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Appendix 3.4A

Geotechnical Engineering Report

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GEOTECHNICAL ENGINEERING REPORT

NEW BATTERY ENERGY STORAGE SYSTEM INSTALLATION
90 MINTO ROAD
WATSONVILLE, CALIFORNIA

by
Haley & Aldrich, Inc.
Walnut Creek, California

for
New Leaf Energy, Inc.
Lowell, Massachusetts

File No. 0210659-004
December 2025

HALEY & ALDRICH, INC.
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December 17, 2025
File No. 0210659-004

New Leaf Energy, Inc.
55 Technology Drive, Suite 102
Lowell, Massachusetts 01851

Attention: William Peregoy, Project Engineer

Subject: Geotechnical Engineering Report
New Battery Energy Storage System Installation
90 Minto Road
Watsonville, California

William Peregoy:

Haley & Aldrich, Inc. is pleased to present the enclosed geotechnical investigation report for the proposed New Leaf Energy, Inc. Battery Energy Storage System (BESS) installation project at 90 Minto Road in Watsonville, California (Site).

We understand the proposed project entails the demolition and clearing of existing orchards and structures and the construction of a new BESS facility. Current plans indicate that the new BESS facility will include a collection substation, numerous energy storage containers, overhead power transmission lines, gravel access roads, and a 7-foot-high chain link perimeter fence. The planned improvements do not include the construction of habitable or occupiable structures. The proposed BESS units will be located within the northern portion of the Site, and the unused portion of the Site will remain essentially vacant. We anticipate earthwork to establish site grades will consist of minor cuts and fills on the order of 1 to 3 feet (ft). Retaining walls are not anticipated. We understand that the battery containers will exert bearing pressures typically less than 750 pounds per square foot (psf) on the subgrade, assuming that shallow concrete footings are used as foundations.

Near-surface soils at the Site (those within the upper approximately 5 ft below ground surface [bgs]) generally consist of sandy clay and sandy silt soils that have been disturbed by agricultural activities and are expected to have low to moderate expansive potential. According to cone penetration test (CPT) soil behavior type correlations, subsurface conditions at greater depths consist of alluvial clayey sand extending to a depth of approximately 18 to 27 ft bgs; this sand layer is generally underlain by thick deposits of clay to a depth of at least 100 ft bgs. The depth to groundwater at the time of our exploration program ranged from about 5 to 16 ft bgs, as inferred from CPT pore pressure dissipation testing.

Based on the results of our subsurface investigation, we conclude that the proposed Site development is geotechnically feasible. The primary geotechnical issues that should be addressed during the design and construction of this project include the potential for fault surface rupture, very strong seismic shaking, seismic hazards including liquefaction and lateral spreading, earthwork considerations concerning potentially loose near-surface agricultural fill material, the presence of near-surface moderately expansive clay soils, and the selection of an appropriate foundation system for support of the proposed structures and energy storage improvements. We recommend that the proposed energy storage units and improvements be supported on shallow spread foundations or mat foundations bearing on compacted subgrade. Our recommendations regarding foundations, site grading, and other aspects of this project are presented in this geotechnical investigation report.

We appreciate the opportunity to provide our geotechnical services on this project. If you have any questions or require additional information, please contact the undersigned.

Sincerely yours,
HALEY & ALDRICH, INC.

Matthew Rogers

Micah D. Hintz, P.E., G.E.
Geotechnical Engineer

Matthew Rogers, P.E., G.E.
Principal Geotechnical Engineer

Enclosures

https://haleyaldrich.sharepoint.com/sites/NewLeafEnergyInc/Shared Documents/0210659.New Leaf 90 Minto Rd Watsonville/004/Deliverables/Geotech Rpt/Final/2025_1217_HAI_WatsonvilleGeotechReport_F.docx

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SIGNATURE PAGE FOR

REPORT ON
NEW BATTERY ENERGY STORAGE SYSTEM INSTALLATION
90 MINTO ROAD
WATSONVILLE, CALIFORNIA

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LOWELL, MASSACHUSETTS

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1. Introduction

This report presents the results of our geotechnical investigation of the proposed Battery Energy Storage System (BESS) project located at 90 Minto Road in Watsonville, California (Site). The Site is comprised of two parcels (APN 05110177 and 05110178). The approximate location of the Site is shown on Figure 1. The Site coordinates are approximately latitude 36.950874° and longitude -121.758950°.

1.1 BACKGROUND AND EXISTING SITE CONDITIONS

The 37-acre Site is bound by Minto Road, agricultural fields, and a Pacific Gas & Electric energy substation to the west, and agricultural fields to the north, south, and east. The Site is occupied by a barn and three sheds on the central west side of the property, with the remainder of the property in use as an orchard. Minto Road crosses through the northern portion of the Site in a northwest-southeast orientation, and unpaved access roads are present at most property boundaries.

Topographic contour data available from Santa Cruz County shows that ground surface elevations at the Site generally vary between about 94 and 122 ft (NAVD 88). The Site and existing conditions are presented on Figure 2.

1.2 PROJECT DESCRIPTION

Our understanding of the project is based on conversations with you and a site plan titled, “Seahawk-200 MWAC - BESS Project,” prepared by New Leaf Energy, Inc. and dated 11 September 2024. Based on that information, we understand the proposed project includes demolition and clearing of existing Site improvements (i.e., shed structures and orchards) and construction of a new BESS facility. The facility will include a collector substation in the northwest area of the Site, and numerous BESS located nearby in the Site’s northern and central areas. The planned improvements do not include the construction of habitable or occupiable structures. Earthwork to establish site grades is expected to consist of minor cuts and fills on the order of 1 to 4 ft. Retaining walls are not anticipated. Based on our previous experience on similar projects, we assume the following regarding the energy storage batteries:

- Weight of each BESS unit will be about 18 to 20 kips.
- Footprint area of each BESS unit pad will be about 7- to 9-ft wide by 7- to 9-ft long.
- A detention basin may be required by the county for stormwater management.

If the project differs significantly from this description, we should be consulted to review our scope for applicability.

2. Scope of Services

This geotechnical investigation was performed to obtain information on subsurface conditions at the Site and develop design recommendations for the subject project. The scope of the investigation as outlined in our proposal dated 9 October 2024 (authorized on 10 October 2024) included the following:

- Reviewed readily available geotechnical and geologic reports including California Geological Survey (CGS) published maps and reports available on the California State Water Resources Control Board GeoTracker website (2024);
- Performed pre-field activities, including preparing a health and safety plan, performing a Site reconnaissance visit to mark proposed exploration locations, and acquiring a well permit for explorations from Santa Cruz County as required by law;
- Performed a geotechnical field investigation that included:
  - Advancing six CPTs and performing shear wave velocity testing at one CPT location to determine the seismic Site Class per American Society of Civil Engineers (ASCE) 7-16;
  - Drilling one mud rotary boring to a depth of 50 ft to and three shallow hand auger borings to visually observe and classify soils and collect samples for laboratory testing;
  - Performing porewater pressure dissipation (PPD) testing to infer the depth of the groundwater table at the Site;
  - Collecting field electrical resistivity measurements using the Wenner Four-Point method along one test transect;
  - Collecting and submitting one sample for laboratory thermal resistivity testing;
  - Conducting one borehole infiltration test; and
  - Performing geotechnical laboratory testing on select soil samples for geotechnical classification.
- Performed engineering analyses and prepared this geotechnical report presenting our conclusions and recommendations regarding:
  - Soil and groundwater conditions at the Site;
  - Site seismicity and seismic hazards including liquefaction potential;
  - Seismic design parameters in accordance with the 2022 California Building Code (CBC);
  - Expansion potential of the onsite soils (if present);
  - Settlement anticipated due to the presence of the new and existing fill materials;
  - Foundation type(s) and design capacities for various foundation systems capable of supporting the planned BESS improvements;
  - Earthwork including excavation, backfill, moisture conditioning and compaction requirements, and import fill material;
  - Design and construction impacts from groundwater;
  - Support of concrete slabs-on-grade;
  - Gravel access roads;
  - Field infiltration criteria for stormwater management (detention basins);
  - Corrosivity of onsite soils; and
  - Other construction considerations.

3. Field Exploration

Haley & Aldrich performed subsurface explorations at the Site on 1 November 2024. The approximate locations and designations of the subsurface explorations performed are shown on *Figure 2 - Site and Exploration Plan*. A summary of the preliminary exploration program is detailed below:

- Six CPTs were performed to evaluate subsurface conditions across the Site. One CPT (designated CPT-1) was advanced to a depth of approximately 100 ft bgs, and five CPTs (designated CPT-2 through CPT-6) were advanced to depths between approximately 30.6 and 34.4 ft bgs. The CPT explorations included PPD tests to interpret the approximate depth to groundwater;
- One mud rotary boring (designated B-1) was drilled to a depth of approximately 50 ft bgs to evaluate subsurface conditions and collect physical soil samples for classification and testing.
- Shear wave velocity testing was performed to a depth of 100 ft bgs at CPT-1 to inform Site Class determination;
- Shallow hand-auger borings were drilled to collect disturbed samples of near-surface soils;
- Field electrical resistivity measurements were collected using the Wenner Four-Point method along one transect;
- One field infiltration test (IT-1) was performed in the western portion of the Site; and
- Samples collected at the Site were submitted to geotechnical laboratory testing, including testing for thermal resistivity, electrical resistivity, and corrosivity properties.

Prior to performing our field investigation, we contacted Underground Service Alert (USA) and acquired a soil boring construction/destruction permit for explorations from Santa Cruz County Environmental Health as required by law.

3.1 CONE PENETRATION TESTS

Middle Earth Geo Testing, Inc. (Middle Earth) of Hayward, California, performed the CPT explorations on November 1, 2024. The CPTs consisted of hydraulically pushing a 1.75-inch diameter, cone-tipped probe into the soil using a rig with a push capacity of 25 tons. The cone tip measures tip resistance and the friction sleeve behind the cone tip measures frictional resistance. Electrical strain gauges within the cone continuously measured soil parameters during the entire depth the cone was advanced. Soil data, including tip resistance and frictional resistance, were recorded and then processed to provide information for use in our geotechnical engineering analyses. PPD testing was performed in the CPT explorations (CPT-1 through CPT-6) to measure hydrostatic water pressures and to infer the approximate depths to groundwater. Once completed, the CPT holes were backfilled with grout in accordance with Santa Cruz County Environmental Health permit requirements.

To conduct the seismic shear wave velocity testing, advancement of the cone was halted at select test depths and the rods were decoupled from the rig. An automatic hammer was triggered to send a shear wave into the soil. The distance from the source to the cone was calculated by knowing the total depth of the cone and the horizontal offset distance between the source and the cone. Seismic shear wave tests were performed at every 5-foot interval to a depth of about 100 ft in CPT-1.

The stratigraphic interpretation of the CPT data was performed based on relationships between cone bearing and sleeve friction versus penetration depth. The friction ratio (defined as sleeve friction divided by cone bearing resistance) is a calculated parameter used to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone bearing, and generate large excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate small excess pore water pressures. The interpretation of soil properties from the cone data has been carried out using recent correlations developed by Robertson and Cabal (2010). The CPT log and shear wave velocity tests, showing tip resistance and friction ratio by depth and interpreted soil classifications and strengths, are presented in Appendix A.

3.2 MUD ROTARY BORING

Gregg Drilling, LLC (Gregg) of Benicia, California, performed a single mud rotary boring on December 2, 2025. The boring was drilled under the direction of our field geologist, who logged the hole and collected samples for soil classification and laboratory testing purposes. Soil samples were obtained at discrete locations during drilling using a standard penetration test (SPT) sampler. The locations where each sample was collected are shown on the boring log. The unlined SPT sampler has a 2-inch outside diameter and a 1.38-inch inside diameter. SPT soil samples were collected by driving the sampler for a distance of 18 inches or to penetration refusal, whichever was encountered first, using a 140-pound, aboveground automatic hammer falling approximately 30 inches. Uncorrected blow counts were recorded for each 6-inch-long interval of sampler penetration and are presented on the boring logs in Appendix A. After each sample was withdrawn from the boring, the samples were removed, examined for logging purposes, labeled, and sealed to retain the natural moisture content.

Once completed, the boring was backfilled with grout in accordance with Santa Cruz County Environmental Health permit requirements.

3.3 HAND-AUGER BORINGS

We performed a total of four hand-auger borings at the site to depths of 3 to 5 ft bgs. One of these borings was performed at the location of our infiltration test IT-1 (discussed in *Section 3.4 - Boring Infiltration Test*). The upper 5 ft of soil were drilled via hand auger at boring B-1 before transitioning to mud rotary drilling, with findings shown on the boring log for B-1. Additionally, two hand-auger borings (HB-1 and HB-2) were performed in the northeastern portion of the site to examine shallow subsurface conditions and collect samples for Atterberg limits testing. Logs for hand-auger borings HB-1 and HB-2 are presented in Appendix A.

3.4 BORING INFILTRATION TEST

One small boring infiltration test was performed at boring location IT-1, located in the western portion of the Site. The test location was drilled to a depth of approximately 3 ft bgs using a 4-inch-diameter hand auger. A 2-inch-diameter slotted PVC pipe was installed in the center of the boring to the full depth and the area around the pipe was backfilled with pea gravel to the ground surface. The test apparatus was presoaked with water for several hours prior to testing. The drop in water level was recorded in 30-minute intervals before refilling the pipe to a set level, to approximate falling-head test conditions. The results of the infiltration tests are presented in Appendix C.

3.5 IN SITU RESISTIVITY TESTS

One set of *in situ* electrical resistivity measurements was collected using the Wenner 4-point method at the location shown on Figure 3. Measurements were collected at pin spacings of 2.5, 5, 7.5, 10, 15, and 20 ft, with pins generally driven to a depth equal to 1/20th of the pin spacing. The resistivity measurements were collected using a Miller 400D S.R.M. Digital Soil Resistivity Meter. The resistivity test results are presented in *Table 1 - Wenner 4-Point Resistivity Test Data*.

Table 1. Wenner 4-Point Resistivity Test Data

Bearing	Probe Spacing (ft)	Probe Depth (inches)	Resistance (kΩ)	Resistivity, ρ (Ωm)[1]
East-West	2.5	1.5	7.78	37.4
East-West	5.0	3.0	2.22	21.4
East-West	7.5	4.5	1.52	21.9
East-West	10.0	6.0	1.26	24.2
East-West	15.0	9.0	0.91	26.3
East-West	20.0	12.0	0.69	26.5

Note:

[1] *Values were calculated based on Resistance (Ω) values collected during field activities.*

3.6 GEOTECHNICAL LABORATORY TESTING

Soil samples collected from the Site were re-examined at our office to confirm field classifications and to select representative samples for geotechnical testing. Laboratory testing was performed by Haley & Aldrich as well as several independent laboratories, with test programs described as follows:

- Inspection Services, Inc. of Berkeley, California, and the Haley & Aldrich geotechnical lab of Portland, Oregon performed geotechnical laboratory testing including Atterberg limits analysis and modified Proctor testing to support thermal resistivity testing.

CERCO Analytical, Inc. (CERCO) of Concord, California, tested one sample for corrosion properties, including pH, redox potential, electrical resistivity, and chloride and sulfate content.

- Geotherm USA, LLC of Cypress, Texas, subjected one bulk sample from the near-surface soils at the Site to thermal resistivity analysis.

The geotechnical laboratory test results, corrosivity test results, and thermal resistivity test results are presented in Appendix B.

4. Geology and Seismicity

4.1 REGIONAL GEOLOGY

The project Site lies within the Santa Cruz Mountains, within the Coast Ranges geomorphic province of California. This province is characterized by northwest-southeast trending mountain ranges and intervening valleys. The Santa Cruz Mountains mark a regional uplift boundary southwest of the San Andreas Fault. The southwest flank of the central Santa Cruz Mountains is mostly underlain by a large structural unit known as the Salinian Block. The Salinian Block is composed of Mesozoic age granitic and metamorphic and separated from basement rock belonging to the Franciscan Complex to the northeast and southwest by the San Andreas and Nacimiento-San Gregorio-Sur Faults, respectively.

4.2 SITE GEOLOGY

The geology of the Site is mapped in the United States Geological Survey (USGS), by Brabb (1989) and Graymer (et al., 2006). The surface geology is mapped as Terrace Deposits of Watsonville, Fluvial facies (Pleistocene), and Terrace Deposits of Watsonville, Alluvial fan facies. The Fluvial facies unit is described by USGS as deposits of "Semiconsolidated, moderately to poorly sorted silt, sand, silty clay, and gravel. May be more than 200 ft thick. Gravel, approximately 50 ft thick, is generally present 50 ft below surface of the deposit and is both a local aquifer and a significant source of gravel. Upper 5 to 15 ft of the unit is moderately indurated owing to clay and iron oxide cementation in a weathered zone."

The Alluvial fan facies are described as "Semiconsolidated, moderately to poorly sorted, discontinuous layers of silty clay, silt, sand, and gravel. Deposited by streams, sheet flow, and debris flow on alluvial fans adjacent to Santa Cruz Mountains. Thickness variable; locally may be more than 50 ft thick."

4.3 SITE SOILS

The near-surface soils at the Site are mapped by the U.S. Department of Agriculture (USDA) as presented on the *Web Soil Survey* (USDA, 2024). USDA mapping indicates Site soils in the northern half of the lot primarily consist of Watsonville loam, 2- to 15-percent slopes. The Watsonville loam is described as somewhat poorly drained, with a typical profile including 1.5 ft of loam overlaying 1.5 ft of clay, followed by sandy clay loam extending to a depth of 5 ft. Hydraulic capacity (permeability) in the most limiting layer is mapped as 0.00 to 0.06 inches per hour. The southern half of the property is mapped as Pinto loam, 2- to 9-percent slopes. The Pinto loam is described as moderately well drained, with a typical profile of 1.5 ft of loam overlaying 4 ft of sandy clay loam. The hydraulic capacity in the most limiting layer is mapped as 0.06 to 0.20 inches per hour (USDA, 2024; Haley & Aldrich, 2024a).

4.4 REGIONAL SEISMICITY

The project Site is located within the greater San Francisco Bay Area, which is recognized as one of the more seismically active regions of California. The Site is in a seismically active area between two major fault zones: the San Andreas to the northeast and the San Gregorio offshore to the west (Working Group on California Earthquake Probabilities, 2003). The Zayante-Vergeles fault zone intersects the Site (Figure 3).

Numerous potentially active faults in the vicinity of the Site have been mapped by the USGS within their Quaternary Fault and Fold Database of the United States (USGS, 2024a). The mapped active faults within 50 kilometers (km) of the Site, along with their closest distance to the Site, direction from the Site, and estimated maximum Moment magnitude, M_w, are summarized in Haley & Aldrich (2024a). The Zayante-Vergeles fault zone is the closest mapped active fault, which is understood to potentially transect the Site (Figure 3).

The Site is located near or within the Zayante-Vergeles fault zone. The southern extension of the Zayante Fault, known as the Vergeles Fault, merges with the San Andreas Fault south of the city of Hollister in San Benito County. The 1989 Loma Prieta earthquake occurred as a result of seismicity along the San Andreas Fault.

The USGS Quaternary Fault and Fold Database shows the Zayante Fault positioned outside of Site limits and is approximately 200 ft southwest of the Zayante Fault and 4.3 km southwest of the San Andreas Fault. The subject property has been identified within the Santa Cruz County and State of California designated fault zones associated with the active Zayante Fault, as shown on Figure 3. We have reviewed mapped alignments of the Zayante-Vergeles fault zone presented by the USGS, the CGS, and Santa Cruz County (County). The USGS/CGS fault trace, shown in orange on Figure 3, sits roughly 200 ft away from the Site's eastern edge, at the base of the hillslope where it comes in contact with the ephemeral College Lake. The County's fault trace is shown in pink on Figure 3, with one trace within College Lake, and a second within and parallel to the Site's eastern edge. The associated zones of required study for the CGS/USGS and the County overlap, with the County zone covering the entire Site and the USGS/State zone covering just the eastern edge of the Site.

