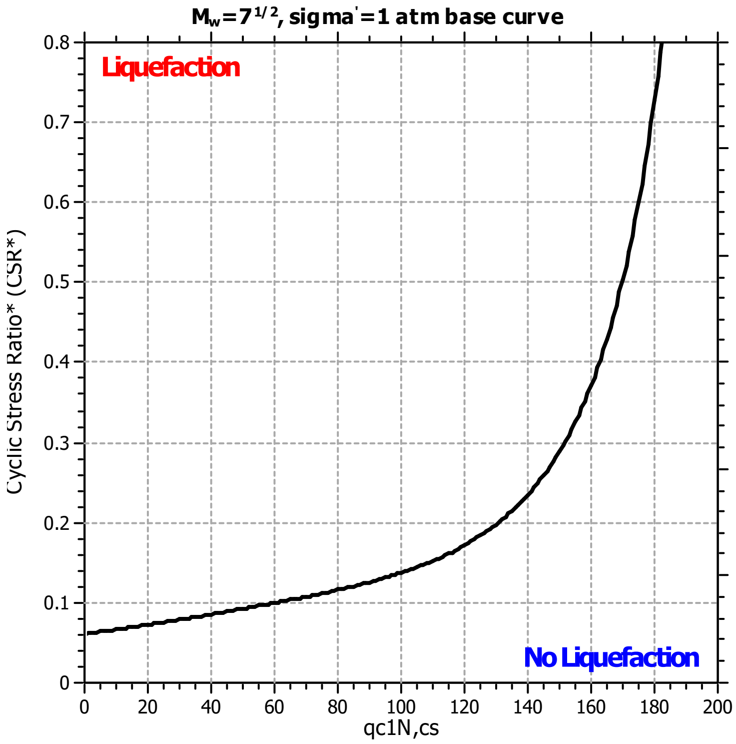
**Project title : 2550 Irving Street** **Location :**

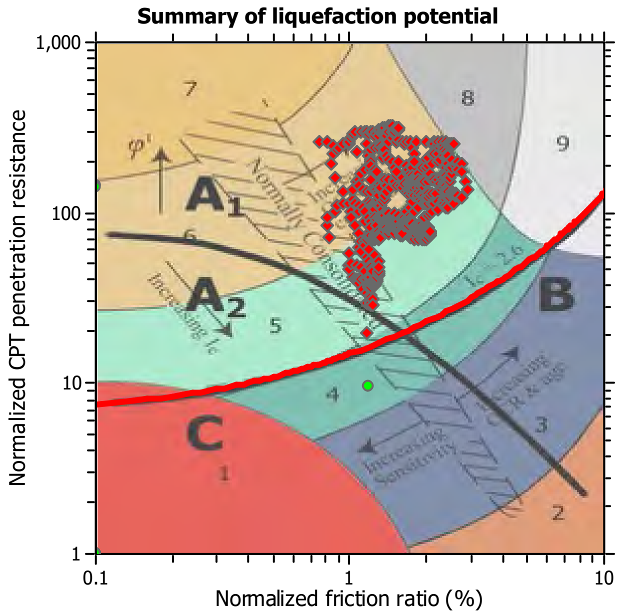
**CPT file : CPT-3**

### Input parameters and analysis data

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | G.W.T. (in-situ): | 60.00 ft | Use fill: | No | Clay like behavior applied: | Sands only |
| Fines correction method: | B&I (2014) | G.W.T. (earthq.): | 40.00 ft | Fill height: | N/A | | |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth applied: | No |
| Earthquake magnitude $M_w$: | 8.00 | Ic cut-off value: | 2.60 | Trans. detect. applied: | No | Limit depth: | N/A |
| Peak ground acceleration: | 0.71 | Unit weight calculation: | Based on SBT | $K_\sigma$ applied: | Yes | MSF method: | Method based |

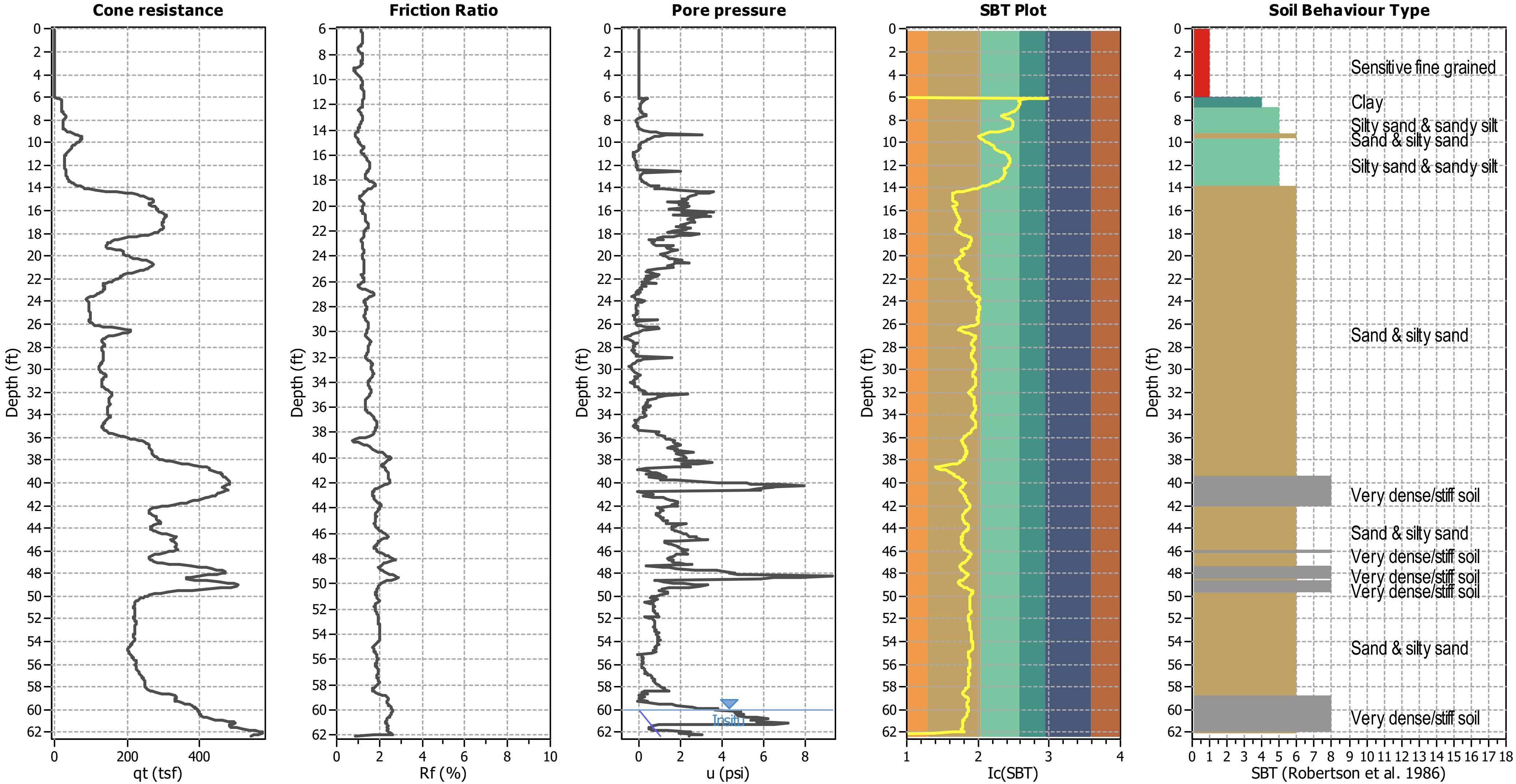



Zone $A_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone $A_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