5. Subsurface Soil and Groundwater Conditions

5.1 SUBSURFACE CONDITIONS

Our geotechnical explorations indicate that the Site subsurface conditions consist of alluvial deposits to a depth of at least 100 ft bgs. Near-surface soil conditions were generally logged as consisting of sandy clays, sandy silts, and fat clays. These soils were disturbed by agricultural activities in the upper approximately 1 ft. Atterberg limits testing on these near-surface soils resulted in a plasticity index of 16 and a liquid limit of 30.

At depths greater than 5 ft bgs, each of our explorations encountered varying amounts of sand, silt, and clay, either by direct examination of soil samples (from our boring) or by correlations to CPT soil behavior type correlations. Conditions at individual exploration locations are described as follows:

- B-1 encountered a mix of lean clay, fat clay, clayey sand, and sandy lean clay within the upper 6 ft bgs. From 6 to 15 ft bgs, soils consisted of dense silty sand, well-graded sand with gravel, and poorly graded sand with silt. Medium dense clayey sand was present between 15 and 18 ft bgs before transitioning to stiff to very stiff fat clay (sample at 20 to 21.5 ft bgs has a plasticity index of 46). At 30 ft bgs, the lean clay transitioned to very stiff fat clay, which persisted to the maximum explored depth of 51.5 ft bgs.
- CPT-1 encountered medium dense to dense silty sand, sandy silt, and clayey sand to a depth of around 27 ft bgs. At depths greater than 27 ft bgs, the soil profile generally consisted of stiff to very stiff clays and silts. These fine-grained soils were interrupted by an approximately 5-ft-thick layer of dense to very dense sands beginning at a depth of about 88 ft bgs.
- CPT-2, CPT-3, CPT-4, CPT-5, and CPT-6 encountered loose to medium dense to dense silty sand and clayey sands to depths of between approximately 14 and 24 ft bgs. Below these sands, stiff to very stiff clays were encountered to the maximum explored depths of about 30 to 34 ft bgs at these locations.

5.2 SHEAR WAVE VELOCITY CONDITIONS

The weighted average shear wave velocity over the upper 100 ft (30 meters) is known as Vs_{30}, and this value is important for determining the seismic site class for a site per ASCE 7-16. Shear wave velocity profiles of subsurface alluvial soils at the Site were recorded utilizing seismic CPT shear wave testing at CPT-1, which was terminated at about 100 ft bgs. The test results a Site Vs_{30} value on the order of 830 ft per second (ft/s; 253 meters per second [m/s]).

5.3 GROUNDWATER CONDITIONS

Interpretations of PPD tests performed on November 1, 2024 indicated a typical depth to groundwater ranging from about 10.2 to 16.0 ft bgs, as shown in *Table 2 - Porewater Pressure Dissipation Tests*, below.

Table 2. Porewater Pressure Dissipation Tests		
CPT ID	**PPD Test Depth (ft, bgs)**	**Inferred Groundwater Depth (ft, bgs)**
CPT-1	18.7	10.2
CPT-2	21.8	5.2
CPT-3	18.5	13.7
CPT-5	15.1	11.6
CPT-6	19.0	16.0

PPD testing does not provide a direct measurement of depth to groundwater, but provides data on the hydraulic head pressure acting on the CPT probe at a given test depth, from which depth to groundwater can be inferred. PPD tests provide the most accurate estimations of local groundwater table elevations when performed in subsurface profiles composed primarily of free-draining sands and gravels; variable subsurface stratigraphy, such as the presence of alternating coarse- and fine-grained soils layers, which were identified at the subject Site, can produce unreliable PPD readings. The PPD test at CPT-2 (location of shallowest inferred groundwater depth) was performed within a 2-foot-thick sand layer confined between clay layers. PPD tests at other locations were performed within thicker sand layers with less potential for confinement impacting water pressure readings. Therefore, we conclude the PPD test reading at CPT-2 is likely an unreliable outlier and/or represents a localized condition that is not typically present Site-wide.

We have also reviewed monitoring well data from several historical monitoring wells in the greater area located between 600 to 3,000 ft from the Site. Monitoring well data collected in the area between 1954 and 2022 indicated depths to groundwater ranging from 12 to 106 ft bgs (California Department of Water Resources, 2024).

Groundwater levels at the Site are expected to be influenced by the nearby College Lake. This ephemeral lake is located approximately 300 ft to the northeast of the Site.

6. Discussion and Conclusions

Based on our review of Site data and the results of our engineering analyses, we conclude the primary geotechnical issues associated with this project include:

- Potential for onsite surface rupture of the Zayante Fault;
- Very strong seismic shaking;
- Liquefaction;
- Near-surface materials disturbed by historical agricultural activities;
- Moderately to highly expansive clay soils throughout the near-surface subgrade; and
- Selection of appropriate foundation system(s) for support of the proposed energy storage structures.

These and other geotechnical issues are discussed in subsequent sections of this report.

6.1 FAULT SURFACE RUPTURE

Historically, ground surface displacements closely follow the trace of geologically young faults. The Site is within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act (California Department of Conservation, 2024), and known potentially active faults exist on the Site as stated in *Section 4.4 - Regional Seismicity*. Haley & Aldrich completed an initial geologic hazard assessment in 2024 addressing the presence of this fault, in which we recommended that a fault investigation report be prepared to more definitively determine the risk for fault surface rupture (Haley & Aldrich, 2024b).

Subsequent to our initial geologic hazard assessment, a geologic fault investigation was performed by Bayside Geology (Bayside) of Santa Cruz, California, with findings and conclusions summarized in a report titled, “Geologic Fault Investigation, New Leaf Energy Inc. Property, 90 Minto Road, Watsonville, California, Santa Cruz County APN 051-101-77 & 78,” dated August 1, 2025. As part of this study, Bayside excavated 1,300 ft of exploratory trench distributed across five separate trenches. Trenches were excavated to a maximum depth of about 7 ft bgs, exposing five distinct soil units. Bayside stated in their report that, “Overall, the exploratory trenching revealed no indication of tectonic faulting of any kind; of offset or truncation of units, no soil mixing or colluvial wedges, no continuous (through going) shear planes, no alignment of gravels, et cetera.” Additionally, Bayside concluded, “The proposed development will be subject to an ‘ordinary’ level of risk,” indicating further that, with respect to seismicity, the development will resist minor earthquakes without damage, resist moderate earthquakes with cosmetic but non-structural damage, and resist major earthquakes without collapse, but with structural damage.

Based primarily on the results of the fault investigation by Bayside Geology, we consider the potential for onsite fault rupture to be low.

6.2 SEISMIC HAZARDS

During a major earthquake, very strong ground shaking has the potential to occur at the Site. Shaking during an earthquake can result in ground failure, such as that associated with soil liquefaction, lateral spreading, and cyclic densification. Haley & Aldrich's assessment of these potential seismic hazards is presented in the following sections.

6.2.1 Site Seismicity

A seismic hazard analysis was performed using the USGS Unified Hazard Tool website (USGS, 2024b). Our deaggregation analysis utilized the USGS Dynamic Conterminous U.S. 2014 (v.4.2) edition. For our analyses, we evaluated the Site as Site Class D (correlating to a Vs_{30} of 850 ft/s (259 m/s)), which is appropriate based on the Vs_{30} of 830 ft/s (253 m/s) measured at CPT-1, as discussed in *Section 5.2 - Shear Wave Velocity Conditions*. However, because of the liquefaction hazard at the Site (see *Section 6.2.2 - Soil Liquefaction and Associated Hazards*), the Site is designated Site Class F per ASCE 7-16. The planned improvements are not expected to have a natural structural period in excess of 0.5 seconds; therefore, ASCE 7-16 permits design per Site Class D.

Based on the seismicity of faults that may impact the Site and the results of the deaggregation analysis, a design earthquake with an M_w of 7.23 was selected for the seismic hazard evaluation. The peak horizontal ground acceleration for the Site, which is based on the Maximum Considered Earthquake (MCE) with a return interval of 2,475 years, or a 2 percent probability of exceedance in 50 years, is 1.19 g. The risk-based site-modified peak ground acceleration (PGA_M) for the Site is 1.03 g; this value was computed based on procedures outlined in ASCE 7-16 (ASCE, 2017) and is based on Site Class D conditions.

Recommended code-based seismic parameters for the design of the proposed structures in conformance with the 2022 CBC (California Building Standards Commission, 2022) and ASCE 7-16 are presented in Table 3.

Table 3. Seismic Design Values for Site Class D Conditions

Seismic Parameter	Design Value
Site Class	D
Risk Category	III
MCE_R Ground Motion (Period = 0.2 seconds), S_s	2.249 g
MCE_R Ground Motion (Period = 1.0 seconds), S_1	0.859 g
Peak Ground Acceleration, PGA	0.933 g
Site Amplification Factor at 0.2 seconds, F_a	1.0
Site Amplification Factor at 1.0 seconds, F_v	--[4]
Site Amplification Factor for PGA, F_{PGA}	1.1
Site-Modified Peak Ground Acceleration, PGA_M	1.026 g
Site-Modified Spectral Acceleration Value at 0.2 seconds, S_{MS}	2.249 g
Site-Modified Spectral Acceleration Value at 1.0 seconds, S_{M1}	--[4]
Design Spectral Acceleration at 0.2 seconds, S_{DS}	1.499 g
Design Spectral Acceleration at 1.0 seconds, S_{D1}	--[4]

Notes:

1. *MCE_R = Risk-targeted maximum considered earthquake*
2. *g = acceleration of gravity*
3. *Design values are based on a site located at latitude / longitude = 36.949725 / -121.757284°.*
4. *Values for F_v, S_{M1}, and S_{D1} are undefined for Site Class D.*

6.2.2 Soil Liquefaction and Associated Hazards

Liquefaction is the process in which saturated, cohesionless soil experiences a temporary loss of strength given the buildup of excess pore water pressure during cyclic loading resulting from earthquake ground motions. The type of soils most susceptible to liquefaction are loose, clean, saturated, uniformly graded sand and silt that have low clay content. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures, and sand boils are evidence of liquefaction.

The County of Santa Cruz has prepared liquefaction hazard maps for the entire county and determined that the majority of the Site is within a zone of “Liquefaction Hazard D - Low,” indicating low liquefaction hazard. This includes the portions of the Site currently intended for development. An approximately 100- to 200-foot-wide strip of land bordering the southwestern Site boundary is mapped by the County as “Liquefaction Hazard B - High.” The limits of mapped liquefaction zones are presented on *Figure 4 - Mapped Liquefaction Hazard*.

We evaluated the potential for soil liquefaction at the Site by performing analyses in accordance with the methodology by Boulanger and Idriss (2014) for CPT and boring explorations. Our evaluation of explorations was from our investigations on November 1, 2024, and December 2, 2025. Our liquefaction analyses were performed using data from CPT explorations CPT-1 through CPT-6 and boring B-1. The parameters used in the liquefaction evaluation are shown in Table 4. A design depth to groundwater of 5 ft was used for these analyses.

Table 4. Values Used in Liquefaction Evaluation

Liquefaction Evaluation Parameter	Value
Depth to Groundwater, Current (ft, bgs)	10 to 16
Depth to Groundwater, during Design Earthquake (ft, bgs)	5
Peak Ground Acceleration	1.03g
Predominant Earthquake Moment Magnitude, M_w	7.23
Factor of Safety (FS) for Liquefaction Triggering	1.3

Based on the level of expected ground shaking, Site conditions, and seasonal groundwater, we anticipate liquefaction triggers near the onset of ground shaking within sand layers at depths ranging between 5 and 27 ft bgs.

Post-liquefaction settlement results from the densification of liquefiable sandy soils following an earthquake. Our analyses indicate that approximately ¾ to 2¾ inches of seismic-induced ground settlement may occur at the Site, as summarized in *Table 5 - Estimated Liquefaction-Induced Settlement*.

Table 5. Estimated Liquefaction-Induced Settlement

Exploration ID	Estimated Settlement (inches)
CPT-1	2.8
CPT-2	2.2
CPT-3	1.2
CPT-4	1.5
CPT-5	0.8
CPT-6	0.9
B-1	0.6

Plots depicting relationships of depth versus an FS against liquefaction, vertical settlements, and lateral displacements at each exploration location are presented in *Appendix D - Liquefaction*.

6.2.3 Global Stability and Lateral Spreading

Lateral spreading occurs when large blocks of ground are displaced down gentle slopes or toward the free face of river channels as a result of earthquake-induced inertial forces acting on the soil mass. Initiation of lateral spreading is often made worse when the soils within and beneath the soil mass liquefy or soften as a result of the shaking. Lateral spreading deformations can be experienced relatively far from a free face. Lateral spreading occurring within zones of deep or shallow foundations is destructive and poses a significant risk to the structures the foundations support.

We conducted analyses using the two-dimensional commercial code Slide2 by RocScience to evaluate the lateral stability of the Site under seismic conditions. The Slide program performs a two-dimensional limit equilibrium analysis to analyze slope stability and to determine an FS against global failure. For modeling of seismic conditions, we coupled earthquake shaking with residual strengths in liquefiable soils due to extreme and long-duration ground shaking anticipated at the Site under design earthquake ground motions. Analyses were performed on Section A-A', the limits of which are shown on *Figure 2 - Site and Exploration Plan*. Subsurface conditions and slope stability analyses for Section A-A' are included in *Appendix E - Slope Stability*.

Initial, coupled, and pseudostatic analyses were conducted using a horizontal seismic coefficient (kh or keq) of 0.27 to evaluate the potential for seismic displacements of about 6 inches or less. This kh value was determined in conformance with Santa Cruz County's *Guidelines for Geotechnical Investigation Reports* (County of Santa Cruz, 2023), using Figure 1 of SP117A (CGS, 2008) and substituting the parameter "MHAr" with the Site $PGA_M/1.5$. Values used to determine this kh value are presented in *Table 6 - Horizontal Seismic Coefficient*.

Table 6. Horizontal Seismic Coefficient

Parameter	Value
PGA_M	1.026 g
$PGA_M/1.5$	0.684 g
feq	0.40
kh or keq	0.27

Pseudostatic slope stability analyses for this preliminary study were performed using soil parameters inferred from the analysis of CPT data. The potentially liquefiable sands were assigned undrained shear strengths ranging from 300 to 1,150 pounds per square foot (psf) to model residual strength, with the assumption that the liquefaction of these soils is coupled with seismic shaking. A graphical plot of residual strength values (overall average of six CPTs and design values) used in design is presented in Appendix D.

Pseudostatic analysis using the kh value derived per Santa Cruz County guidelines results in a critical FS of over 1.4. However, for conservatism and given the fact that the development will likely be classified Risk Category III, we also analyzed seismic stability using a kh value equal to $1/2PGA_M$, which resulted in a factor of safety of approximately 0.8, indicating the potential for seismic instability. We then performed a supplemental analysis to calculate the seismic yield acceleration, resulting in a yield acceleration of 0.41. The results of this analysis are included in *Appendix E - Slope Stability* as shown on Figures E1 and E2.

We performed several supplemental calculations to estimate the level of seismic-induced lateral displacements at the Site, including lateral displacement index (LDI) calculation and seismic slope displacement analysis per Bray and Macedo (2019).

Estimations of LDI were performed for each exploration location using the software package CLiq (Geologismiki, 2024). For LDI calculations, we used a design free face height of 50 ft and a design length to the free face of 650 ft, based on the location and elevation of the proposed Site improvements relative to College Lake. These simplified analyses indicate that lateral displacements ranging from about 5 to 19 inches are possible as a result of design-level seismic shaking at the Site, with an average overall predicted lateral spread displacement of approximately 10 inches.

Following LDI calculations, we conducted seismic displacement analyses following the procedures of Bray and Macedo (2019) for crustal events. This procedure requires that the critical seismic yield coefficient (ky) for a slope be calculated using limit-equilibrium software, and that earthquake characteristics (magnitude and spectral acceleration at a given period), slope geometry (height and shape), and shear wave velocity profile be determined and input into the calculations. The analysis was performed using the “PulsesD100” equations, which are conservative and suitable for sites with near-fault effects. A peak ground velocity of 54 centimeters per second was used based on a M7.0 scenario earthquake on the Zayante-Vergeles Fault, prepared by USGS (USGS, 2013). As shown on Figure E3, the analysis results in a critical ky value along Section A-A’ of 0.41. Based on our analyses, we estimate that the design earthquake may induce approximately 5 inches of lateral displacement on average at the proposed Site; there is an 84-percent probability that displacements will be less than about 9 inches.

The findings from the Bray and Macedo (2019) analysis are on the same order of magnitude as those estimated by the LDI calculations, equating to an estimated lateral spread displacement at the site of 10 inches or less under design-level seismic shaking. We reiterate that the pseudostatic analysis performed using the kh value for displacements of 6 inches or less as defined in SP117A resulted in a factor of safety greater than 1.4, suggesting seismic displacements significantly lower than 6 inches.

Based on these findings, we conclude that the potential for lateral spread at the Site is high, though lateral spread induced displacements will be limited to 10 inches or less.

6.2.4 Cyclic Densification

Seismically induced compaction or densification of non-saturated granular soil (such as sand above the groundwater table seasonally) due to earthquake vibrations can result in settlement of the ground surface. Unsaturated near-surface soils are generally composed of sandy clay soils, which are expected to experience negligible seismic-induced settlement. We find the potential for cyclic densification at the Site to be very low.

6.3 EXPANSION POTENTIAL

Expansive soils are characterized by their ability to undergo significant volume change (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from rainfall, landscape irrigation, perched groundwater, drought, or other factors. Changes in soil moisture may result in unacceptable settlement or heave of structures, concrete slabs supported on grade, or pavements supported on these materials.

Atterberg limits testing on two samples within the upper 1.5 ft bgs indicates that the near-surface lean clays that blanket the site have plasticity indexes of 16 and 19 with liquid limits of 30 and 35. These results indicated that the near-surface soils at the Site have a low to moderate expansion potential. However, a layer of fat clay underlies the lean clay throughout much of the site (as shown on fault trench logs by Bayside Geology as included in Appendix F). One Atterberg limits test on this material showed a plasticity index of 34 with a liquid limit of 53, indicating high expansion potential. We conclude that the potential for expansive soils to affect the proposed Site improvements is moderate to high and should be accounted for in the foundation design.

6.4 FOUNDATIONS AND SETTLEMENT

Based on our subsurface exploration program and our understanding of current and historical Site conditions, we conclude that the Site subsurface generally consists of up to a foot of agriculturally disturbed native sandy clay and sandy silt soils underlain by alluvium predominantly composed of medium dense to dense sands to depths as great as approximately 27 ft bgs. We judge that the proposed energy storage containers and connector substation improvements may be supported on shallow foundations consisting of shallow spread footings or mat foundations, provided the subgrade soils are scarified, overexcavated, and compacted in conformance with the recommendations presented in this report. However, improvements bearing on shallow foundations without ground improvement may experience significant distress under design seismic shaking.

We estimate that new foundations constructed in conformance with our recommendations will experience less than ½ inch of total settlement and less than ½ inch of differential settlement across a 50-foot horizontal distance under static loading conditions. As discussed in *Section 6.2.2 - Soil Liquefaction and Associated Hazards*, total liquefaction-induced vertical settlements of as high as 2¾ inches are possible at the Site, with up to 1½ inches of predicted differential settlement over a lateral distance of 50 ft. Additionally, as discussed in *Section 6.2.3 - Global Stability and Lateral Spreading*, seismic-induced lateral displacements of 10 inches or less (84 percent probability per Bray and Macedo [2019]) are expected to occur as a result of the design seismic event. We assume that the proposed improvements consisting of non-occupiable equipment pads and collector substation equipment are considered “nonbuilding structures” per the 2022 CBC and ASCE 7-16 and could conceptually be designed to withstand lateral movements of this magnitude. (Note that per ASCE 7-16 Section 12.13.9.2, “building structures” may not be founded on shallow foundations where lateral displacements exceed 12 inches for Risk Category III sites). Should the estimated degree of seismic-induced lateral displacements exceed allowable limits as determined by the project structural engineer, the degree of lateral displacement may be significantly diminished using ground improvement, as discussed in *Section 7.3 - Ground Improvement*.

7. Recommendations

Our geotechnical recommendations for foundation support and other geotechnical aspects of the project are presented in this section of the report.

7.1 FOUNDATIONS

Proposed Site improvements may be supported on a number of different foundation types including shallow foundations consisting of mat foundations, slabs-on-grade, and deep foundations, as appropriate for the loading demands of overlying improvements and cost-effectiveness.

We assume that shallow foundations consisting of spread footings or mat foundations will be the most cost-effective foundation option for support of the proposed improvements. These improvements are described in *Section 7.1.1 - Spread Footings* and *Section 7.1.2 - Mat Foundations*. The use of shallow foundations will require special considerations with respect to subgrade preparation and fill placement to mitigate the effects of moderately expansive soils present at the Site. Ground improvement may be considered for the limitation of lateral spread-induced ground movements resulting from seismic shaking, as discussed in *Section 7.3 - Ground Improvement*; however, the estimated degree of lateral spread-induced ground movement (10 inches or lower) is sufficiently low to allow use of shallow foundations without performance of ground improvement, provided the provisions of ASCE 7-16 Section 12.13.9 are followed for structural design of foundations.

7.1.1 Spread Footings

Equipment and facilities supported on spread-type footings should be founded at least 24 inches below the lowest adjacent soil subgrade and should bear on a layer of scarified, moisture-conditioned, and recompacted subgrade. Continuous and isolated footings or grade beams should be at least 18 inches wide and 24 inches square, respectively. We recommend the proposed footings be designed using an allowable bearing pressure of 1,500 psf for dead plus live load conditions. This value contains an FS of at least 3 and may be increased by one-third for total loads, including wind or seismic forces. We estimate that the total and differential settlements of new footings under static loading conditions should be less than ½ inch and ½ inch over a 50-foot horizontal distance, respectively.

Lateral loads on spread footings may be resisted by a combination of passive pressure on the embedded vertical faces of the footings and friction between the bottoms of the footings and the supporting soil. For passive resistance, we recommend using a constant pressure of 100 psf for footings embedded in compacted fill; the upper foot of soil should be ignored unless it is confined by a slab or pavement. Frictional resistance should be computed using an allowable cohesion of 130 psf. Uplift resistance may be computed based on the dead weight of the planned foundation elements.

The footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. The bottoms and sides of the footing excavations should be maintained in a moist condition until concrete is placed. We should check footing excavations prior to the placement of reinforcing steel. Any loose or soft soil exposed beneath footing excavations should be removed, and the resulting overexcavations should be backfilled with compacted fill in accordance with *Section 7.2 - Earthwork.*

7.1.2 Mat Foundations

Reinforced concrete mat foundations are suitable for support of improvements requiring relatively low bearing demand (about 750 psf or less), such as the proposed energy storage containers. Based on our prior experience on similar energy storage system projects, we assume each energy storage container will weigh approximately 20 kips or less, distributed over a typical footprint area of about 7- to 9-ft wide by about 7- to 9-ft long, exerting an average loading stress on the order of about 400 psf or less.

For planning purposes, we recommend a typical slab thickness of 6 inches, with at least 18-inch thickened edges to provide perimeter support and reduce the potential for surface water intrusion into the subgrade. The edge of the slab should extend at least 18 inches below the adjacent subgrade to reduce the potential for intrusion of surface water into the subgrade and aggregate base. Final design slab thickness and reinforcement details should be provided by the project structural engineer.

Lateral loads on mat foundations may be resisted by friction between the bottom of the slab and the supporting aggregate base. Frictional resistance should be computed using an allowable base friction coefficient of 0.40 for slabs bearing on CalTrans Class 2 Aggregate Base over compacted clayey subgrade. Should additional lateral resistance be required to resist design loading, slab edges may be deepened to gain resistance from passive pressure on the embedded vertical slab faces. A constant lateral pressure of 300 psf may be used for mat foundations embedded in compacted fill. The upper 12 inches of soil should be ignored unless it is confined by a slab or pavement.

We recommend that concrete mat foundations be underlain by at least 6 inches of imported California Department of Transportation Class 2 Aggregate Base bearing on subgrade prepared and compacted as described in *Section 7.2.6 - Fill Placement and Compaction*. Subgrade preparation should extend a minimum horizontal distance of 3 ft beyond the edge of the slab. The subgrade should not be allowed to dry during construction; subgrade that is dried or disturbed during construction should be scarified, moisture-conditioned, and rerolled to provide a firm, unyielding surface prior to placement of the aggregate base or casting of the overlying slab-on-grade.

7.1.3 Sound Wall Foundations

We recommend that proposed sound walls be designed with spread footing foundations conforming to the recommendations presented in Section 7.1.1, except where recommendations provided by the Caltrans Geotechnical Manual – Sound Walls (Caltrans, 2020) are applicable and more conservative. Caltrans recommends the minimum dimensions for sound walls bearing on spread footings as presented in Table 7.

Table 7. Minimum Sound Wall Footing Dimensions per Caltrans (2020)

Wall Height (ft)	Minimum Footing Width (ft)
6	3
8	4
10	5
12	5.75
14	6.5
16	7.5

7.1.4 Exterior Slabs-on-Grade (Flatwork)

Exterior flatwork may be supported on grade. We recommend that exterior concrete slabs and pedestrian walkways surrounding the substation facilities and any other Site improvements be designed using 4 inches of concrete at minimum. Typically, construction joints are spaced at horizontal distances no greater than 30 times the concrete slab thickness. If there is a conflict between the civil and geotechnical design recommendations for contraction joint spacing, Haley & Aldrich defers to the Civil Engineer's recommendations. Where concrete flatwork is to be exposed to vehicle traffic, we recommend that the flatwork be supported on a minimum of 4 inches of Class 2 aggregate base. Recommendations for subgrade preparation and aggregate base compaction for concrete slabs and walkways are the same as those we have described in *Section 7.2.6 - Fill Placement and Compaction*.

7.2 EARTHWORK

7.2.1 Site Preparation and Grading

Portions of the Site receiving new structures, slabs, and foundations should be cleared of existing pavements, abandoned utilities, and other obstructions, if present. The areas should be stripped of soil containing over 3 percent organic matter (if present). We anticipate the excavation for this project can be made using conventional earth-moving equipment.

The near-surface soils will include areas disturbed by agricultural activities and may include areas of undocumented cuts and fills. Loose or disturbed soil or undocumented fill soils underlying proposed improvements should be identified during initial Site clearing and grading, and may require overexcavation and recompaction following our assessment.

Fault trenches excavated by Bayside Geology were reportedly backfilled with uncompacted fill. Where these trenches coincide with planned improvements, uncompacted fill should be excavated in its entirety (to an average depth of 5 to 6 ft bgs) and replaced and compacted as specified in conformance with recommendations of Section 7.2. Locations and limits of fault trenches are shown in the Bayside Geology report attached in Appendix F.

7.2.2 Subgrade Preparation

The Site should be rough graded to accommodate the proposed grading plan. In areas that will receive new fills, building loads, and improvements such as slabs-on-grade and foundations supporting low bearing pressure (approximately 750 psf) improvements such as BESS features, the exposed soil subgrade should be prepared by scarifying to a depth of at least 12 inches, and moisture conditioning and compacting the subgrade material in accordance with the recommendations given in *Section 7.2.6 - Fill Placement and Compaction* of this report. Subgrade preparation should extend at least 3 ft beyond the limits of improvements supported on shallow or mat foundations. Measures for preparing the subgrade are presented in *Section 7.2.1 - Site Preparation and Grading* above.

Prepared soil subgrades should be non-yielding when proof rolled by a fully loaded water truck or equipment of similar weight. Moisture conditioning of subgrade soil should consist of adding water if the soil is too dry and allowing the soil to dry if the soil is too wet. After the subgrades are properly prepared, the areas may be raised to design grades by placement of engineered fill.

7.2.3 Import Fill

Import fill is expected to be required to raise grades at the Site. The imported material should meet the requirements presented in Table 8, below, if used as import fill.

Table 8. Import Fill Grading Requirements

Sieve Size	Percentage Passing Sieve
3 inches	100
1½ inches	85-100
#200 Screen	8-40
Atterberg Limits	**Percent**
Plasticity Index	12 or less
Liquid Limit	Less than 30

All onsite or import fill material should be compacted to the recommendations provided for engineered fill in *Section 7.2.6 - Fill Placement and Compaction* of this report. Fill materials should be approved by the project geotechnical engineer prior to delivery to and placement at the Site. At least five working days prior to importing to the Site, a representative sample of the proposed import fill should be delivered to our laboratory for evaluation.

7.2.4 General Fill

The onsite soil is suitable for use as general fill if it is free of deleterious matter and satisfies the criteria in *Table 9 - General Fill Grading Requirements*.

Onsite soil has a moderate to high expansion potential and should be placed and compacted in conformance with the criteria presented in *Section 7.2.6 - Fill Placement and Compaction*, especially with respect to moisture content. General fill should not be allowed to dry out prior to the construction of foundations or overlying improvements, as this may result in a heightened potential for expansion-related distress to occur over time. To further reduce the risk of expansion-related distress, import fill meeting the criteria presented in *Section 7.2.3 - Import Fill* may be considered for use as subgrade below foundations, mats, and slabs, placed to a depth of at least 6 inches beneath said improvements.

Soils used as general fill should be inorganic, and free of deleterious materials and hazardous substances. For this project, inorganic soil is soil with an organic content of less than 3 percent by weight or without visible organic matter deemed excessive by Haley & Aldrich.

Table 9. General Fill Grading Requirements

Sieve Size	Percentage Passing Sieve
3 inches	100
1½ inches	85-100

7.2.5 "Non-expansive" Fill

Because of the high expansion potential of the onsite near-surface soil, we recommend the concrete slabs-on-grade be constructed on a layer of "non-expansive" fill, meeting the requirements presented above under *Section 7.2.3 - Import Fill*. In areas that will support concrete slabs-on-grade, the "non-expansive" fill layer should be at least 6 inches thick and should extend at least 5 ft horizontally beyond the limits of the concrete slabs. Where Class 2 aggregate base will be used under concrete slabs, this material may be considered as the upper 4 inches of the “non-expansive” fill.

7.2.6 Fill Placement and Compaction

Fill materials shall be moisture conditioned to at least two percentage points above optimum moisture content, and be placed and compacted in horizontal lifts, each not exceeding 8 inches in uncompacted thickness. Compaction of fill should be performed by mechanical means only. Given equipment limitations, thinner lifts may be necessary to achieve the recommended degree of compaction. Fill should be placed in accordance with *Table 10 - Summary of Compaction Recommendations.*

Table 10. Summary of Compaction Recommendations

Area	Compaction Recommendations (See Notes 1 through 4)
Subgrade Preparation and Placement of General Fill and Import Fill[5]	Compact upper 12 inches of subgrade and entire fill to a minimum of 90 percent compaction with a moisture level of at least 2 percent over optimum moisture content. Where flatwork is exposed to vehicular traffic, compact aggregate base to a minimum of 95 percent compaction at or slightly over optimum moisture content.
Trenches[6]	Compact trench backfill to a minimum of 90 percent compaction with a moisture level of at least 2 percent over optimum moisture content. Where trenches will be under the pavement section, flatwork, or other improvements, the upper 12 inches, measured from finished grade of the trench backfill should be compacted to a minimum of 95 percent compaction.
Exterior Flatwork and Interior Slabs-on-Grade	Compact upper 12 inches of subgrade to a minimum of 90 percent compaction with a moisture level of at least 2 percent over optimum moisture content. Compact aggregate base to a minimum of 95 percent compaction at near or slightly over the optimum moisture content. Where exterior flatwork is exposed to vehicular traffic, compact aggregate base to a minimum of 95 percent compaction or slightly over optimum moisture content.
Asphalt-Paved and Gravel-Paved Roads	Compact upper 12 inches of subgrade to a minimum of 92 percent compaction with a moisture level of at least 2 percent over optimum moisture content. Compact aggregate base to a minimum of 95 percent compaction.

Notes:

1. *Depths are below finished subgrade elevation.*
2. *All compaction requirements refer to relative compaction as a percentage of the laboratory standard described by ASTM International D-1557 (latest version). All lifts to be compacted shall be a maximum of 8 inches loose thickness.*
3. *All compacted surfaces, such as fills, subgrades, and backfills, need to be firm and stable and should be unyielding under compaction equipment.*
4. *Where fills, such as backfill placement after removal of existing underground utility lines, are greater than 7 ft in depth, the portion of the fill deeper than 7 ft should be compacted to a minimum of 95 percent compaction.*
5. *Includes building pads.*
6. *In landscaping areas, this percent compaction in trenches may be reduced to 85 percent.*
7. *Water jetting or flooding to obtain compaction of backfill should not be permitted.*

7.2.7 Underground Utilities

Excavations for utility trenches can be made with a backhoe. All trenches should conform to the current California Division of Occupational Safety and Health requirements. Backfill for utility trenches and other excavations is also considered to be filled, and it should be compacted according to the recommendations presented in *Section 7.2.6 - Fill Placement and Compaction*. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

Underground utilities should be located above a 1.5:1 (horizontal to vertical) plane projected downward from the bottom of the new footings to avoid undermining the footings during the excavation of the utility trench.

Pipes or conduits should be supported on bedding material with a thickness equal to D/4 (with *D* equal to the outside diameter of the pipe) or 4 inches of sand or fine non-angular gravel below the pipe, whichever is greater. After the pipes and conduits are tested, inspected (if required), and approved, they should be covered to a minimum depth of 6 inches with sand or fine non-angular gravel and mechanically tamped.

7.3 GROUND IMPROVEMENT

Sandy soils identified below near-surface clays to depths of about 16 to 27 ft bgs across the Site are likely susceptible to the effects of liquefaction during and after a design-level earthquake. The use of ground improvement measures to improve the strength of the soils can mitigate the potential for liquefaction and its effects, such as settlement, ground disruption, and lateral spreading. In general, if ground improvement is implemented into Site development plans, it will need to extend through the liquefiable sands and into the deeper non-liquefiable clay soils.

We considered several potential methods of ground improvement that could theoretically be employed at the Site, should the design team wish to limit potential lateral spread induced settlements to no greater than 10 inches and limit liquefaction-induced vertical settlements to lower than 1 1/2 inch over 50 ft. As lateral displacement poses a more severe hazard to the planned Site improvements than vertical settlement, our considerations were limited to such methods that could most effectively limit the risk of lateral spreading in a cost-efficient manner. We conclude that soil mixing to produce a cut-off buttress between the proposed BESS improvements and College Lake would most efficiently meet this end.

Soil mixing is a ground improvement technique that creates *in situ* soil-cement ("soilcrete") elements that are liquefaction resistant and have a relatively high strength and stiffness. Soil mixing techniques generally consist of mass mixing (or shallow mixing) and deep soil mixing, described as follows:

- **Mass mixing** is typically feasible for the improvement of soils to a depth of 30 ft bgs or shallower. This technique uses a specialized rotary head mounted on the boom of an excavator. The head is advanced into the native soils while pumping cement through the mixing drum and mixing the soil to the target depth. Continuous rectangular "walls" are typically mixed in the field.
- **Deep soil mixing** is required for improvements of soils deeper than 30 ft bgs. This technique requires the mobilization of one or more drill rigs using a large-diameter auger(s) to mix wet cement slurry or dry cement into the soil. This process constructs individual soil columns that are designed to overlap.

Soil mixing produces a significant amount of spoils that must be managed. These spoils contain high pH due to the cement content and will likely require onsite usage to avoid high fees associated with offsite disposal.

Soil mixing is considered a preferred ground improvement measure for future consideration, if required to reduce estimated lateral displacements. Due to the relatively shallow depth of needed improvement, mass stabilization via excavator-mounted equipment is feasible and likely presents the most cost-efficient alternative for the soil mixing approach.

7.4 SURFACE DRAINAGE

Site grading should provide surface drainage away from the proposed foundations, aboveground improvements, and gravel roads. Surface water should not be allowed to collect adjacent to the proposed foundations and along the edges of concrete slabs or pavements. Grades should be sloped away from pads and foundations as required in the CBC (current edition). Surface water should be directed away from exposed soil slopes.

7.5 STORMWATER INFILTRATION

One small-diameter boring infiltration test was performed at boring location IT-1 (see location on Figure 2). Infiltration test results are presented in Appendix C. The interpretation of the field infiltration test data provided the rate for infiltration features with a depth at about 3 ft bgs is about 0.8 inches per hour. We recommend that a reduction factor of 5.5 be applied to this rate, to account for variation associated with the small-diameter boring infiltration test method and potential for long-term siltation or maintenance issues. Based on the above, a design infiltration rate of 0.15 inches per hour is recommended.

The effective infiltration rate of finished stormwater infiltration features can vary significantly from the rates estimated from the infiltration test. The following activities may diminish the infiltration rate of proposed stormwater features and should be avoided:

- Placement of artificial fill within the stormwater infiltration feature during grading, especially placement of fill materials with poor drainage properties.
- Allowing construction runoff containing fine-grained soils to drain into the feature and cause siltation during Site grading.
- Grading methods that result in smearing or compacting of soils at the feature or basin invert, including compaction by driving heavy equipment over the area.
- Design and siting of infiltration features at locations or elevations significantly different from those tested.

7.6 FLEXIBLE PAVEMENT DESIGN

We assume that vehicle access roads will be designed for periodic maintenance-vehicle access and infrequent loading from heavy vehicles. Based on this assumption and the results of our subsurface exploration indicating the presence of clayey soils in the near-surface zone, we recommend that gravel access roads at the Site be designed with a minimum thickness of 7 inches. We recommend that the aggregate base used to construct the access roads be underlain by a layer of geotextile separation fabric (Mirafi 140N or approved equivalent) to improve road performance.

We recommend that the subgrade soil be moisture-conditioned and compacted according to the recommendations in the “Fill Placement and Compaction” Section of this report. Subgrade preparation should extend a minimum of 2 ft laterally beyond the edge of the access road. The road surface should be sloped, and drainage gradients maintained to carry all surface water to appropriate collection points. Periodic releveling and maintenance of gravel roads over time should be expected.

7.7 CORROSION POTENTIAL

One sample was collected and tested for corrosivity characteristics by CERCO of Concord, California. The tested sample consisted of near-surface soils sampled from bulk samples collected at boring location CPT-2. The sample was tested for resistivity, redox potential, sulfate and chloride ion concentrations, and pH. The test results indicate that near-surface soils should be considered “moderately corrosive” to buried iron and steel improvements, based on laboratory resistivity measurements, according to CERCO. Chloride and sulfate ion concentrations were not detected above the 15 milligrams per kilogram reporting limit and were thus determined by CERCO to be insufficient to attach steel embedded in concrete mortar coating nor cause damage to reinforced concrete structures and cement mortar coated steel. Per CERCO, the pH level of the soil was essentially neutral at 6.9 and is not a corrosion concern for buried iron, steel, mortar-coated steel, and reinforced concrete structures. The results of our corrosion testing and a copy of CERCO’s brief evaluation are presented in Appendix B.