# CPT basic interpretation plots



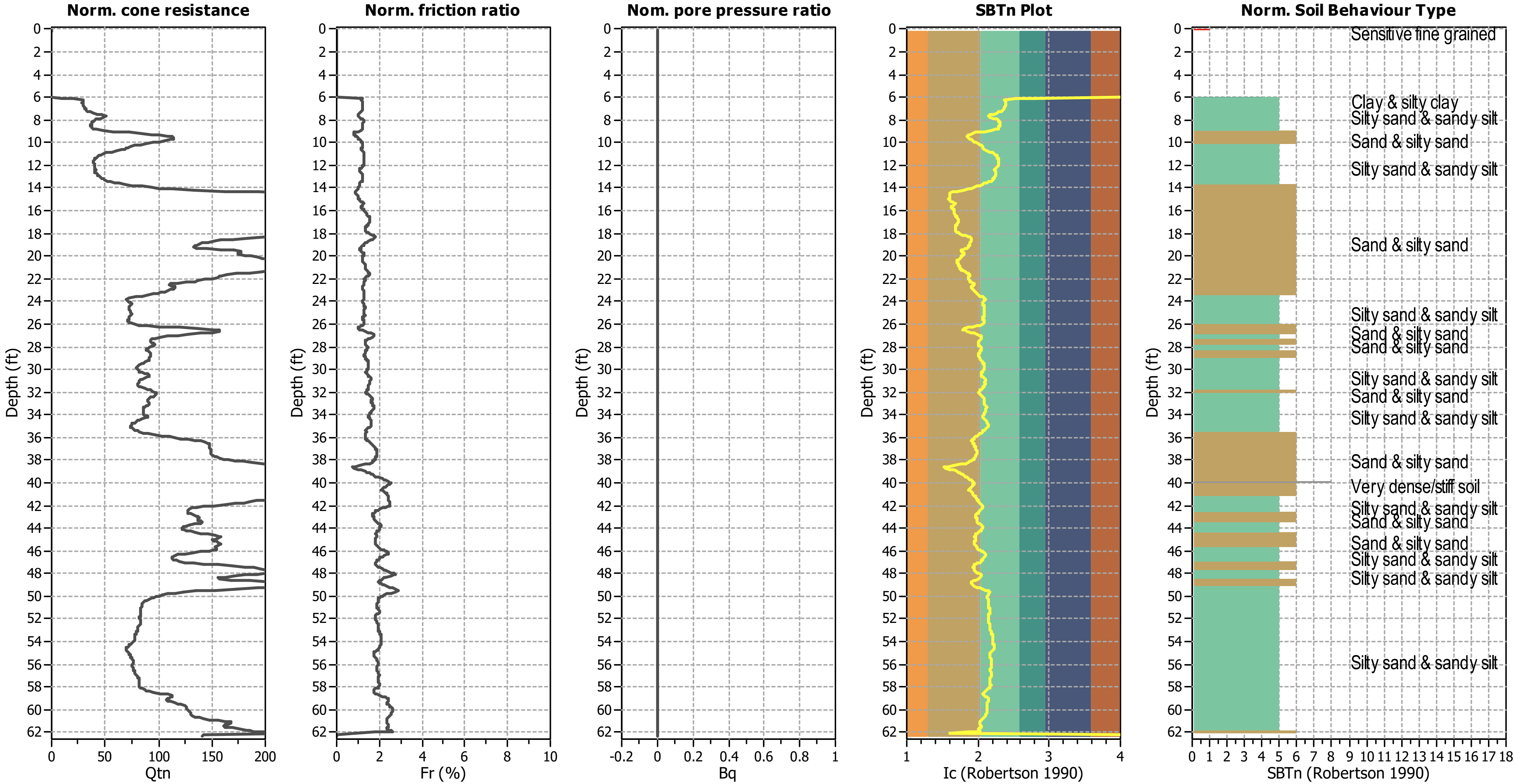

**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | Depth to GWT (erthq.): | 40.00 ft | Fill weight: | N/A |
| Fines correction method: | B&I (2014) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 8.00 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.71 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 60.00 ft | Fill height: | N/A | Limit depth: | N/A |