8. Supplemental Geotechnical Services

The findings and recommendations presented within this geotechnical design report are tentative pending the findings of the geologic hazards study and planned fault trenching activities. We recommend that Haley & Aldrich be consulted to review the findings of these studies prior to the preparation of a final design-level geotechnical report for the proposed improvements. Location of any onsite active faults, or positive determination of lack of active onsite faulting, is critical for this project to proceed from a geotechnical standpoint.

Supplemental field exploration (beyond the planned fault trenching activities) may be required by Santa Cruz County for acceptance of the design-level geotechnical report. Supplemental geotechnical exploration activities may include drilling at least one boring at the Site, performing standard penetration test sampling, and conducting a limited program of geotechnical laboratory testing.

Once a final geotechnical design-level report has been prepared, the final project plans and specifications should be reviewed by Haley & Aldrich prior to construction to check that they are in general conformance with the intent of our recommendations. During construction, we should observe and document the installation of foundations and observe the condition and test the compaction of any fill placed at the Site. These observations will allow us to check that the contractor’s work conforms to the geotechnical aspects of the plans and specifications, and ensure the foundation system and paved areas are constructed in accordance with our design recommendations.

9. Limitations

This report has been prepared for specific application to the proposed construction as understood at this time. In the event that changes in the nature, design, or location of the project are planned, the conclusions and recommendations contained in this report should not be considered valid, unless the changes are reviewed by Haley & Aldrich and the conclusions of this report are modified or verified in writing.

The geotechnical analyses and recommendations are based, in part, upon the data obtained from the referenced subsurface explorations. The nature and extent of variations between explorations may not become evident until construction. If variations appear at that time, it may be necessary to re-evaluate the recommendations of this report.

This report is prepared for the exclusive use of New Leaf Energy, Inc. and their subconsultants in connection with the design and construction of the proposed energy storage site located at 90 Minto Road in Watsonville, California. There are no intended beneficiaries other than New Leaf Energy and their subconsultants. Haley & Aldrich shall owe no duty whatsoever to any other person or entity on account of the Agreement or the report. Use of this report by any person or entity other than New Leaf Energy, Inc. and their subconsultants for any purpose whatsoever is expressly forbidden unless such other person or entity obtains written authorization from New Leaf Energy, Inc. and Haley & Aldrich. Use of this report by such other person or entity without the written authorization of New Leaf Energy, Inc. and Haley & Aldrich shall be at such other person's or entity's sole risk and shall be without legal exposure or liability to Haley & Aldrich.

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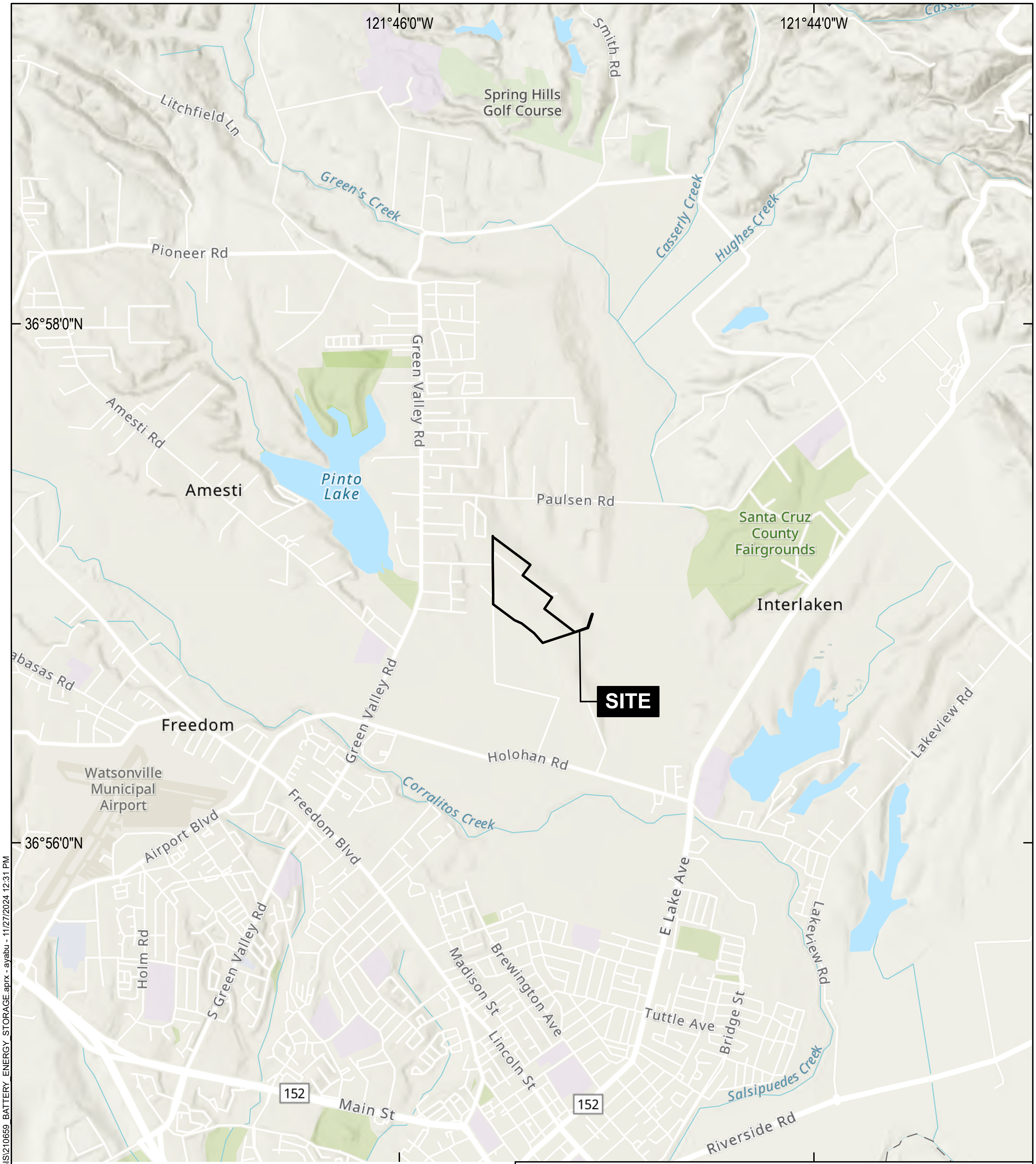
FIGURES



MAP SOURCE: ESRI
SITE COORDINATES: 36°56'58"N, 121°45'26"W

HALEY ALDRICH

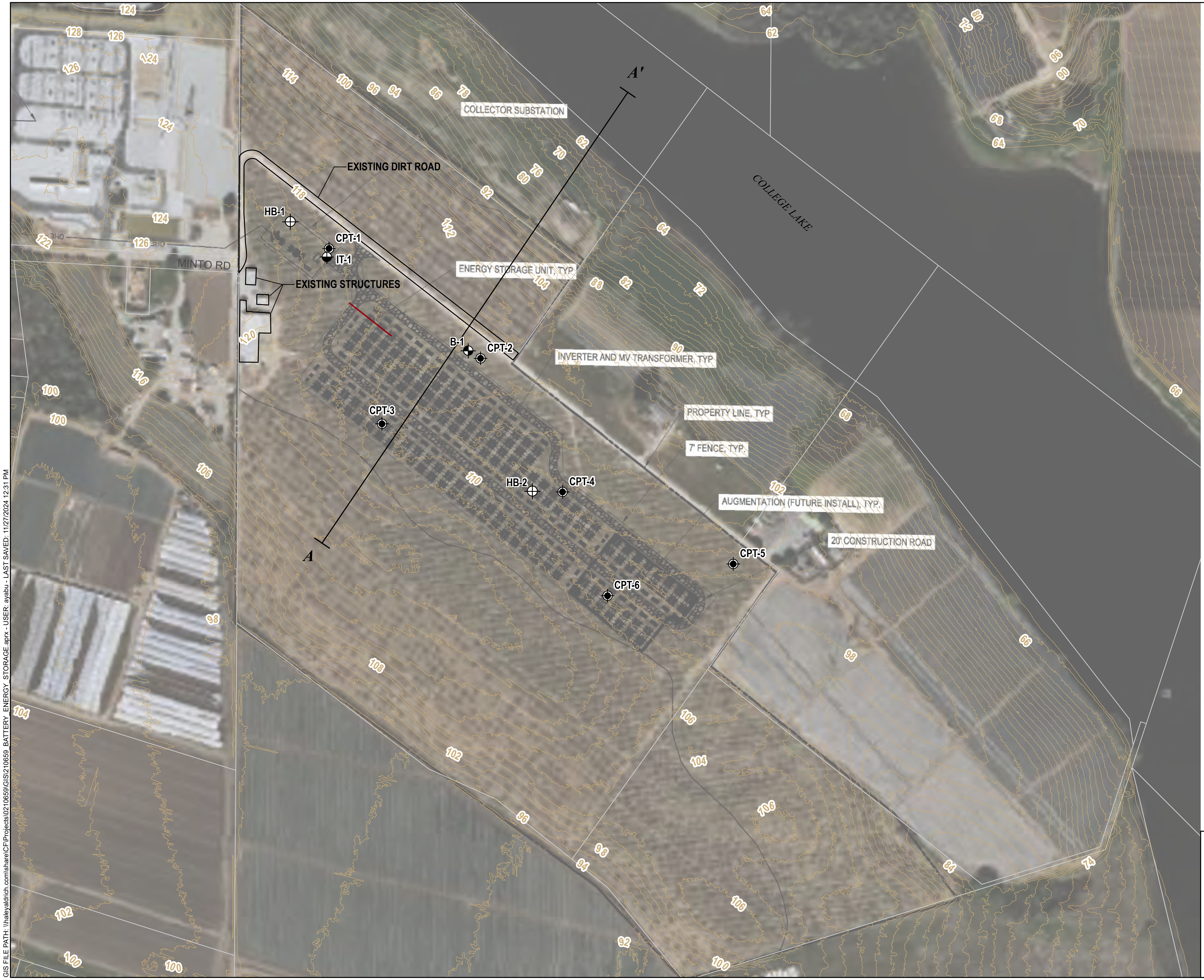
NEW BATTERY ENERGY STORAGE SYSTEMS INSTALLATION
90 MINTO ROAD
WATSONVILLE, CALIFORNIA

VICINITY MAP

APPROXIMATE SCALE: 1 IN = 4000 FT
DECEMBER 2024

FIGURE 1

GIS: \\haleyaldrich.com\share\CF\Projects\0210659\GIS\210659_BATTERY_ENERGY_STORAGE.aprx - ayabu - 11/27/2024 12:31 PM



LEGEND

● CONE PENETRATION TEST (CPT)

◓ INFILTRATION TEST (IT)

◈ MUD ROTARY BORING

⊕ HAND AUGER BORING

TOPOGRAPHIC ELEVATION CONTOUR, 2-FT INTERVAL

PROFILE

RESISTIVITY LINE

☐ PARCEL BOUNDARY

NOTES

1. ALL LOCATIONS AND DIMENSIONS ARE APPROXIMATE.
2. TOPOGRAPHIC CONTOUR DATA SOURCE: SANTA CRUZ COUNTY 2020 (NAVD88)
3. BASEMAP PROVIDED BY NEW LEAF ENERGY, INC.
4. ASSESSOR PARCEL DATA SOURCE: SANTA CRUZ COUNTY

N

0 250 500

SCALE IN FEET

HALEY ALDRICH

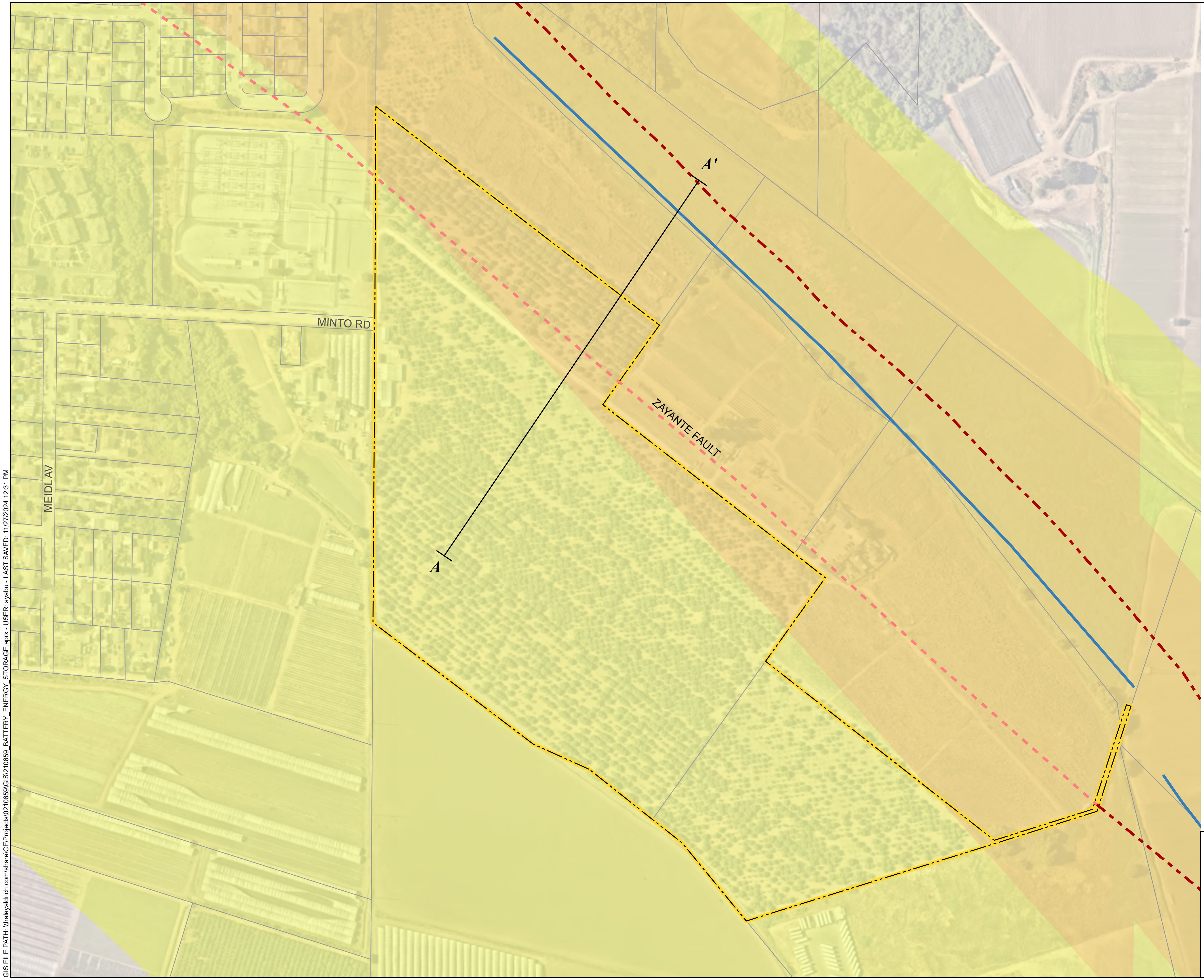
NEW BATTERY ENERGY STORAGE SYSTEMS INSTALLATION
90 MINTO ROAD
WATSONVILLE, CALIFORNIA

SITE AND EXPLORATION PLAN

DECEMBER 2025

FIGURE 2

GIS FILE PATH: \\haleyaldrich.com\share\CF\Projects\0210659\GIS\210659_BATTERY_ENERGY_STORAGE.aprx - USER: ayabu - LAST SAVED: 11/27/2024 12:31 PM



LEGEND

├─┤ PROFILE

MAPPED FAULTS (COUNTY OF SANTA CRUZ)

- - - APPROXIMATELY LOCATED

- - - CONCEALED

MAPPED FAULTS (STATE OF CALIFORNIA)

— APPROXIMATELY LOCATED FAULT TRACE

FAULT ZONES

🟨 COUNTY DEFINED FAULT ZONE

🟧 STATE DESIGNATED SEISMIC REVIEW ZONE

▭ SITE BOUNDARY

☐ PARCEL BOUNDARY

NOTES

1. ALL LOCATIONS AND DIMENSIONS ARE APPROXIMATE.
2. FAULT DATA SOURCE: SANTA CRUZ COUNTY
3. ASSESSOR PARCEL DATA SOURCE: SANTA CRUZ COUNTY
4. AERIAL IMAGERY SOURCE: NEARMAP, 30 JUNE 2023

HALEY ALDRICH

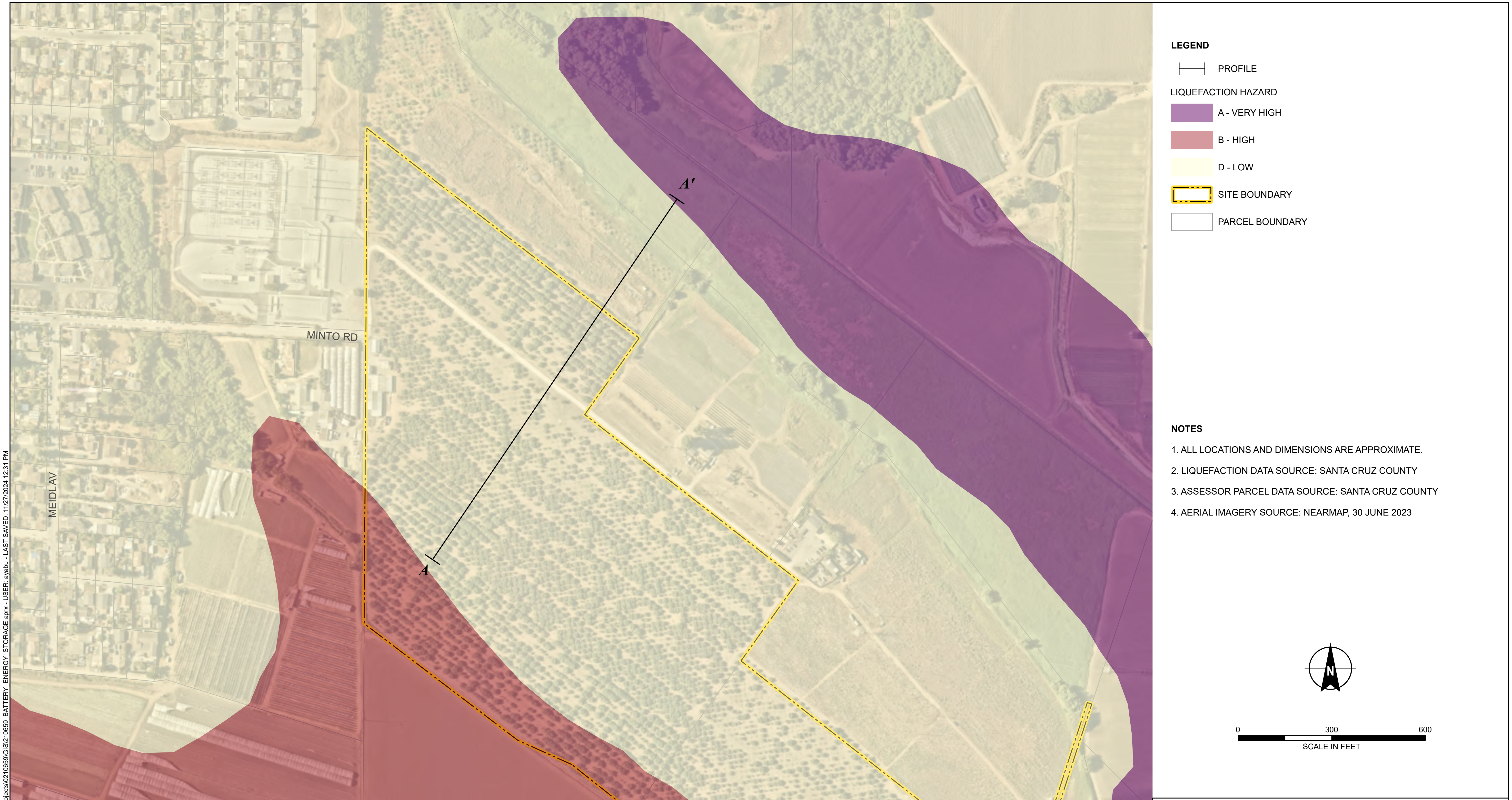
NEW BATTERY ENERGY STORAGE SYSTEMS INSTALLATION
90 MINTO ROAD
WATSONVILLE, CALIFORNIA

MAPPED FAULTS

DECEMBER 2024

FIGURE 3

GIS FILE PATH: \\haleyaldrich.com\share\CF\Projects\0210659\GIS\210659_BATTERY_ENERGY_STORAGE.aprx - USER: ayabu - LAST SAVED: 11/27/2024 12:31 PM



LEGEND

├─┤ PROFILE

LIQUEFACTION HAZARD

🟪 A - VERY HIGH

🟥 B - HIGH

🟨 D - LOW

▭ SITE BOUNDARY

▭ PARCEL BOUNDARY

NOTES

1. ALL LOCATIONS AND DIMENSIONS ARE APPROXIMATE.
2. LIQUEFACTION DATA SOURCE: SANTA CRUZ COUNTY
3. ASSESSOR PARCEL DATA SOURCE: SANTA CRUZ COUNTY
4. AERIAL IMAGERY SOURCE: NEARMAP, 30 JUNE 2023

HALEY ALDRICH

NEW BATTERY ENERGY STORAGE SYSTEMS INSTALLATION
90 MINTO ROAD
WATSONVILLE, CALIFORNIA

MAPPED LIQUEFACTION HAZARD

DECEMBER 2024

FIGURE 4

GIS FILE PATH: \\haleyaldrich.com\share\CF\Projects\0210659\GIS\210659_BATTERY_ENERGY_STORAGE.aprx - USER: ayabu - LAST SAVED: 11/27/2024 12:31 PM

APPENDIX A
Exploration Logs

Sample Description

Identification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field nor laboratory testing unless presented herein. ASTM D 2488 visual-manual identification methods were used as a guide. Where laboratory testing confirmed visual-manual identifications, then ASTM D 2487 was used to classify the soils.

Relative Density/Consistency

Soil density/consistency in borings is related primarily to the standard penetration resistance (N). Soil density/consistency in test pits and probes is estimated based on visual observation and is presented parenthetically on the logs.

SAND or GRAVEL Relative Density	N (Blows/Foot)	SILT or CLAY Consistency	N (Blows/Foot)
Very loose	0 to 4	Very soft	0 to 1
Loose	5 to 10	Soft	2 to 4
Medium dense	11 to 30	Medium stiff	5 to 8
Dense	31 to 50	Stiff	9 to 15
Very dense	>50	Very stiff	16 to 30
		Hard	>30

Moisture

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

USCS Soil Classification Chart (ASTM D 2487)

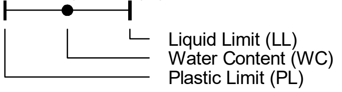
Major Divisions			Symbols		Typical Descriptions
			Graph	USCS	
Coarse Grained Soils More than 50% of Material Retained on No. 200 Sieve	Gravel and Gravelly Soils More than 50% of Coarse Fraction Retained on No. 4 Sieve	Clean Gravels *(<5% fines)*	●	GW	Well-Graded Gravel; Well-Graded Gravel with Sand
			○	GP	Poorly Graded Gravel; Poorly Graded Gravel with Sand
		Gravels *(5-12% fines)*	●	GW-GM	Well-Graded Gravel with Silt; Well-Graded Gravel with Silt and Sand
			●	GW-GC	Well-Graded Gravel with Clay; Well-Graded Gravel with Clay and Sand
			○	GP-GM	Poorly Graded Gravel with Silt; Poorly Graded Gravel with Silt and Sand
			○	GP-GC	Poorly Graded Gravel with Clay; Poorly Graded Gravel with Clay and Sand
		Gravels with Fines *(>12% fines)*	○	GM	Silty Gravel; Silty Gravel with Sand
			▨	GC	Clayey Gravel; Clayey Gravel with Sand
	Sand and Sandy Soils More than 50% of Coarse Fraction Passing No. 4 Sieve	Sands with few Fines *(<5% fines)*	⁘	SW	Well-Graded Sand; Well-Graded Sand with Gravel
			⁘	SP	Poorly Graded Sand; Poorly Graded Sand with Gravel
		Sands *(5-12% fines)*	⁘	SW-SM	Well-Graded Sand with Silt Well-Graded Sand with Silt and Gravel
			▨	SW-SC	Well-Graded Sand with Clay; Well-Graded Sand with Clay and Gravel
			⁘	SP-SM	Poorly Graded Sand with Silt; Poorly Graded Sand with Silt and Gravel
			▨	SP-SC	Poorly Graded Sand with Clay; Poorly Graded Sand with Clay and Gravel
		Sands with Fines *(>12% fines)*	⁘	SM	Silty Sand; Silty Sand with Gravel
			▨	SC	Clayey Sand; Clayey Sand with Gravel
Fine Grained Soils More than 50% of Material Passing No. 200 Sieve	Silts		▥	ML	Silt; Silt with Sand or Gravel; Sandy or Gravelly Silt
			▥	MH	Elastic Silt; Elastic Silt with Sand or Gravel; Sandy or Gravelly Elastic Silt
	Silty Clay *(based on Atterberg Limits)*		▨	CL-ML	Silty Clay; Silty Clay with Sand or Gravel; Gravelly or Sandy Silty Clay
	Clays		▨	CL	Lean Clay; Lean Clay with Sand or Gravel; Sandy or Gravelly Lean Clay
			▨	CH	Fat Clay; Fat Clay with Sand or Gravel; Sandy or Gravelly Fat Clay
	Organics		▤	OL/OH	Organic Soil; Organic Soil with Sand or Gravel; Sandy or Gravelly Organic Soil
Highly Organic *(>50% organic material)*			⊥	PT	Peat - Decomposing Vegetation - Fibrous to Amorphous Texture

Minor Constituents

	Estimated Percentage
Sand, Gravel	
Trace	<5
Few	5 - 15
Cobbles, Boulders	
Trace	<5
Few	5 - 10
Little	15 - 25
Some	30 - 45

Soil Test Symbols

%F	Percent Passing No. 200 Sieve
AL	Atterberg Limits (%)



CA	Chemical Analysis
CAUC	Consolidated Anisotropic Undrained Compression
CAUE	Consolidated Anisotropic Undrained Extension
CBR	California Bearing Ratio
CIDC	Consolidated Drained Isotropic Triaxial Compression
CIUC	Consolidated Isotropic Undrained Compression
CK0DC	Consolidated Drained k0 Triaxial Compression
CK0DSS	Consolidated k0 Undrained Direct Simple Shear
CK0UC	Consolidated k0 Undrained Compression
CK0UE	Consolidated k0 Undrained Extension
CRSCN	Constant Rate of Strain Consolidation
DS	Direct Shear
DSS	Direct Simple Shear
DT	In Situ Density
GS	Grain Size Classification
HYD	Hydrometer
ILCN	Incremental Load Consolidation
K0CN	k0 Consolidation
kc	Constant Head Permeability
kf	Falling Head Permeability
MD	Moisture Density Relationship
OC	Organic Content
OT	Tests by Others
P	Pressuremeter
PID	Photoionization Detector Reading
PP	Pocket Penetrometer
SG	Specific Gravity
TRS	Torsional Ring Shear
TV	Torvane
UC	Unconfined Compression
UUC	Unconsolidated Undrained Triaxial Compression
VS	Vane Shear
WC	Water Content (%)

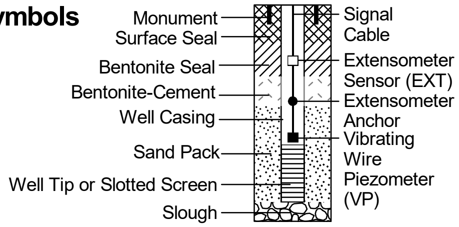
Groundwater Indicators

▽	Groundwater Level on Date or At Time of Drilling (ATD)
▼	Groundwater Level on Date Measured in Piezometer
♀	Groundwater Seepage (Test Pits)

Sample Symbols

☒	SPT Split Spoon	▐	Rock Core Run	☒	Grab
◣	Oversized Split Spoon (see appendix text)	▒	Sonic Core	▥	Cuttings
▐	Push Probe	◫	Thin-walled Sampler		

Well Symbols



HALEY ALDRICH

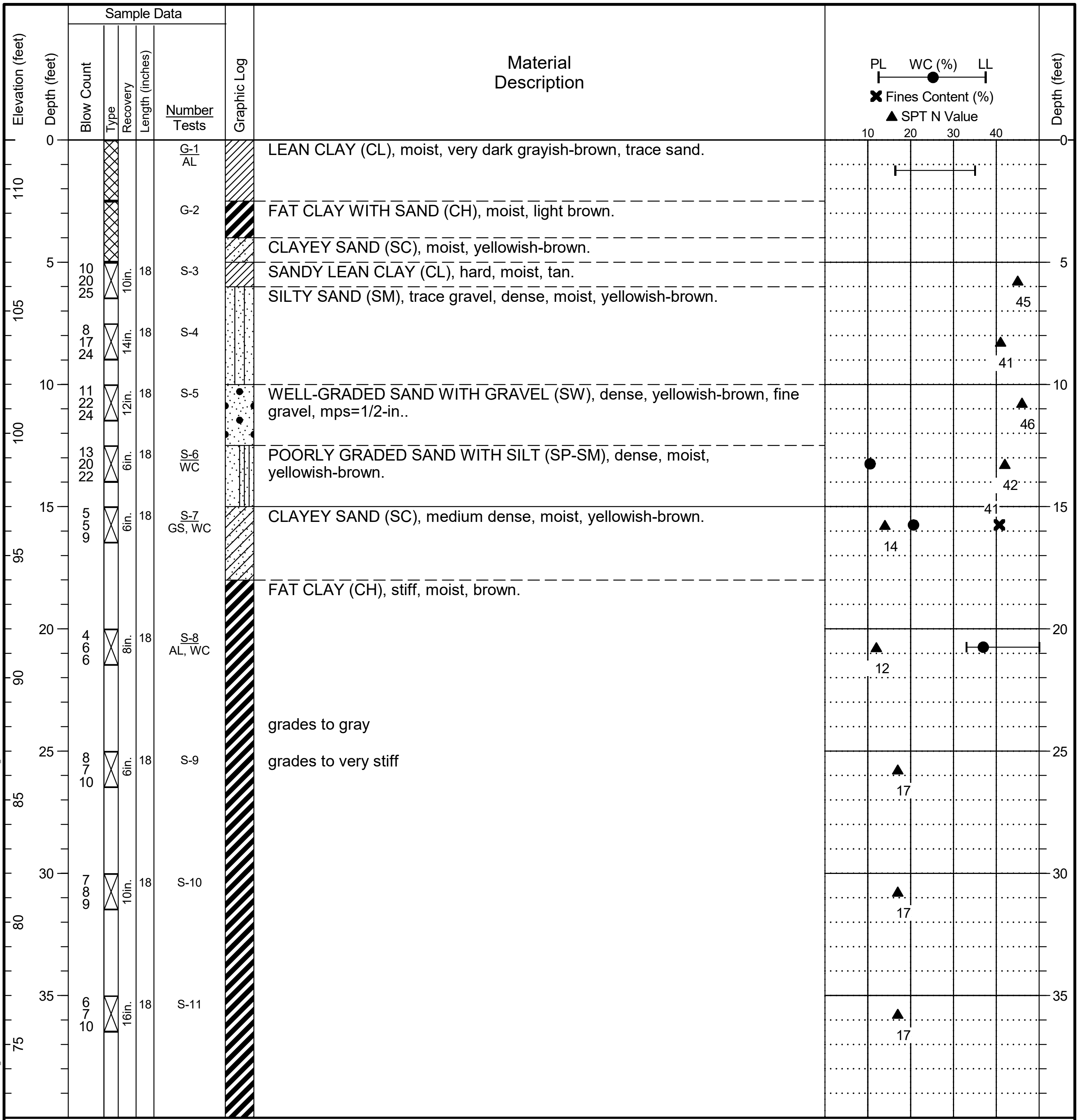
Project: NEW LEAF SEAHAWK WATSONVILLE
Location: WATSONVILLE, CALIFORNIA
Project No.: 0210659-004

Key to Exploration Logs

Figure **A-1**
Sheet **1 of 1**

HA KEY TO EXP LOGS (SOIL ONLY) - \\HALEYALDRICH.COM\SHARE\PDX_DATA\GEOMATICS\GINT\HC_LIBRARY.GLB - 12/16/25 14:09 - \\HALEYALDRICH.COM\SHARE\CF\PROJECTS\0210659\GINT\0210659-004_GINT.GPJ - mschweitzer

Date Started: 12/02/2025
Date Completed: 12/02/2025
Logged by: T. Poehlmann
Checked by: M. Hintz
Location: Lat: 36.950454 Long: -121.757187 (WGS 84)
Ground Surface Elevation: 112.00 feet (NAVD 88)
Comments:
Drilling Contractor/Crew: Gregg Drilling
Drilling Method: Mud Rotary
Rig Model/Type: Track-mounted drill rig
Hammer Type: Auto-hammer
Hammer Weight (pounds): 140
Hammer Drop Height (inches): 30
Measured Hammer Efficiency (%): Not Available
Hole Diameter: 4.5 inches
Well Casing Diameter: NA
Total Depth: 51.5 feet
Depth to Groundwater: Not Identified



Depth (feet)	Blow Count	Type	Recovery	Length (inches)	Number / Tests	Material Description
0					G-1 / AL	LEAN CLAY (CL), moist, very dark grayish-brown, trace sand.
					G-2	FAT CLAY WITH SAND (CH), moist, light brown.
						CLAYEY SAND (SC), moist, yellowish-brown.
5	10 20 25		10in.	18	S-3	SANDY LEAN CLAY (CL), hard, moist, tan.
						SILTY SAND (SM), trace gravel, dense, moist, yellowish-brown.
	8 17 24		14in.	18	S-4	
10	11 22 24		12in.	18	S-5	WELL-GRADED SAND WITH GRAVEL (SW), dense, yellowish-brown, fine gravel, mps=1/2-in..
	13 20 22		6in.	18	S-6 / WC	POORLY GRADED SAND WITH SILT (SP-SM), dense, moist, yellowish-brown.
15	5 5 9		6in.	18	S-7 / GS, WC	CLAYEY SAND (SC), medium dense, moist, yellowish-brown.
						FAT CLAY (CH), stiff, moist, brown.
20	4 6 6		8in.	18	S-8 / AL, WC	
						grades to gray
25	8 7 10		6in.	18	S-9	grades to very stiff
30	7 8 9		10in.	18	S-10	
35	6 7 10		16in.	18	S-11	

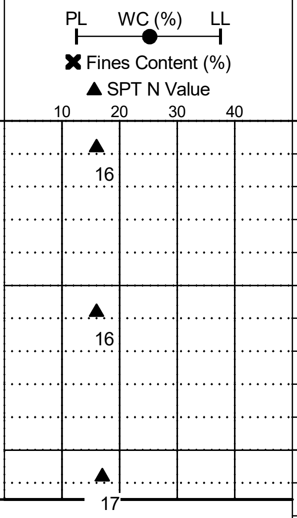
General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

Date Started: 12/02/2025
Date Completed: 12/02/2025
Logged by: T. Poehlmann
Checked by: M. Hintz
Location: Lat: 36.950454 Long: -121.757187 (WGS 84)
Ground Surface Elevation: 112.00 feet (NAVD 88)
Comments:

Drilling Contractor/Crew: Gregg Drilling
Drilling Method: Mud Rotary
Rig Model/Type: Track-mounted drill rig
Hammer Type: Auto-hammer
Hammer Weight (pounds): 140
Hammer Drop Height (inches): 30
Measured Hammer Efficiency (%): Not Available
Hole Diameter: 4.5 inches
Well Casing Diameter: NA
Total Depth: 51.5 feet
Depth to Groundwater: Not Identified

Depth (feet)	Blow Count	Type	Recovery	Length (inches)	Number Tests	Graphic Log	Material Description	SPT N Value
40	6 7 9	☒	18in.	18	S-12	▨	FAT CLAY (CH), very stiff, moist, gray.	16
45	6 7 9	☒	18in.	18	S-13	▨	grades to dark gray	16
50	6 8 9	☒	18in.	18	S-14	▨		17

Bottom of Borehole at 51.5 feet.



Elevation (feet): 70, 65, 60, 55, 50, 45, 40, 35
Depth (feet): 40, 45, 50, 55, 60, 65, 70, 75

General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

HA BORING LOG - \\HALEYALDRICH.COM\SHARE\PDX_DATA\GEOMATICS\GINT\HC_LIBRARY.GLB - 12/16/25 14:08 - \\HALEYALDRICH.COM\SHARE\CF\PROJECTS\0210659\GINT\0210659-004_GINT.GPJ - mschweitzer

Date Started: 12/04/2025 Date Completed: 12/04/2025 Contractor/Crew: Haley & Aldrich, Inc.
Logged by: T. Poehlmann Checked by: M. Hintz Rig Model/Type: Hand Auger
Location: Lat: 36.951320 Long: -121.758539 (WGS 84) Hole Diameter: 4 inches Well Casing Diameter: NA
Ground Surface Elevation: 120.00 feet (NAVD 88) Total Depth: 3.0 feet Depth to Groundwater: Not Identified
Comments:

Elevation (feet)	Depth (feet)	Graphic Log	Material Description	Depth (feet)
	0	▨	SANDY LEAN CLAY (CL), moist, dark brown.	0
		▨	FAT CLAY (CH), moist, light brown.	
			Bottom of Borehole at 3.0 feet.	
115	5			5
110	10			10
105	15			15
100	20			20
95	25			25
90	30			30
85	35			35