**SBT legend**

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

## CPT basic interpretation plots (normalized)



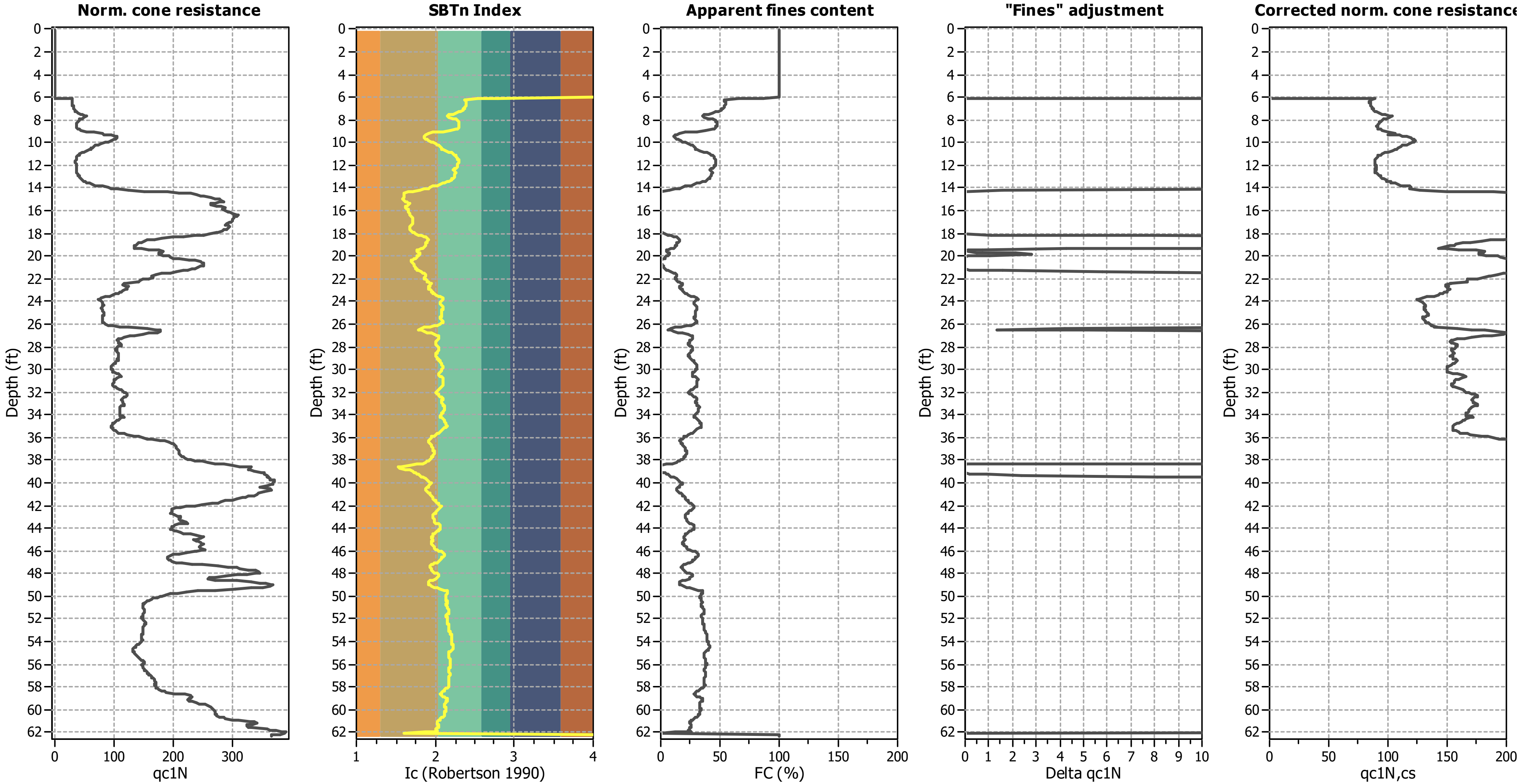

**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | Depth to GWT (erthq.): | 40.00 ft | Fill weight: | N/A |
| Fines correction method: | B&I (2014) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 8.00 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.71 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 60.00 ft | Fill height: | N/A | Limit depth: | N/A |

**SBTn legend**

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

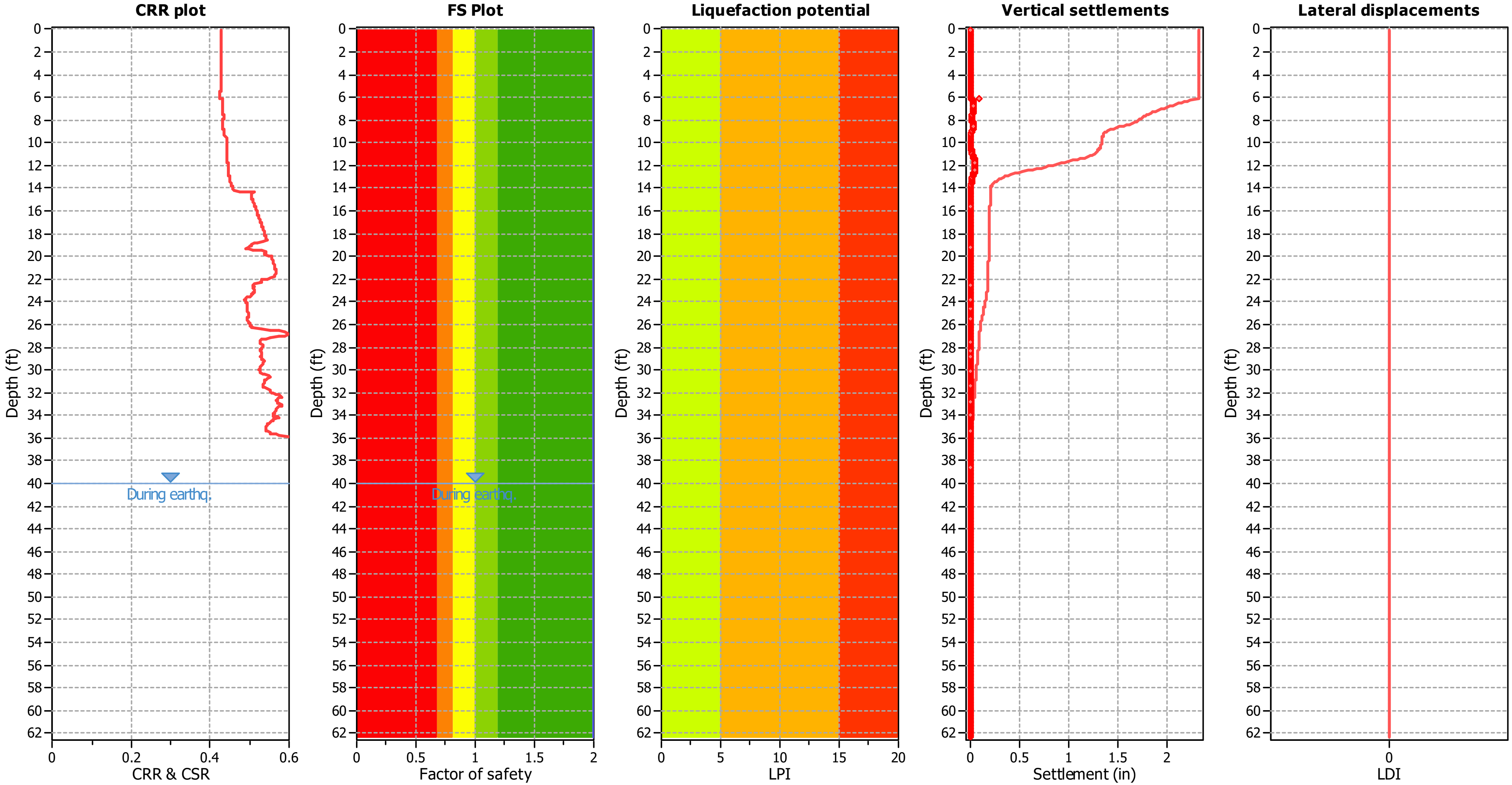
# Liquefaction analysis overall plots (intermediate results)




**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | Depth to GWT (erthq.): | 40.00 ft | Fill weight: | N/A |
| Fines correction method: | B&I (2014) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 8.00 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.71 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 60.00 ft | Fill height: | N/A | Limit depth: | N/A |

## Liquefaction analysis overall plots




**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | Depth to GWT (erthq.): | 40.00 ft | Fill weight: | N/A |
| Fines correction method: | B&I (2014) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 8.00 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.71 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 60.00 ft | Fill height: | N/A | Limit depth: | N/A |