General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

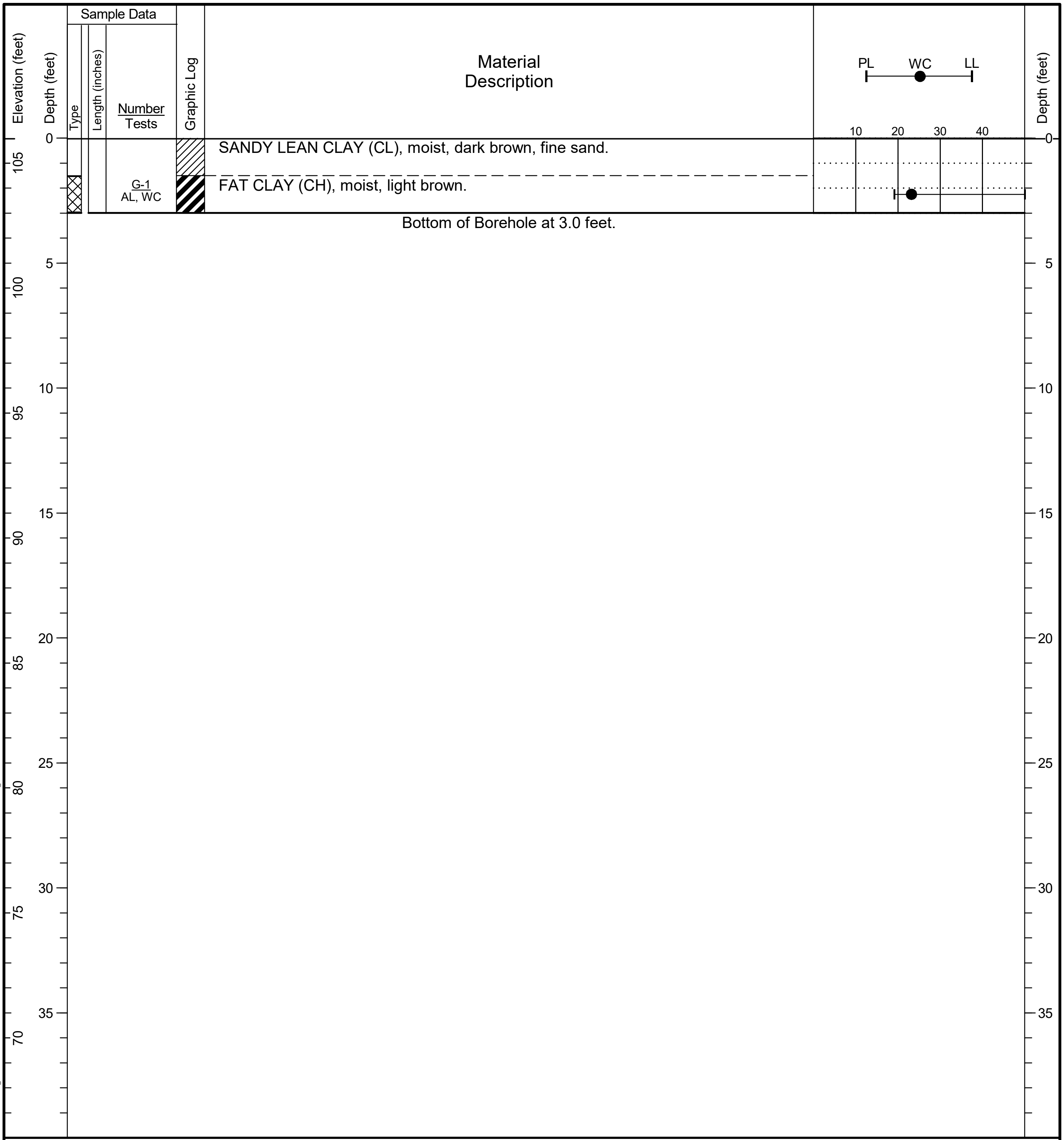
HALEY ALDRICH

Project: NEW LEAF SEAHAWK WATSONVILLE
Location: WATSONVILLE, CALIFORNIA
Project No.: 0210659-004

Hand-Auger Log
HB-01

Figure **A-3**
Sheet **1 of 1**

Date Started: 12/04/2025
Date Completed: 12/04/2025
Contractor/Crew: Haley & Aldrich, Inc.
Logged by: T. Pehlmann
Checked by: M. Hintz
Rig Model/Type: Hand Auger
Location: Lat: 36.949477 Long: -121.756338 (WGS 84)
Hole Diameter: 4 inches
Well Casing Diameter: NA
Ground Surface Elevation: 106.00 feet (NAVD 88)
Total Depth: 3.0 feet
Depth to Groundwater: Not Identified
Comments:



SANDY LEAN CLAY (CL), moist, dark brown, fine sand.

FAT CLAY (CH), moist, light brown.

Bottom of Borehole at 3.0 feet.

General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

HALEY ALDRICH

Project: NEW LEAF SEAHAWK WATSONVILLE
Location: WATSONVILLE, CALIFORNIA
Project No.: 0210659-004

Hand-Auger Log
HB-02

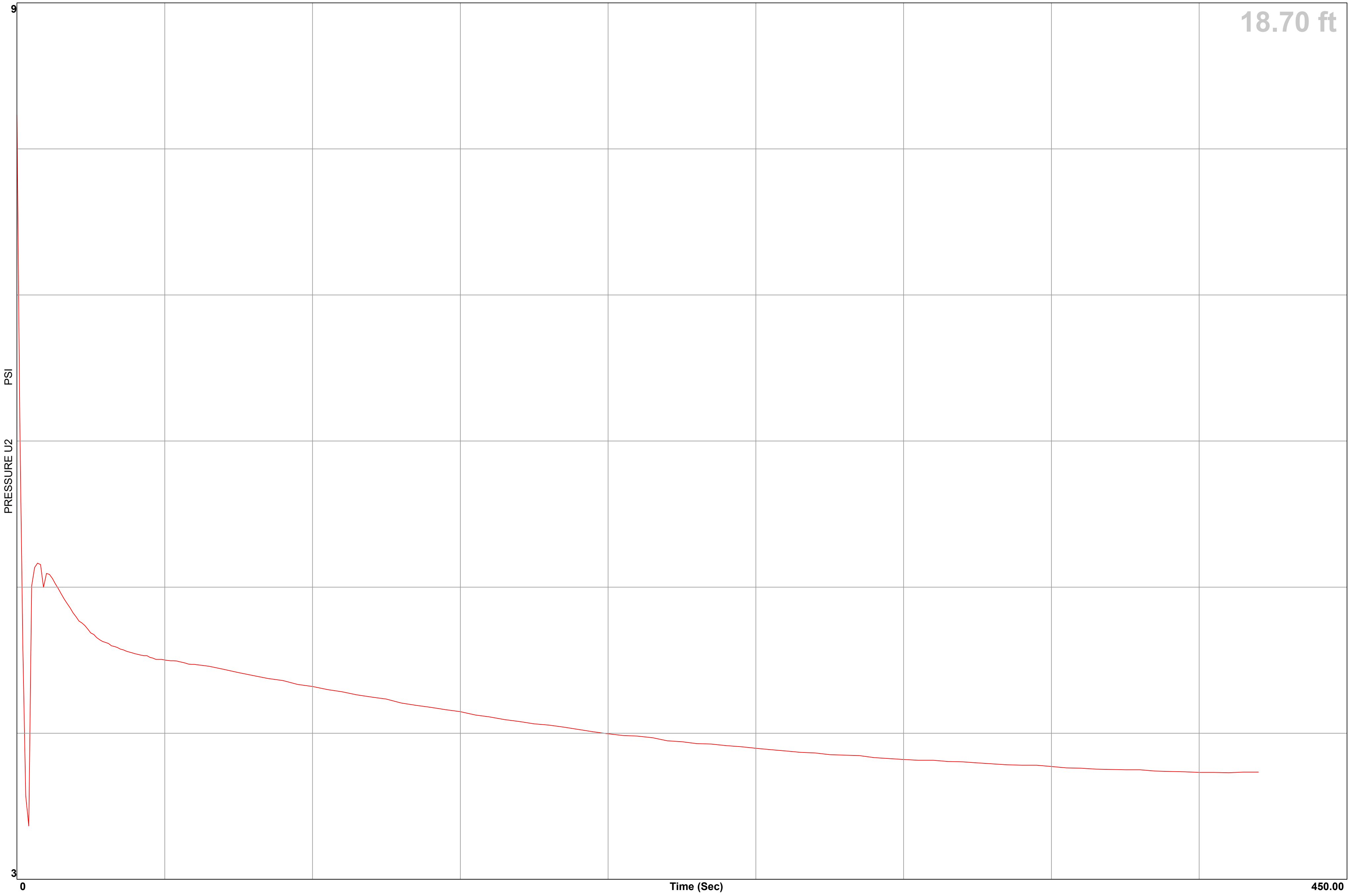
Figure **A-4**
Sheet **1 of 1**

Haley & Aldrich Inc.

Location New Leaf Minto Road Watsonville
Job Number 021059-003
Hole Number CPT-01
Equilized Pressure 3.7

Operator JM-IY
Cone Number DDG1589
Date and Time 11/1/2024 8:52:46 AM
EST GW Depth During Test 10.0

GPS



CPT: CPT-01

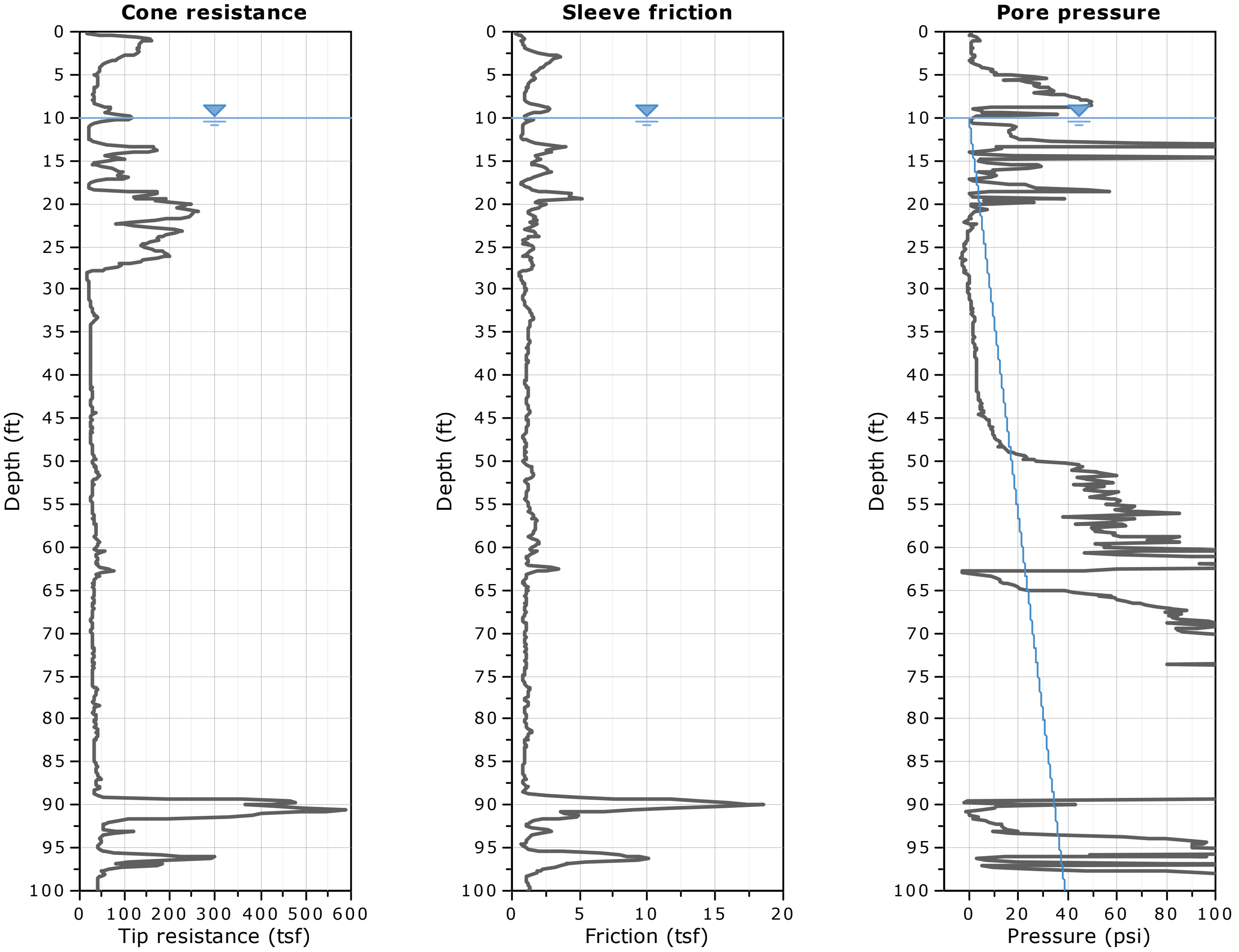
Total depth: 100.39 ft, Date: 11/1/2024

Cone Type: 15cm²

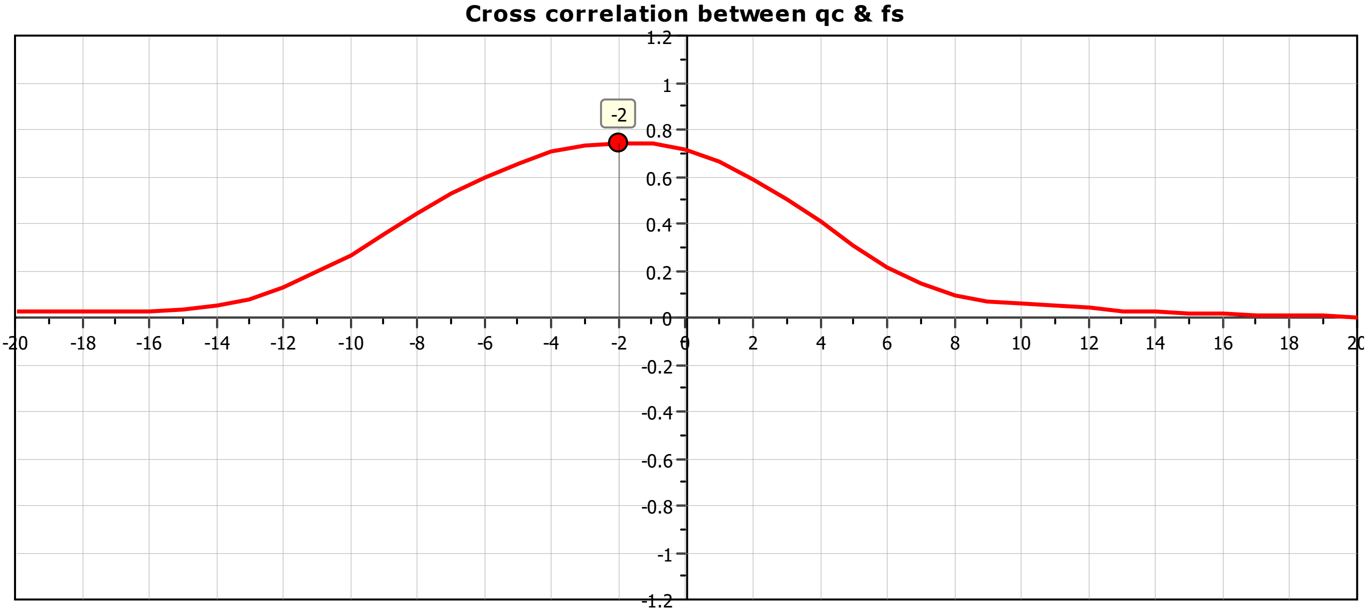
Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**

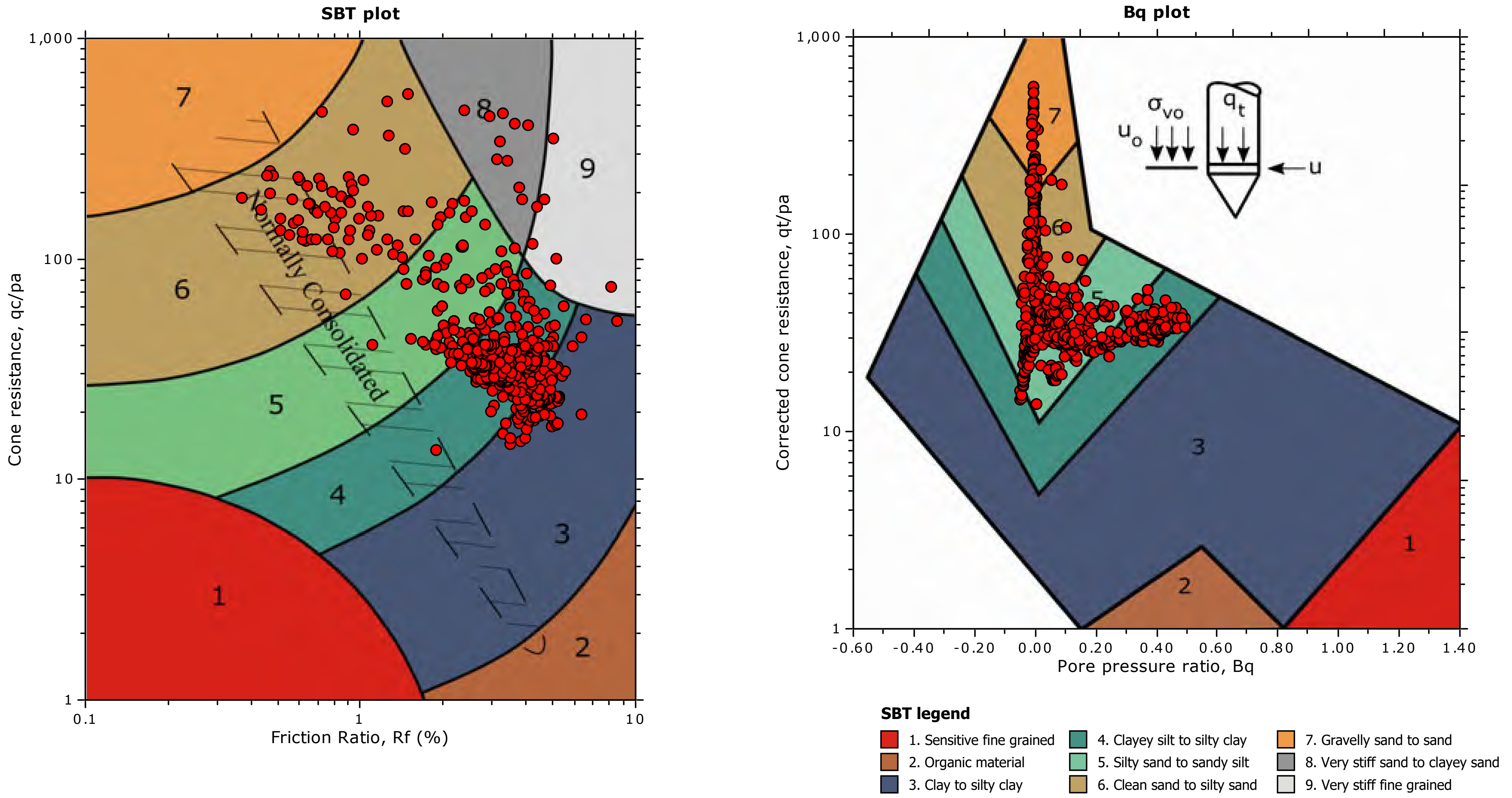
Location: **90 Minto Road, Watsonville, CA 95076**



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).

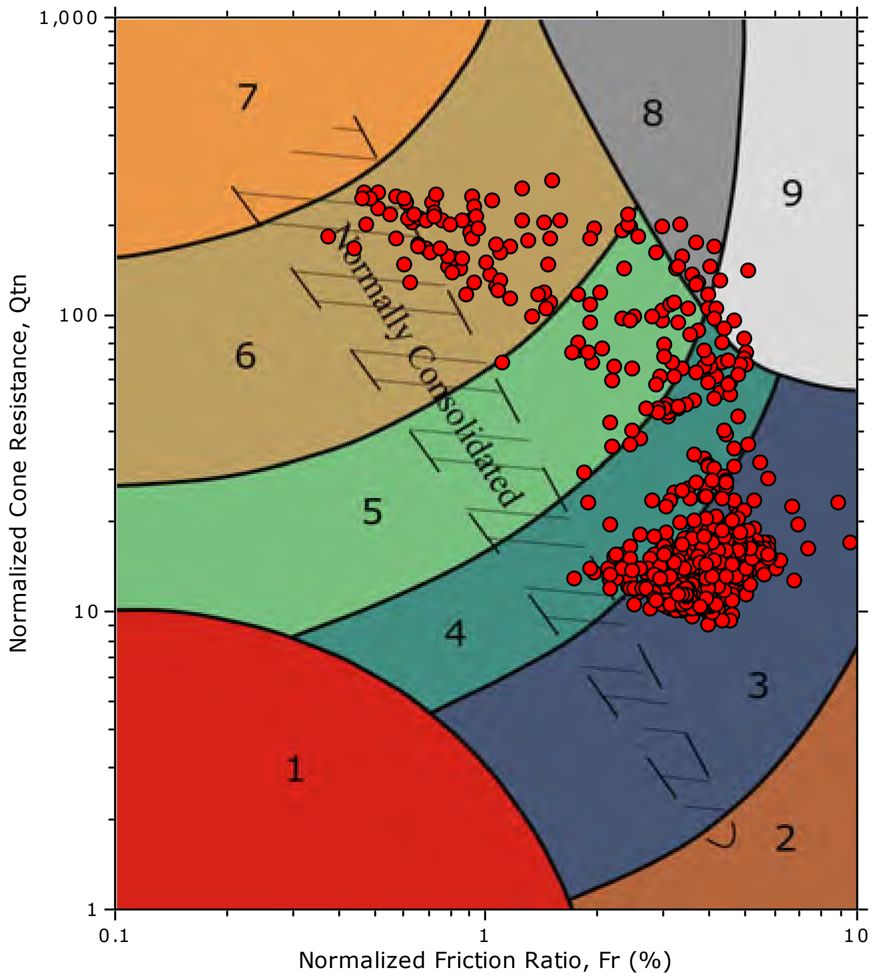


SBT - Bq plots

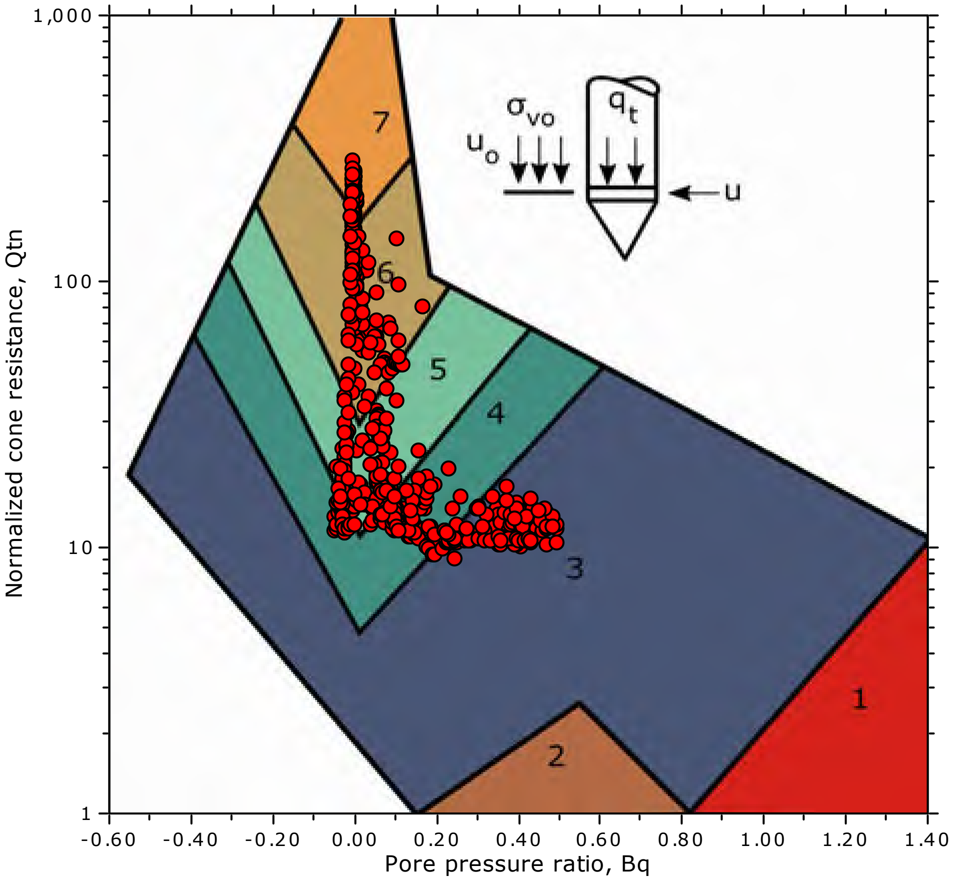

SBT plot
Cone resistance, qc/pa
1,000
100
10
1
0.1
1
10
Friction Ratio, Rf (%)
7
6
5
4
3
2
1
8
9
Normally Consolidated
Bq plot
Corrected cone resistance, qt/pa
1,000
100
10
1
-0.60
-0.40
-0.20
0.00
0.20
0.40
0.60
0.80
1.00
1.20
1.40
Pore pressure ratio, Bq
σ_{vo}
u_o
q_t
u
7
6
5
3
2
1
SBT legend
🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟫 6. Clean sand to silty sand
🟧 7. Gravelly sand to sand
⬜ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

SBT - Bq plots (normalized)

SBTn plot



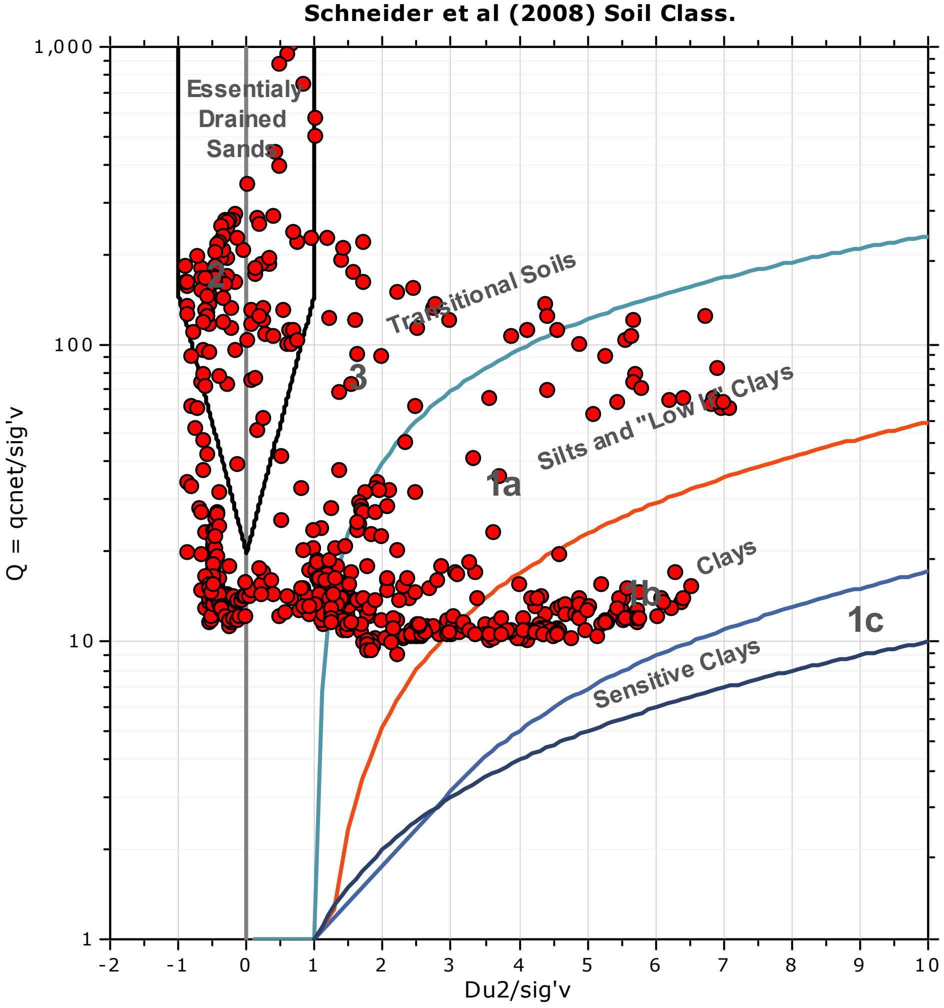
Normalized Bq plot

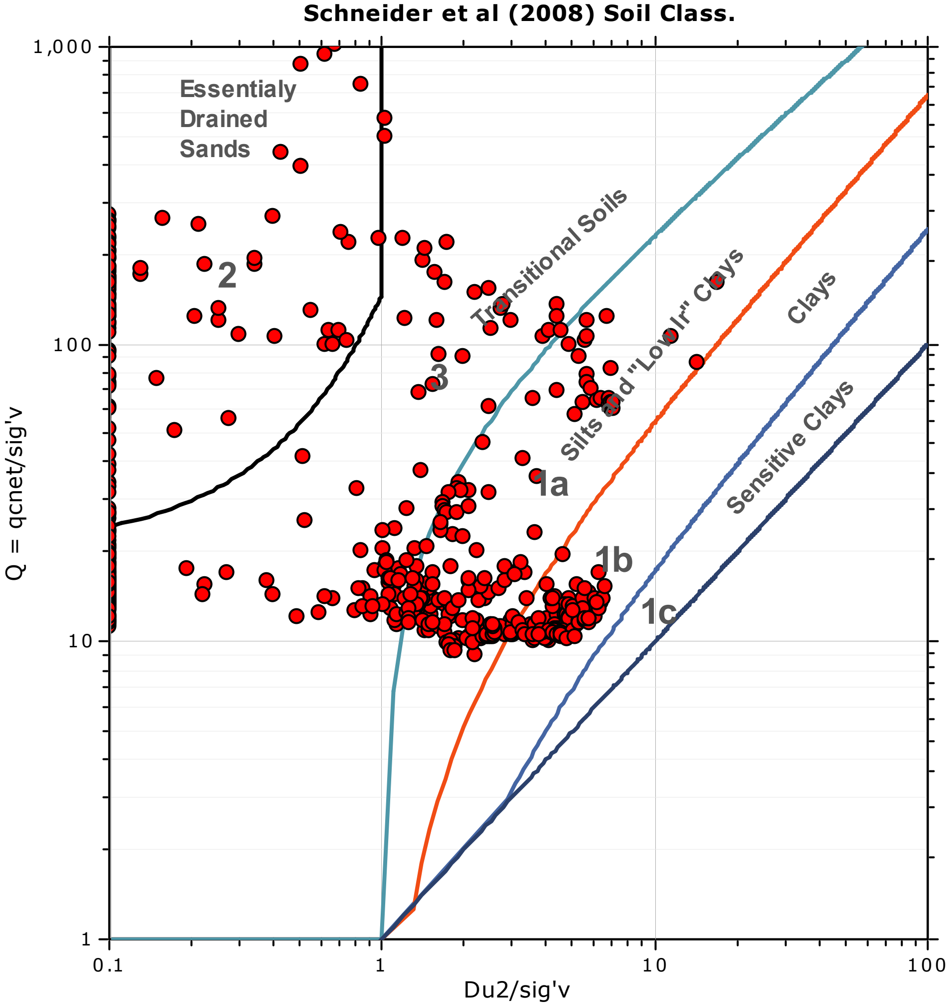


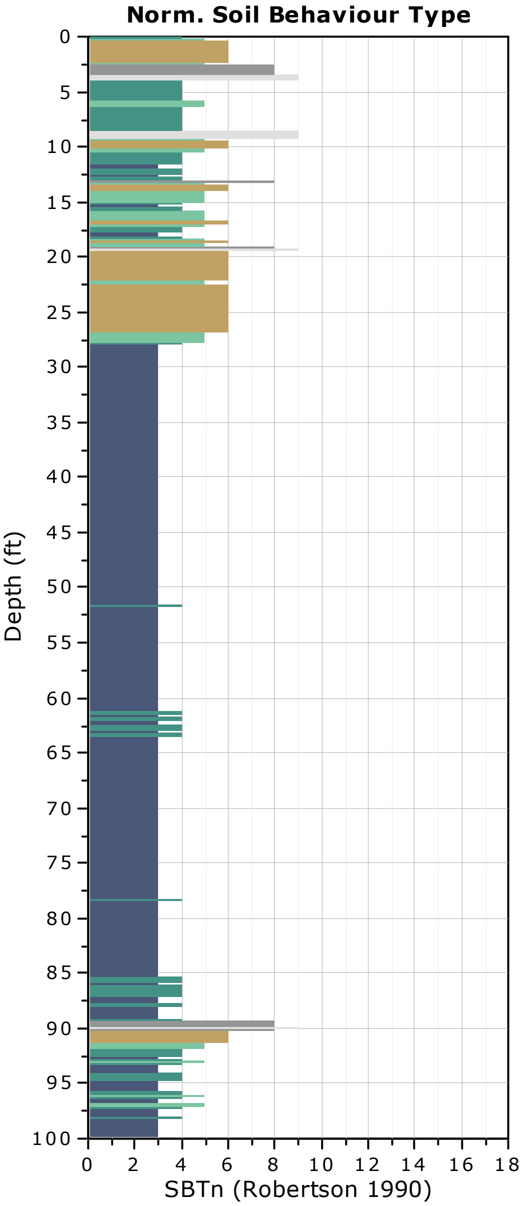
SBTn legend

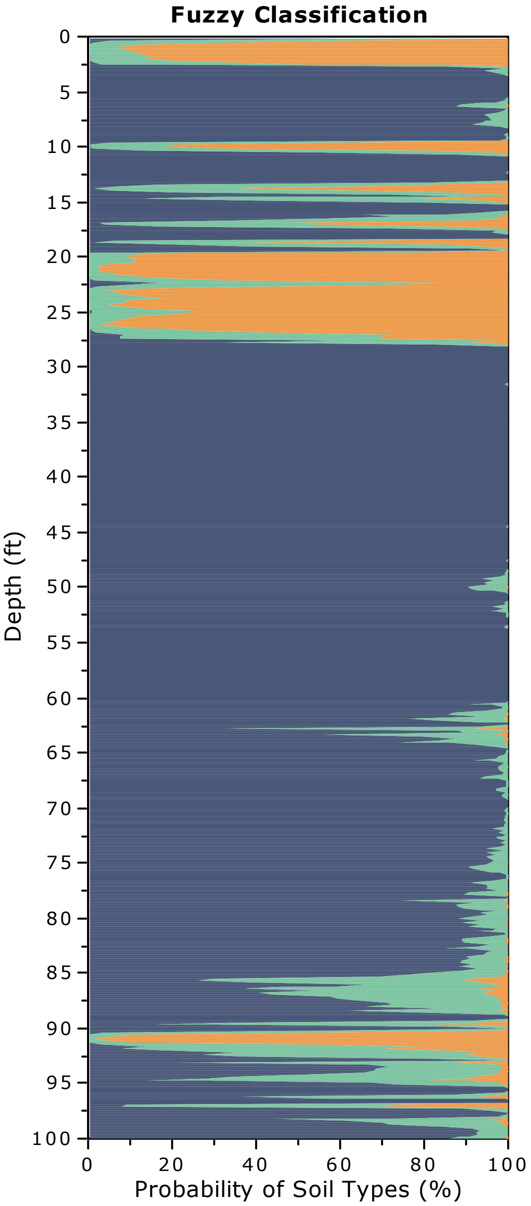
- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravelly sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

Bq plots (Schneider)









Fuzzy classifiaction legend

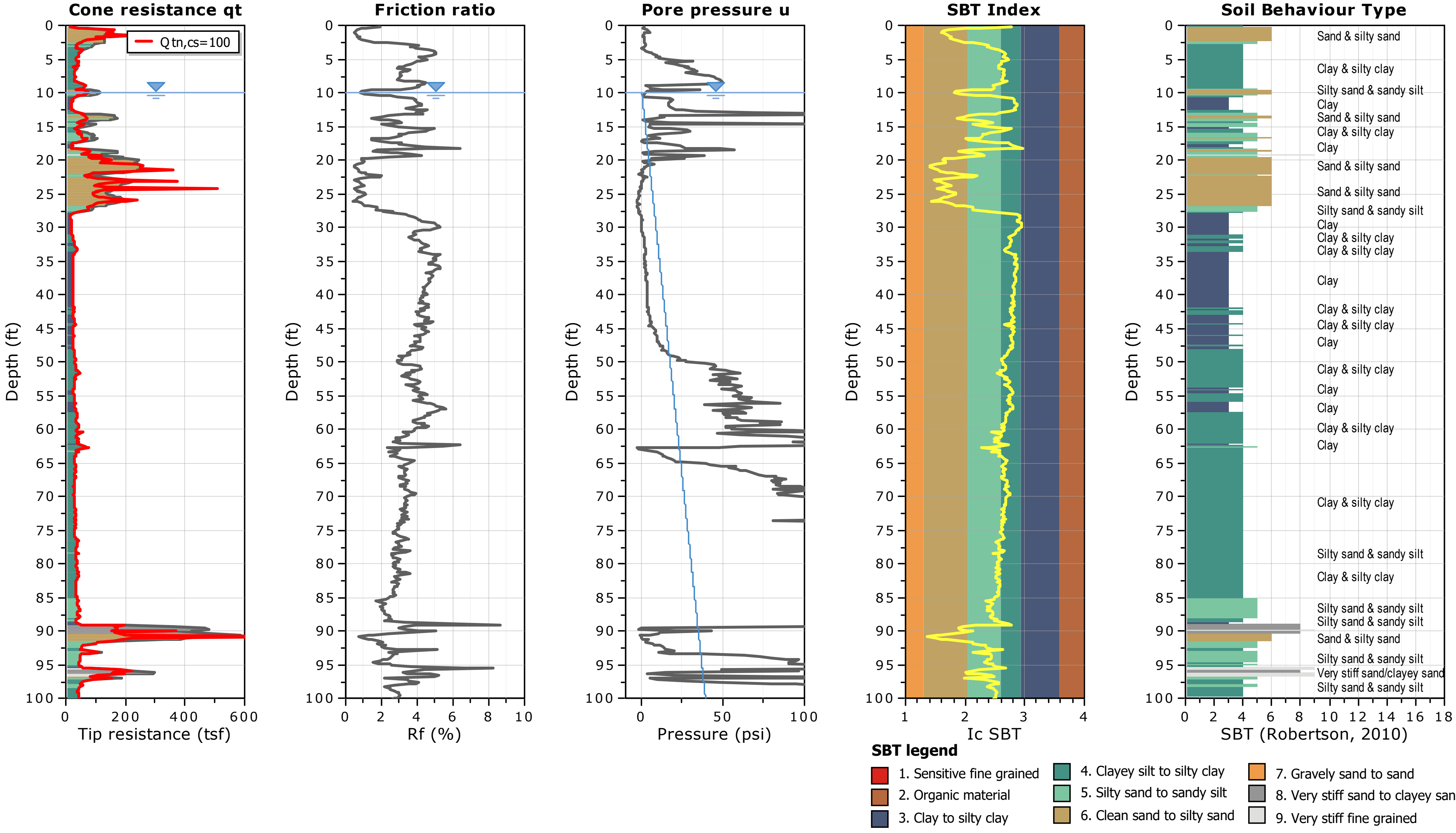
- 🟦 Highly probable clayey soil
- 🟩 Highly probable mixture soil
- 🟧 Highly probable sandy soil

Haley & Aldrich Inc.

https://www.haleyaldrich.com/

Project: **New Leaf Minto Road Watsonville**
Location: **90 Minto Road, Watsonville, CA 95076**

CPT: CPT-01
Total depth: 100.39 ft, Date: 11/1/2024
Cone Type: 15cm²
Cone Operator: JM-IY