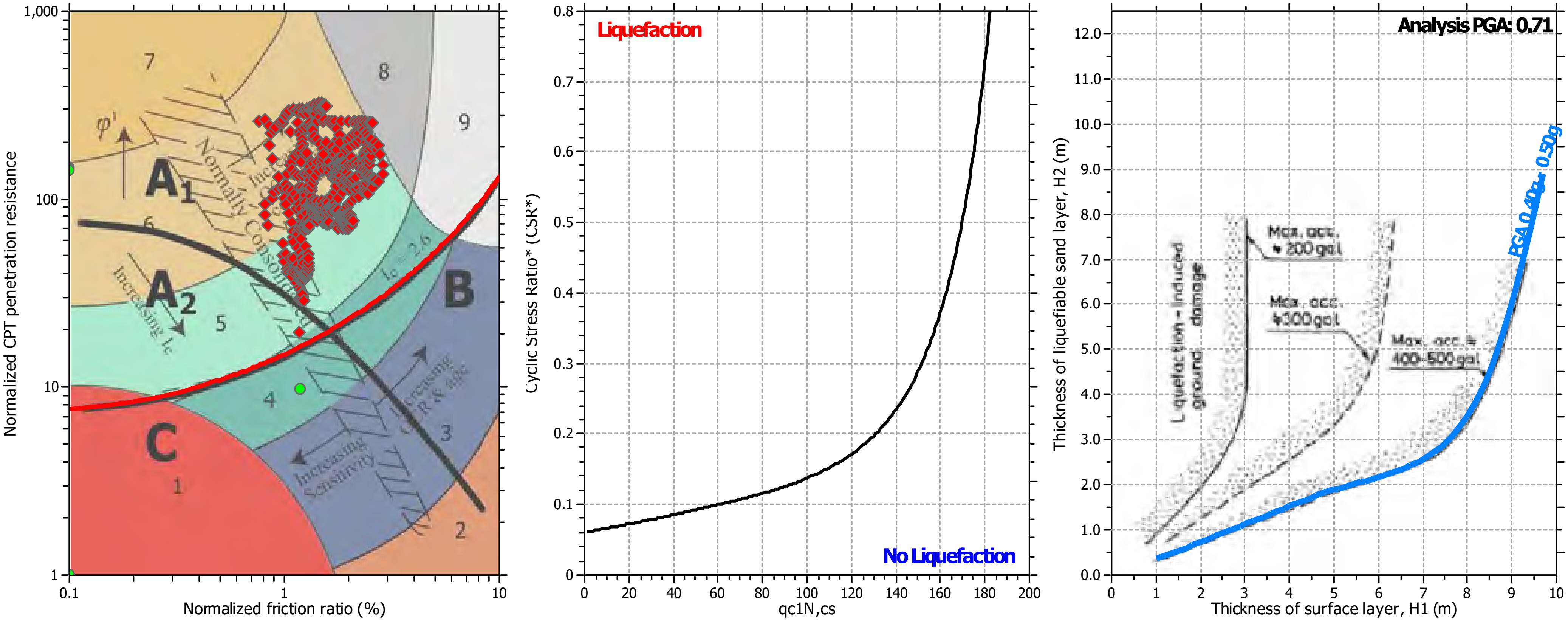
**F.S. color scheme**

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

**LPI color scheme**

- Very high risk
- High risk
- Low risk

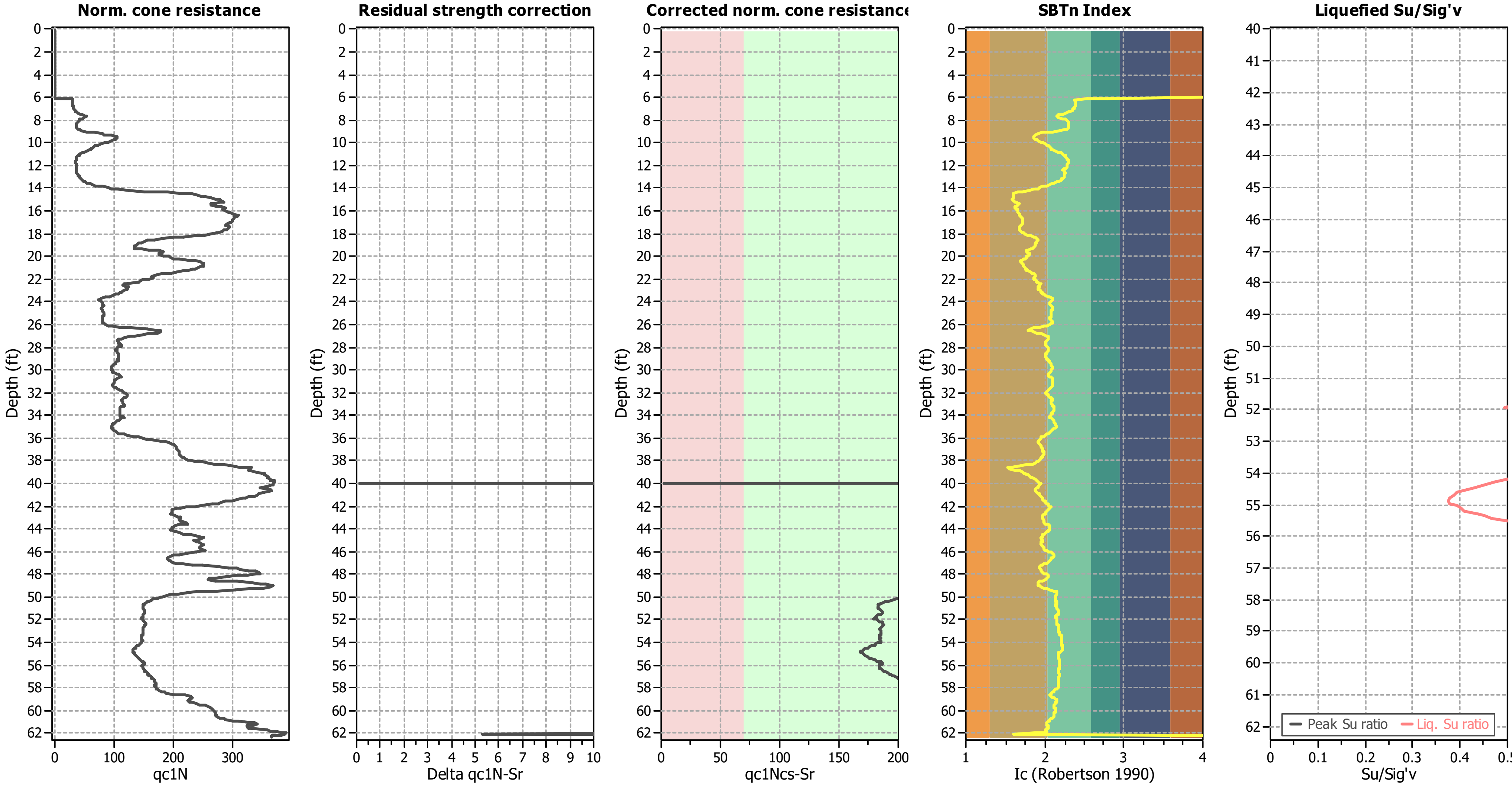
## Liquefaction analysis summary plots




### Input parameters and analysis data

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | Depth to GWT (erthq.): | 40.00 ft | Fill weight: | N/A |
| Fines correction method: | B&I (2014) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 8.00 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.71 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 60.00 ft | Fill height: | N/A | Limit depth: | N/A |

## Check for strength loss plots (Idriss & Boulanger (2008))

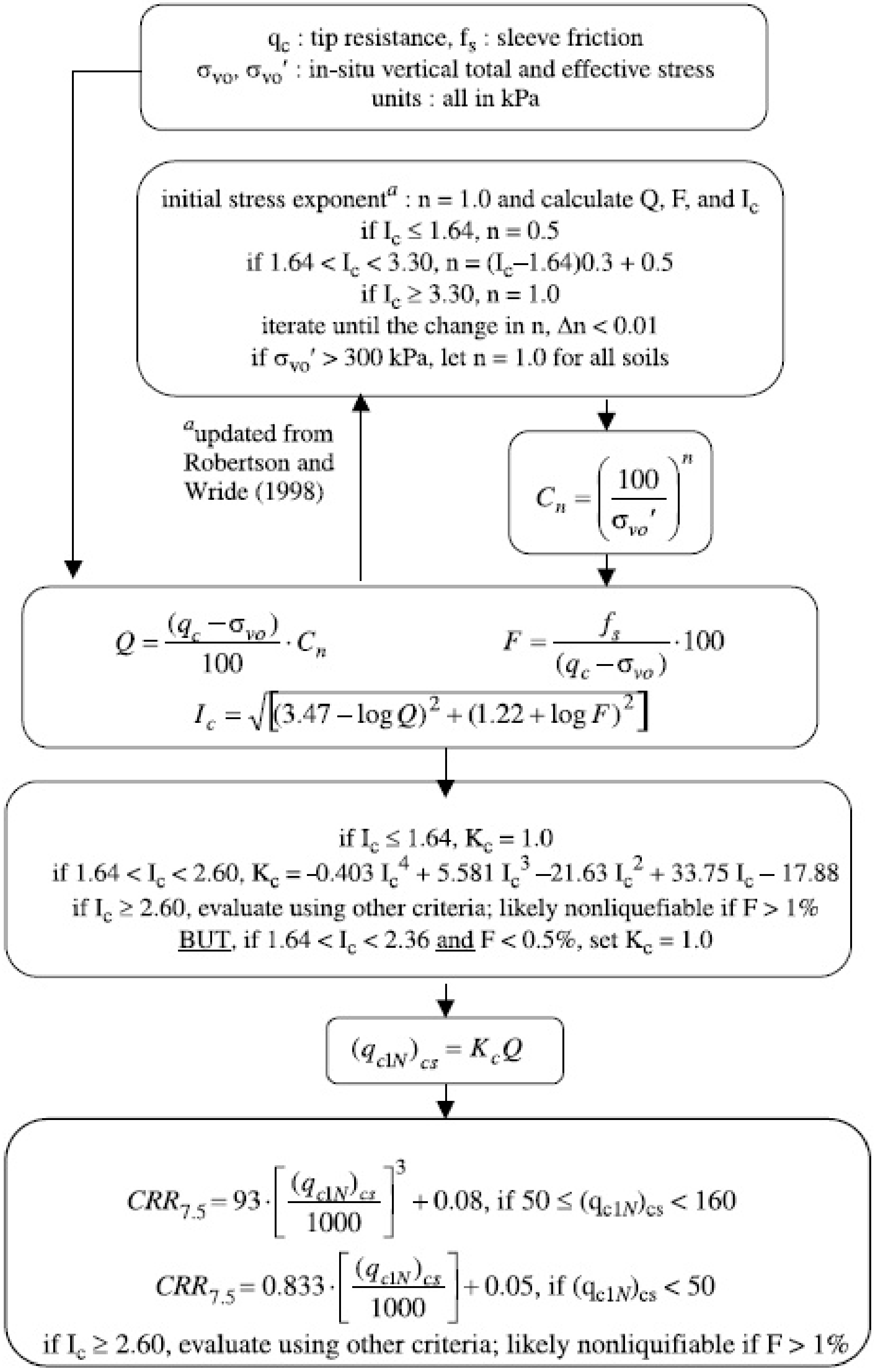



**Input parameters and analysis data**

| | | | | | |
|---|---|---|---|---|---|
| Analysis method: | B&I (2014) | Depth to GWT (erthq.): | 40.00 ft | Fill weight: | N/A |
| Fines correction method: | B&I (2014) | Average results interval: | 3 | Transition detect. applied: | No |
| Points to test: | Based on Ic value | Ic cut-off value: | 2.60 | $K_\sigma$ applied: | Yes |
| Earthquake magnitude $M_w$: | 8.00 | Unit weight calculation: | Based on SBT | Clay like behavior applied: | Sands only |
| Peak ground acceleration: | 0.71 | Use fill: | No | Limit depth applied: | No |
| Depth to water table (insitu): | 60.00 ft | Fill height: | N/A | Limit depth: | N/A |

**Procedure for the evaluation of soil liquefaction resistance, NCEER (1998)**

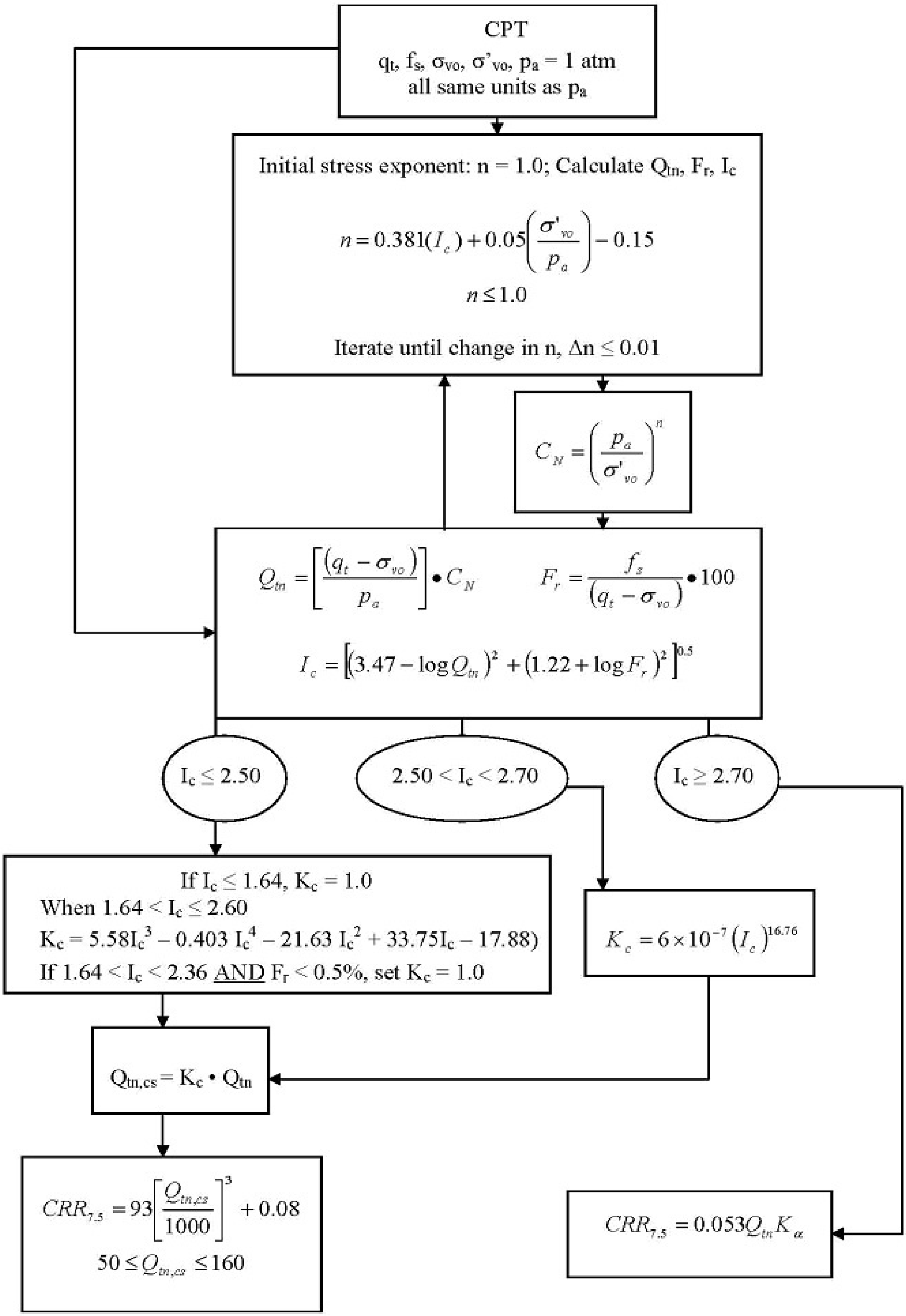
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:




[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

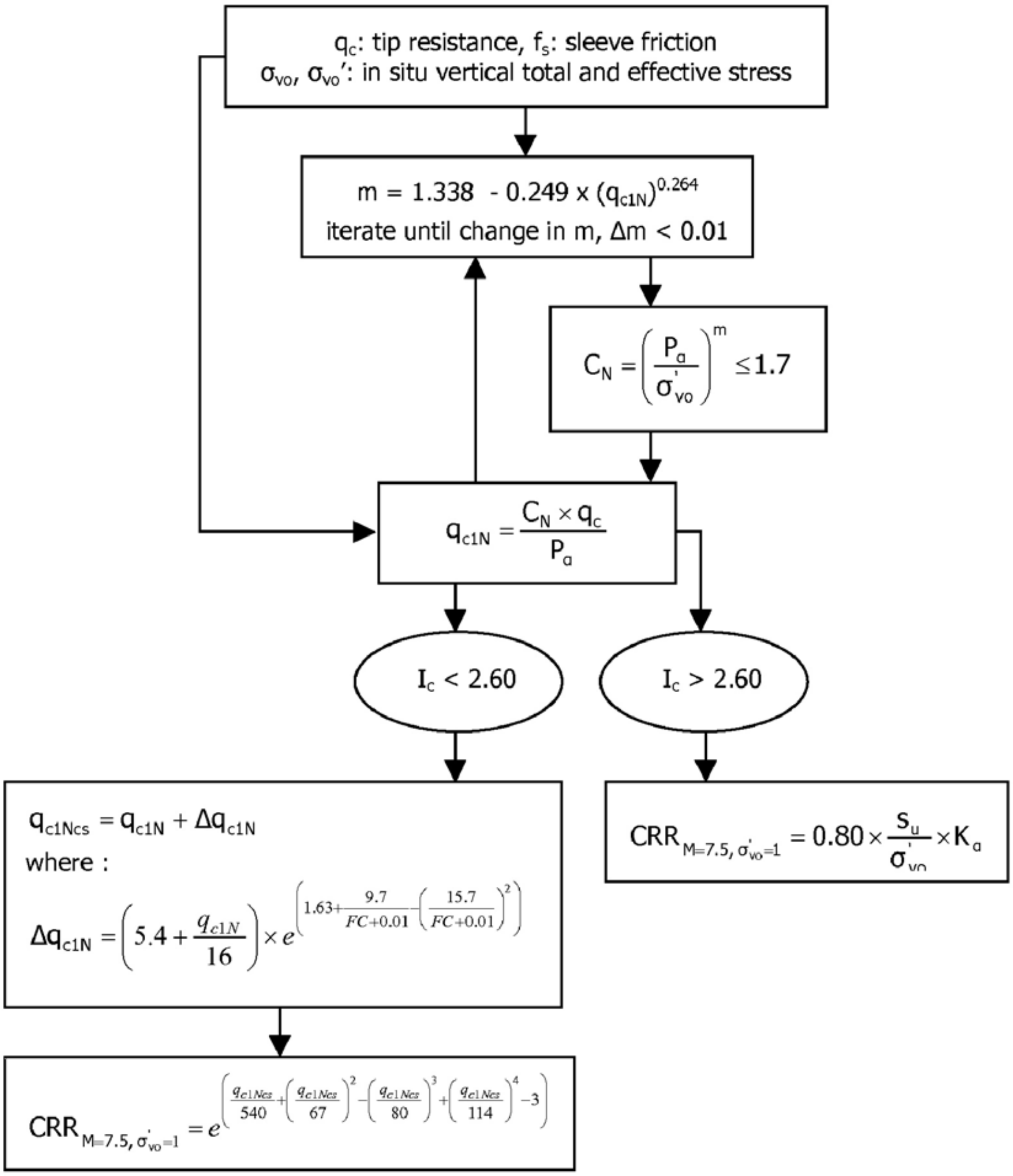
### **Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)**

Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. This procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:

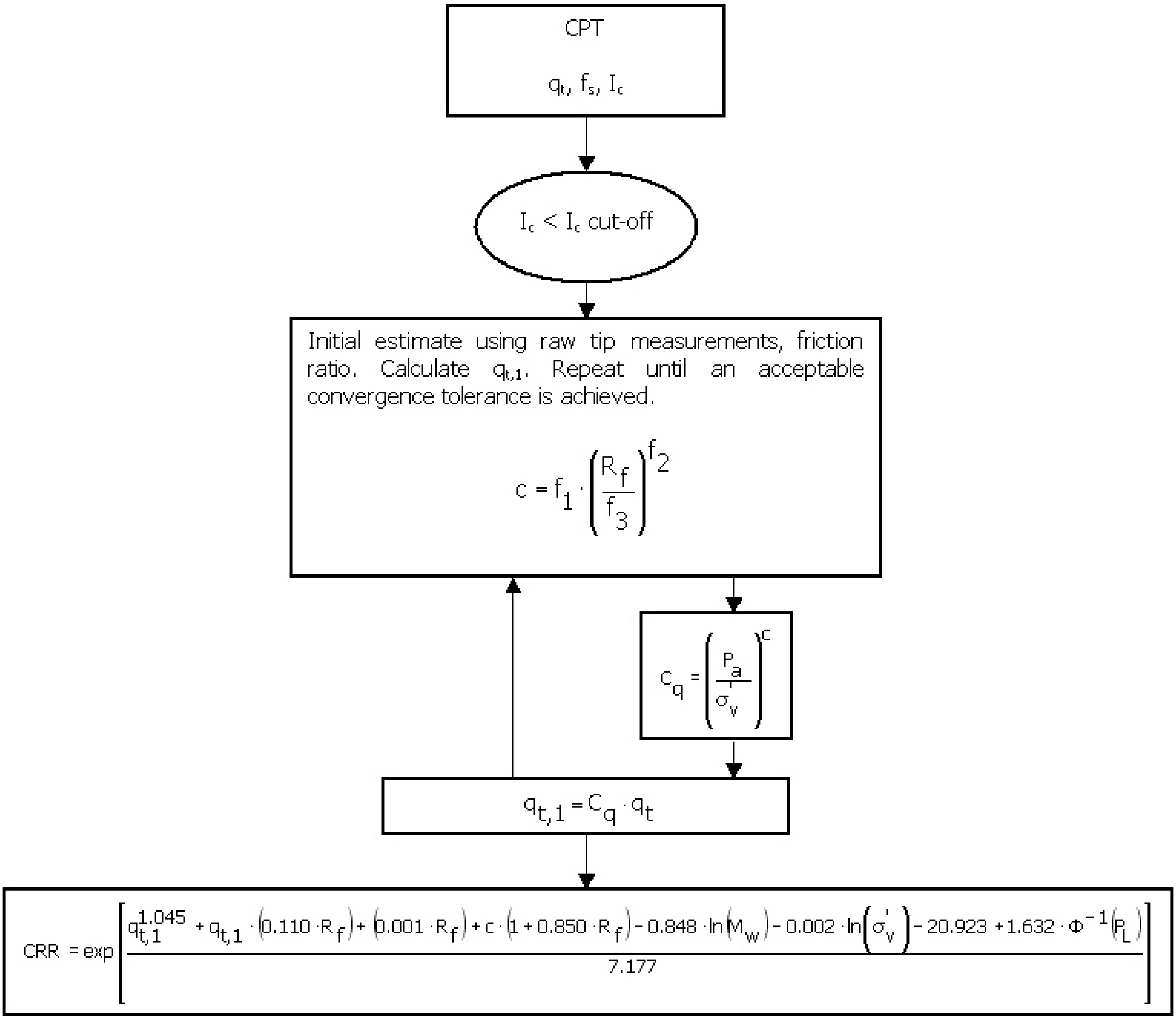



[1] P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009

**Procedure for the evaluation of soil liquefaction resistance, Idriss & Boulanger (2008)**



**Procedure for the evaluation of soil liquefaction resistance (sandy soils), Moss et al. (2006)**



**Procedure for the evaluation of soil liquefaction resistance, Boulanger & Idriss(2014)**

$$FS_{liq} = \frac{CRR_{M=7.5,\sigma_v'=1atm}}{CSR_{M=7.5,\sigma_v'=1atm}}$$

$$CSR_{M=7.5,\sigma_v'=1atm} = 0.65\frac{\sigma_v}{\sigma_v'}\frac{a_{\max}}{g}r_d\frac{1}{MSF}\frac{1}{K_\sigma}$$

$$r_d = \exp\left[\alpha(z)+\beta(z)\cdot M\right]$$

$$\alpha(z) = -1.012-1.126\sin\left(\frac{z}{11.73}+5.133\right)$$

$$\beta(z) = 0.106+0.118\sin\left(\frac{z}{11.28}+5.142\right)$$

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma_v'}{P_a}\right) \leq 1.1$$

$$C_\sigma = \frac{1}{37.3-8.27\left(q_{c1Ncs}\right)^{0.264}} \leq 0.3$$

$$MSF = 1+\left(MSF_{max}-1\right)\left(8.64\exp\left(\frac{-M}{4}\right)-1.325\right)$$

$$MSF_{max} = 1.09+\left(\frac{q_{c1Ncs}}{180}\right)^3 \leq 2.2$$

$\sigma_v'$ *and* $\sigma_v$ *at start of earthquake shaking*

$$CRR_{M=7.5,\sigma_v'=1atm} = \exp\left(\frac{q_{c1Ncs}}{113}+\left(\frac{q_{c1Ncs}}{1000}\right)^2-\left(\frac{q_{c1Ncs}}{140}\right)^3+\left(\frac{q_{c1Ncs}}{137}\right)^4-2.80\right)$$

$$q_{c1Ncs} = q_{c1N}+\Delta q_{c1N}$$

$$\Delta q_{c1N} = \left(11.9+\frac{q_{c1N}}{14.6}\right)\exp\left(1.63-\frac{9.7}{FC+2}-\left(\frac{15.7}{FC+2}\right)^2\right)$$

$$q_{c1N} = C_{N}\frac{q_c}{P_a}$$

$$C_{N} = \left(\frac{P_a}{\sigma_v'}\right)^m \leq 1.7$$

$$m = 1.338-0.249\left(q_{c1Ncs}\right)^{0.264} \quad with \quad 0.264 \leq m \leq 0.782$$

$$FC = 80\left(I_C+C_{FC}\right)-137 \quad with \quad 0\% \leq FC \leq 100\%$$

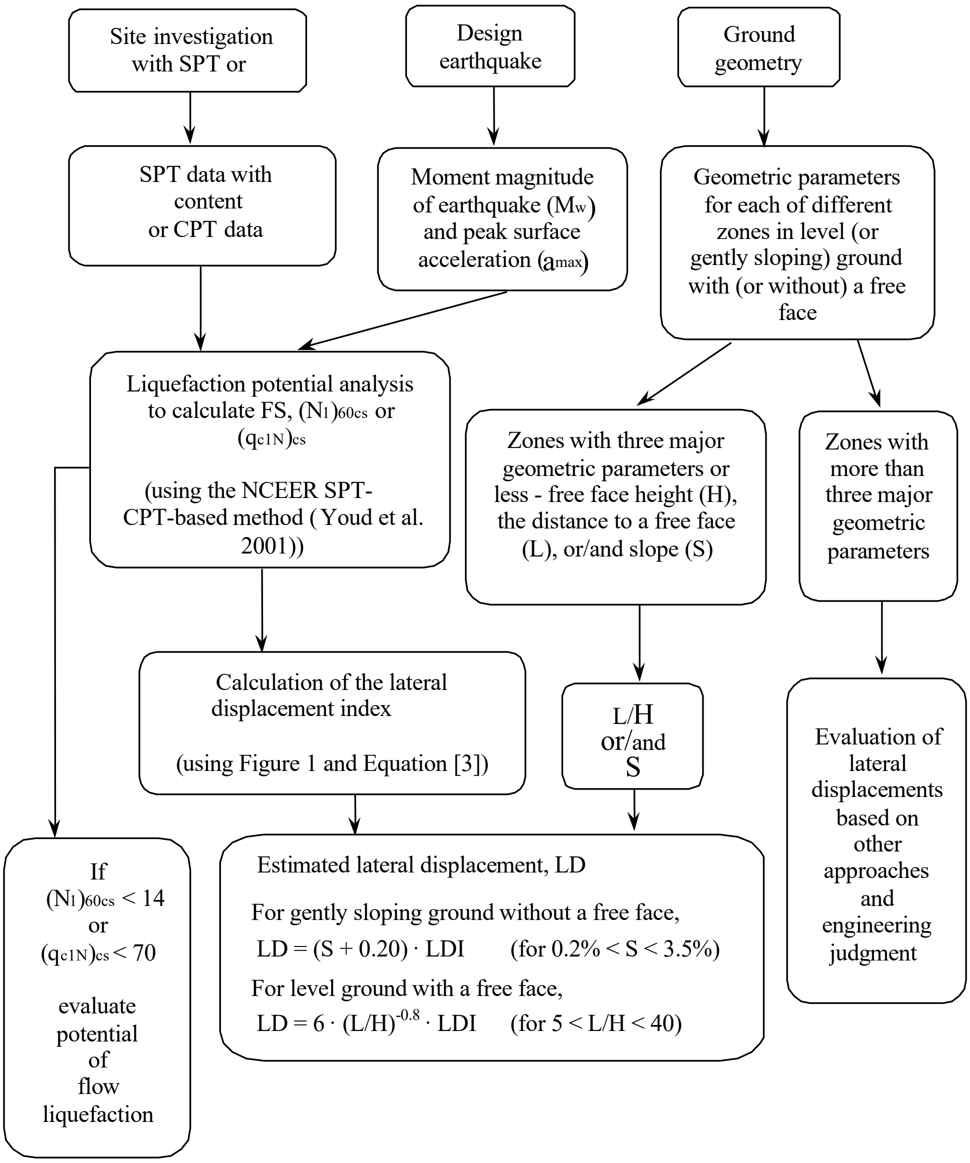
$$I_c = \left[\left(3.47-\log(Q)\right)^2+\left(1.22+\log(F)\right)^2\right]^{0.5}$$

$$Q = \left(\frac{q_c-\sigma_{vc}}{P_a}\right)\left(\frac{P_a}{\sigma_{vc}'}\right)^n \; with\; 0.5 \leq n \leq 1.0 \; per\; Robertson\;\&\;Wride\;(1998)$$

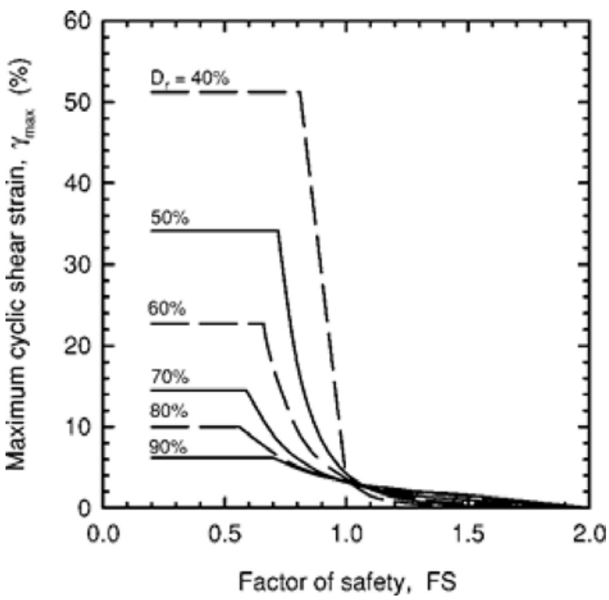
$$F = \left(\frac{f_s}{q_c-\sigma_{vc}}\right)\cdot 100\%$$

$\sigma_v'$ *at time of CPT sounding*

## Procedure for the evaluation of liquefaction-induced lateral spreading displacements




[1] Flow chart illustrating major steps in estimating liquefaction-induced lateral spreading displacements using the proposed approach




[1] **Figure 1**

$$LDI = \int_0^{Z_{max}} \gamma_{max} \, dz$$

[1] **Equation [3]**

[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

## Procedure for the estimation of seismic induced settlements in dry sands

### Average shear stress, $\tau_{av}$

$$\tau_{av} = CSR^{*} \cdot \sigma_{v0}' = 0.65 \cdot \frac{a_{max}}{g} \cdot \sigma_{v0} \cdot \tau_a$$

### Estimate small shear strain modulus, $G_0$

$$G_0 = 0.0188 \cdot \left[10^{(0.55 I_c - 1.68)}\right] \cdot (q_t - \sigma_v)$$

### Estimate shear strain amplitude, $\gamma$ (based on Pradel (1998))

$$\gamma = \left[\frac{1 + \alpha \cdot e^{bR}}{1 + \alpha}\right] \cdot R \cdot 100\ (\%)$$

$$R = \frac{\tau_{av}}{G_0} \text{ (Note } \tau_{av} \text{ and } G_0 \text{ same units)}$$

$$\alpha = 0.0389 \cdot \left(\frac{\sigma_v}{p_a}\right) + 0.124$$

$$b = 6400 \cdot \left(\frac{\sigma_v}{p_a}\right)$$

### Estimate volumetric strain in 15 cycles

$$\varepsilon_{vol(15)} = \gamma \cdot \left[\frac{(N_1)_{60,cs}}{20}\right]^{-1.20}$$

$$(N_1)_{60,cs} = \frac{Q_{tn,cs}}{8.5 \cdot \left[1 - \frac{I_c}{4.6}\right]}$$

### Volumetric strain in design earthquake

$$\Sigma_{vol} = \varepsilon_{vol(15)} \cdot \left[\frac{N_c}{15}\right]^{0.45}$$

$$N_c = (M - 4)^{2.17}$$

### Seismic settlement, s

$$s = 2 \cdot \int_0^{GWT} \Sigma_{vol} \cdot dz$$

Robertson, P.K. and Lisheng, S., 2010, "Estimation of seismic compression in dry soils using the CPT" FIFTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS, Symposium in honor of professor I. M. Idriss, San Diego, CA

## Liquefaction Potential Index (LPI) calculation procedure

Calculation of the Liquefaction Potential Index (LPI) is used to interpret the liquefaction assessment calculations in terms of severity over depth. The calculation procedure is based on the methology developed by Iwasaki (1982) and is adopted by AFPS.

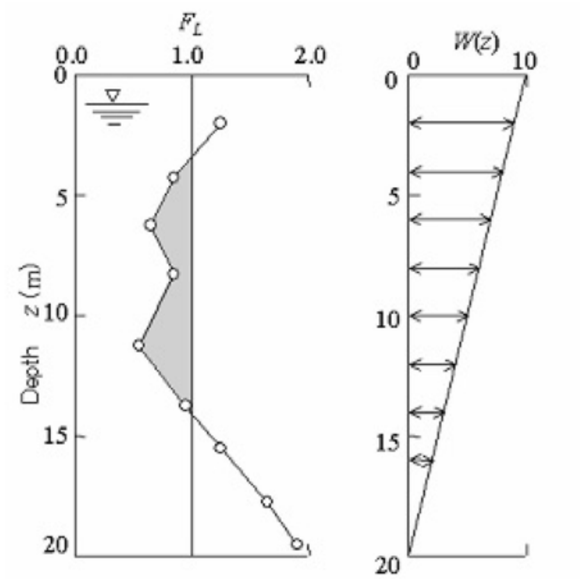
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

$$\textbf{LPI} = \int_0^{20} (10 - 0{,}5_z) \times F_L \times d_z$$

where:
$F_L$ = 1 - F.S. when F.S. less than 1
$F_L$ = 0 when F.S. greater than 1
z depth of measurment in meters

Values of LPI range between zero (0) when no test point is characterized as liquefiable and 100 when all points are characterized as susceptible to liquefaction. Iwasaki proposed four (4) discrete categories based on the numeric value of LPI:

- LPI = 0 : Liquefaction risk is very low
- 0 < LPI <= 5 : Liquefaction risk is low
- 5 < LPI <= 15 : Liquefaction risk is high
- LPI > 15 : Liquefaction risk is very high




**Graphical presentation of the LPI calculation procedure**

## Shear-Induced Building Settlement (Ds) calculation procedure

The shear-induced building settlement (Ds) due to liquefaction below the building can be estimated using the relationship developed by Bray and Macedo (2017):

$$Ln(Ds) = c1 + c2 * LBS + 0.58 * Ln\left(Tanh\left(\frac{HL}{6}\right)\right) +$$
$$4.59 * Ln(Q) - 0.42 * Ln(Q)^2 - 0.02 * B +$$
$$0.84 * Ln(CAVdp) + 0.41 * Ln(Sa1) + \varepsilon$$

where Ds is in the units of mm, c1= -8.35 and c2= 0.072 for LBS ≤ 16, and c1= -7.48 and c2= 0.014 otherwise. Q is the building contact pressure in units of kPa, HL is the cumulative thickness of the liquefiable layers in the units of m, B is the building width in the units of m, CAVdp is a standardized version of the cumulative absolute velocity in the units of g-s, Sa1 is 5%-damped pseudo-acceleration response spectral value at a period of 1 s in the units of g, and ε is a normal random variable with zero mean and 0.50 standard deviation in Ln units. The liquefaction-induced building settlement index (LBS) is:

$$LBS = \sum W * \frac{\varepsilon_{shear}}{z}\, dz$$

where z (m) is the depth measured from the ground surface > 0, W is a foundation-weighting factor wherein W = 0.0 for z less than Df, which is the embedment depth of the foundation, and W = 1.0 otherwise. The shear strain parameter (ε_shear) is the liquefaction-induced free-field shear strain (in %) estimated using Zhang et al. (2004). It is calculated based on the estimated Dr of the liquefied soil layer and the calculated safety factor against liquefaction triggering (FSL).

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# END OF REPORT