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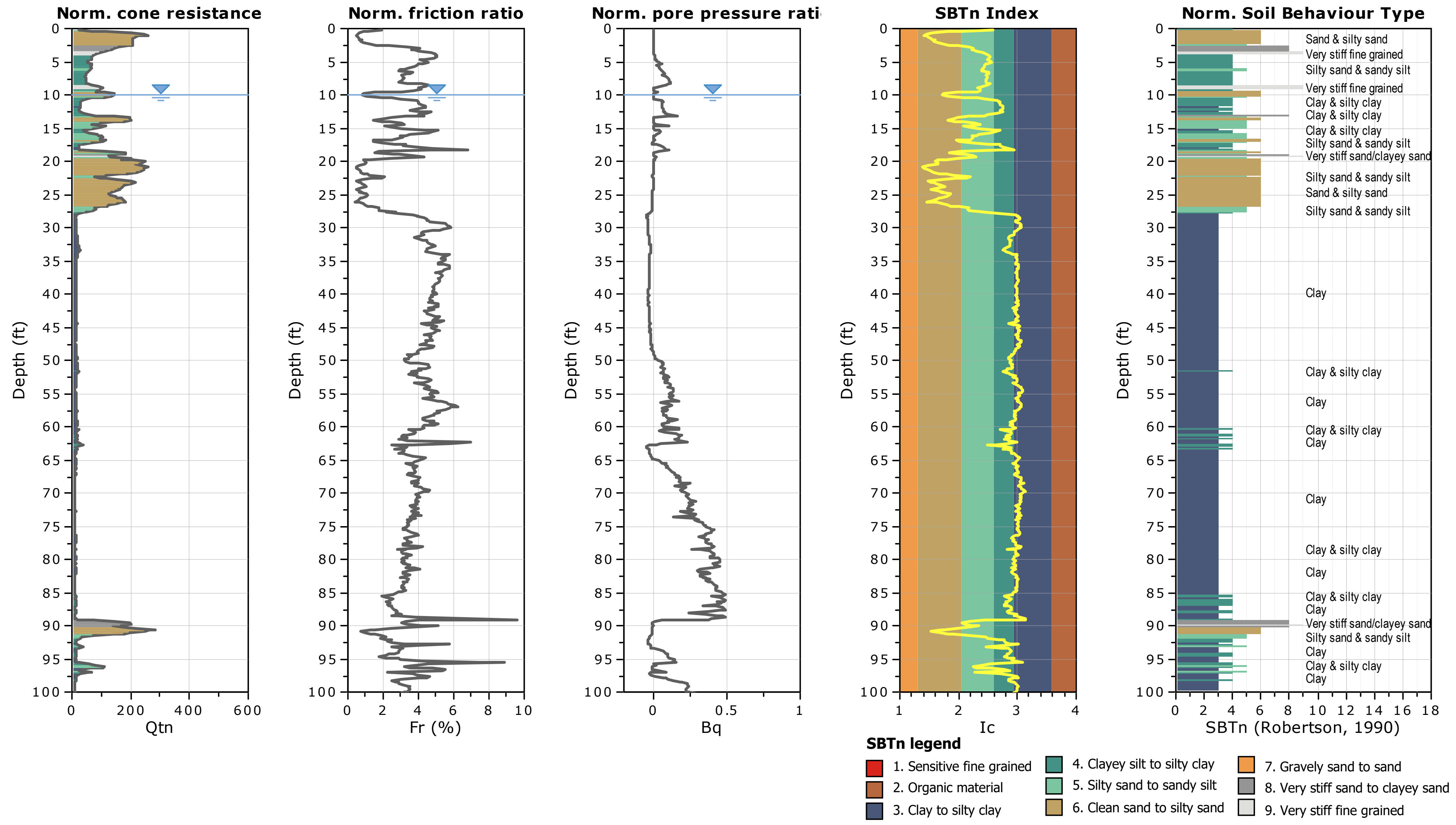

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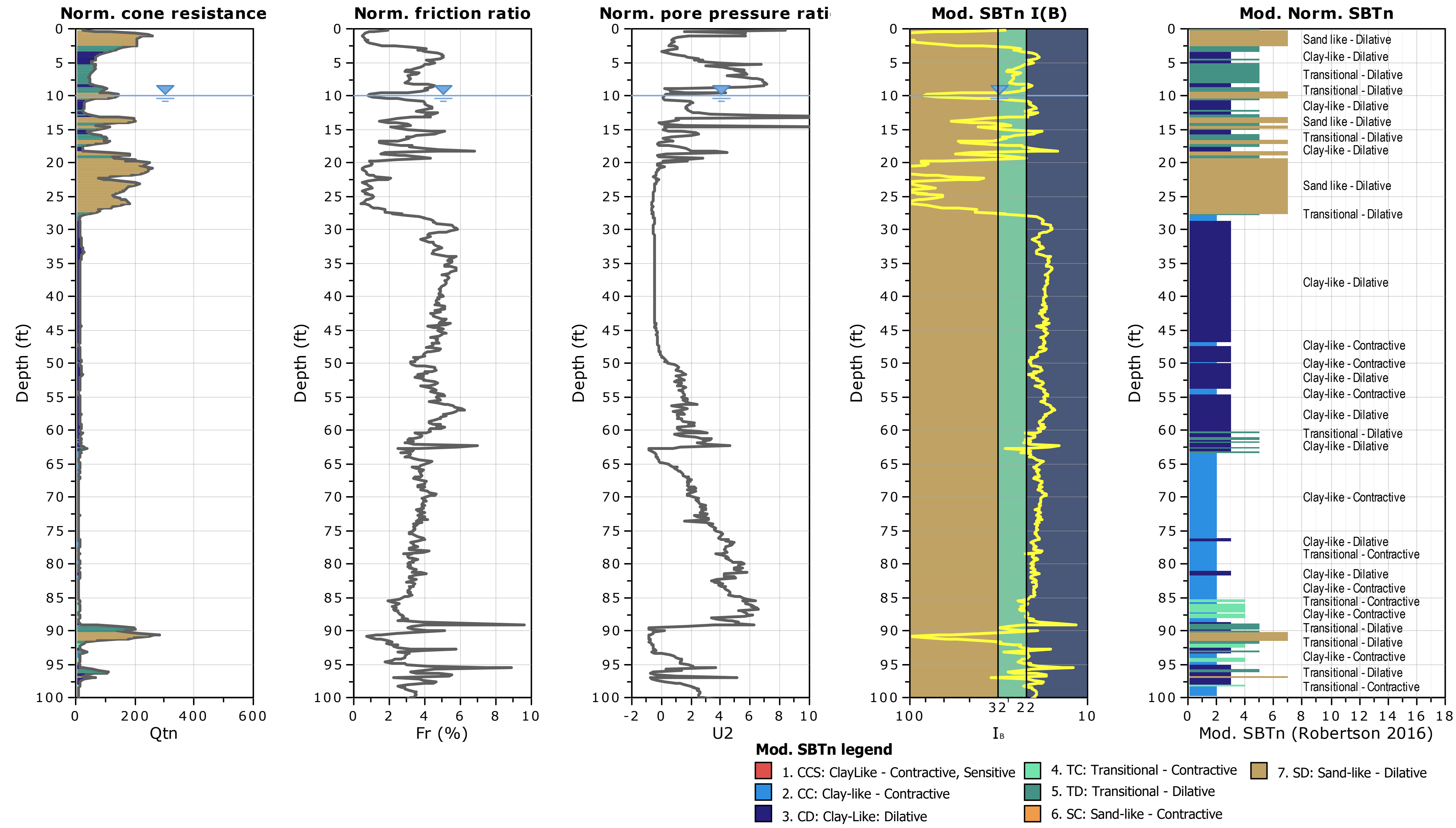
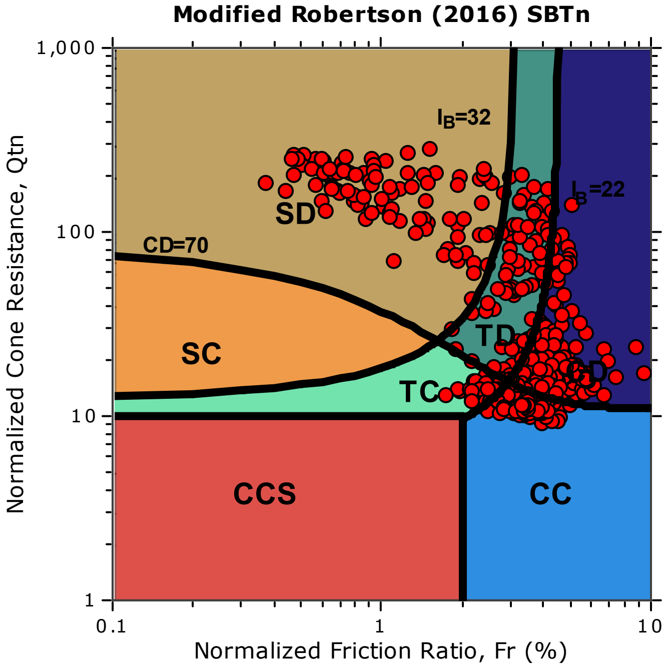
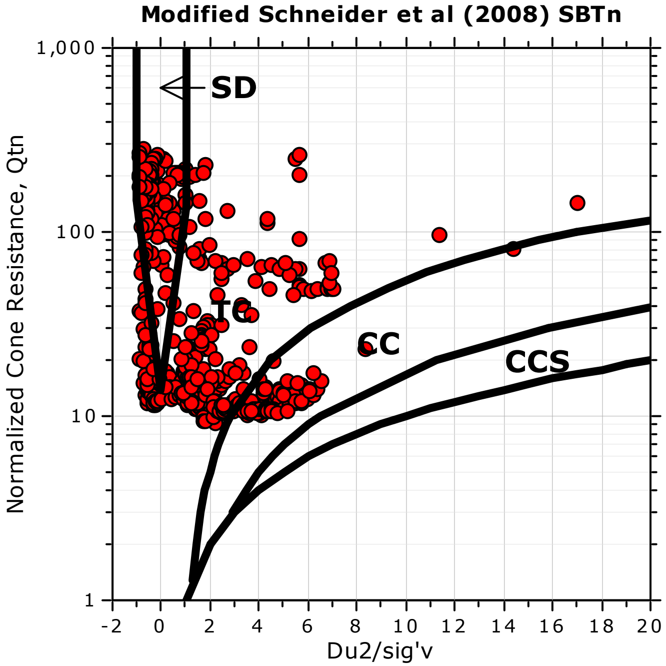
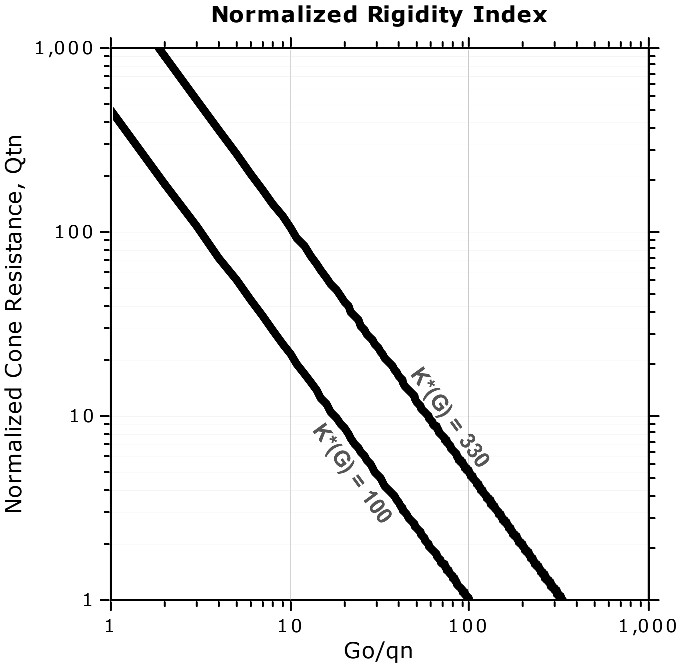


Updated SBTn plots







CCS: Clay-like - Contractive - Sensitive
CC: Clay-like - Contractive
CD: Clay-like - Dilative
TC: Transitional - Contractive
TD: Transitional - Dilative
SC: Sand-like - Contractive
SD: Sand-like - Dilative

K(G) > 330: Soils with significant microstructure
(e.g. age/cementation)

CPT: CPT-01

Total depth: 100.39 ft, Date: 11/1/2024

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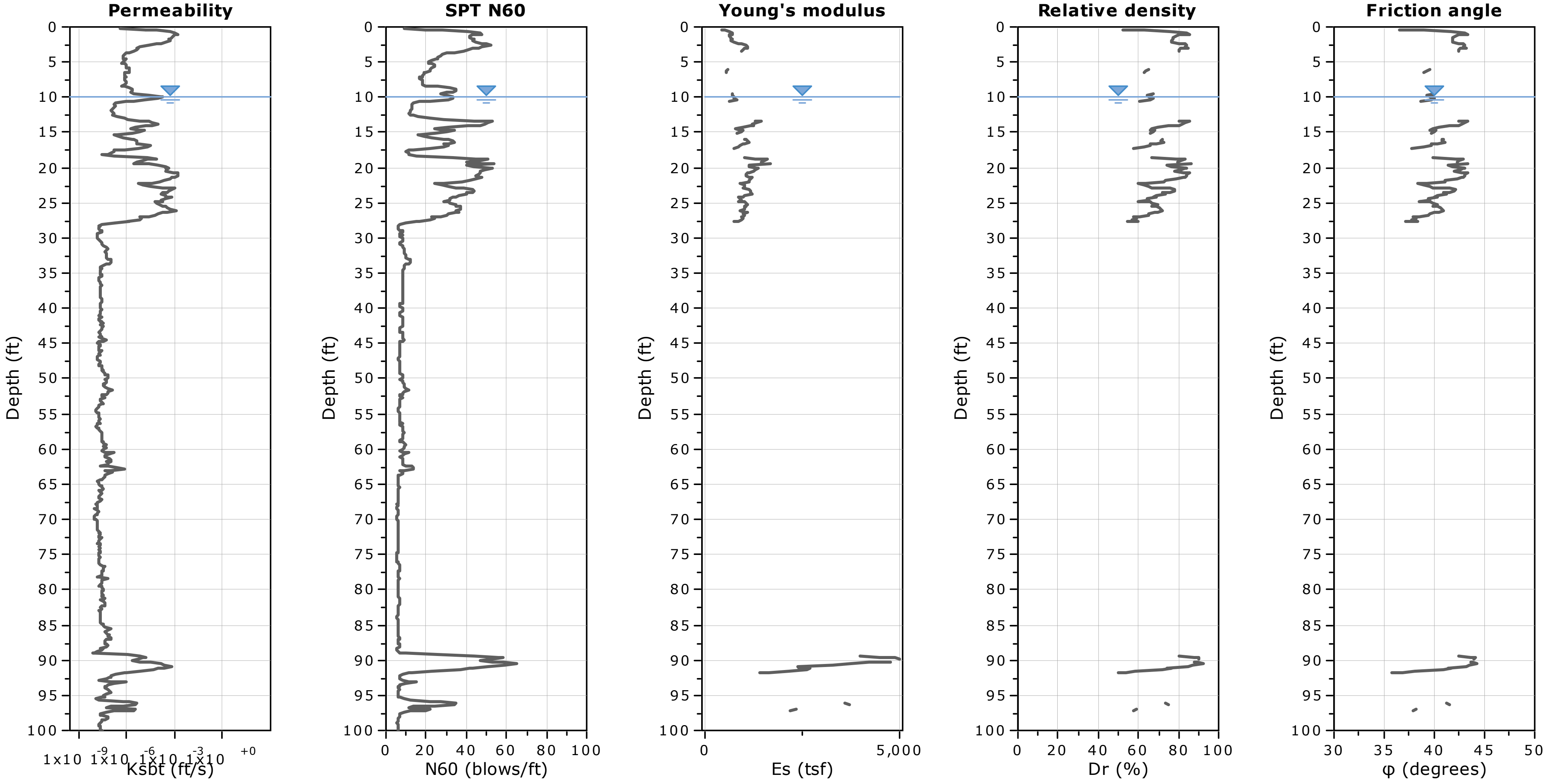

Calculation parameters

Permeability: Based on SBT_n

SPT N_{60}: Based on I_c and q_t

Young's modulus: Based on variable alpha using I_c (Robertson, 2009)

Relative density constant, C_{Dr}: 350.0

Phi: Based on Kulhawy & Mayne (1990)

CPT: CPT-01

Total depth: 100.39 ft, Date: 11/1/2024

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Cone Operator: JM-IY

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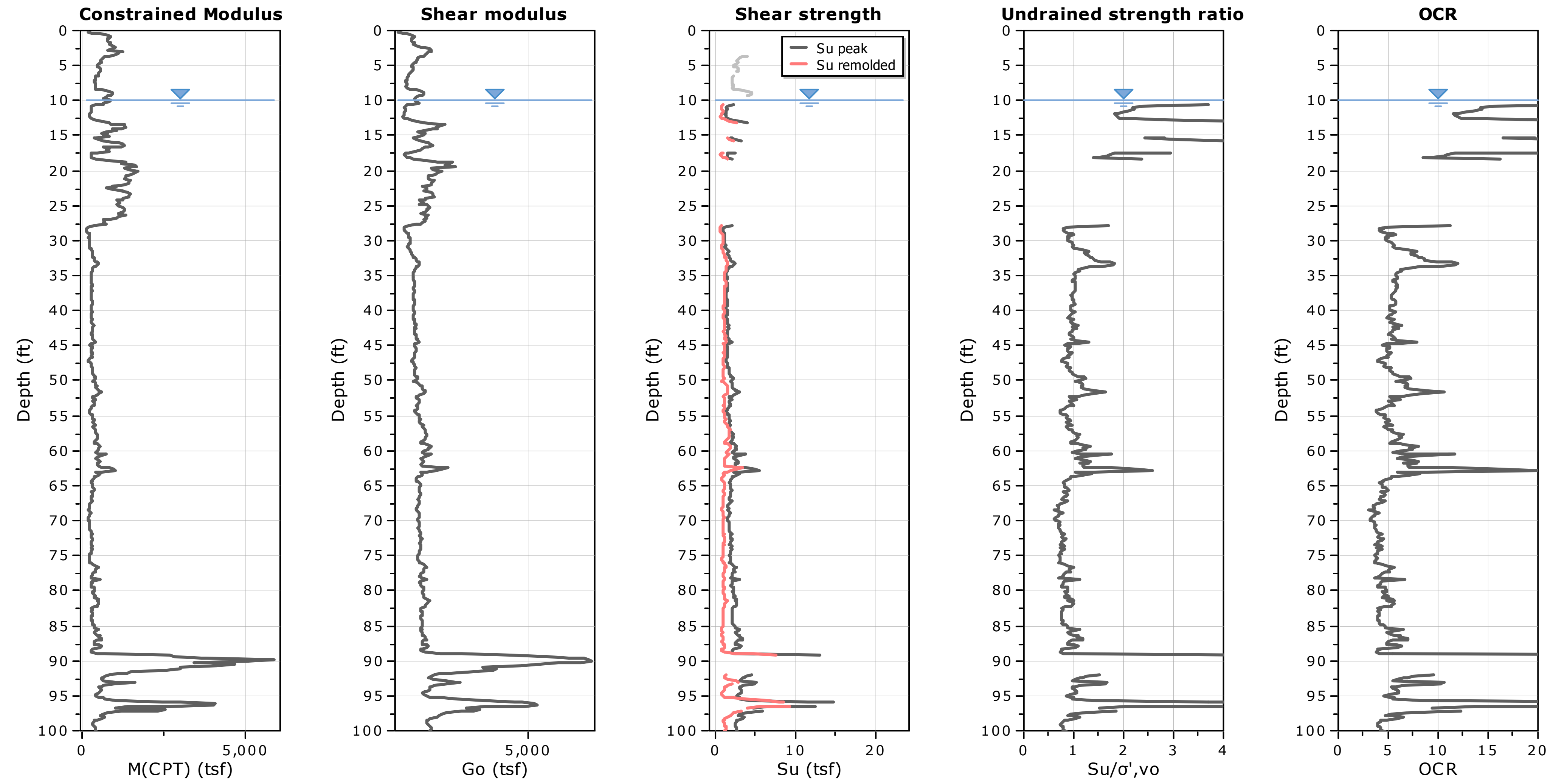

Calculation parameters

Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)

Go: Based on variable *alpha* using I_c (Robertson, 2009)

Undrained shear strength cone factor for clays, N_{kt}: Auto

OCR factor for clays, N_{kt}: Auto

—●— Flat Dilatometer Test data

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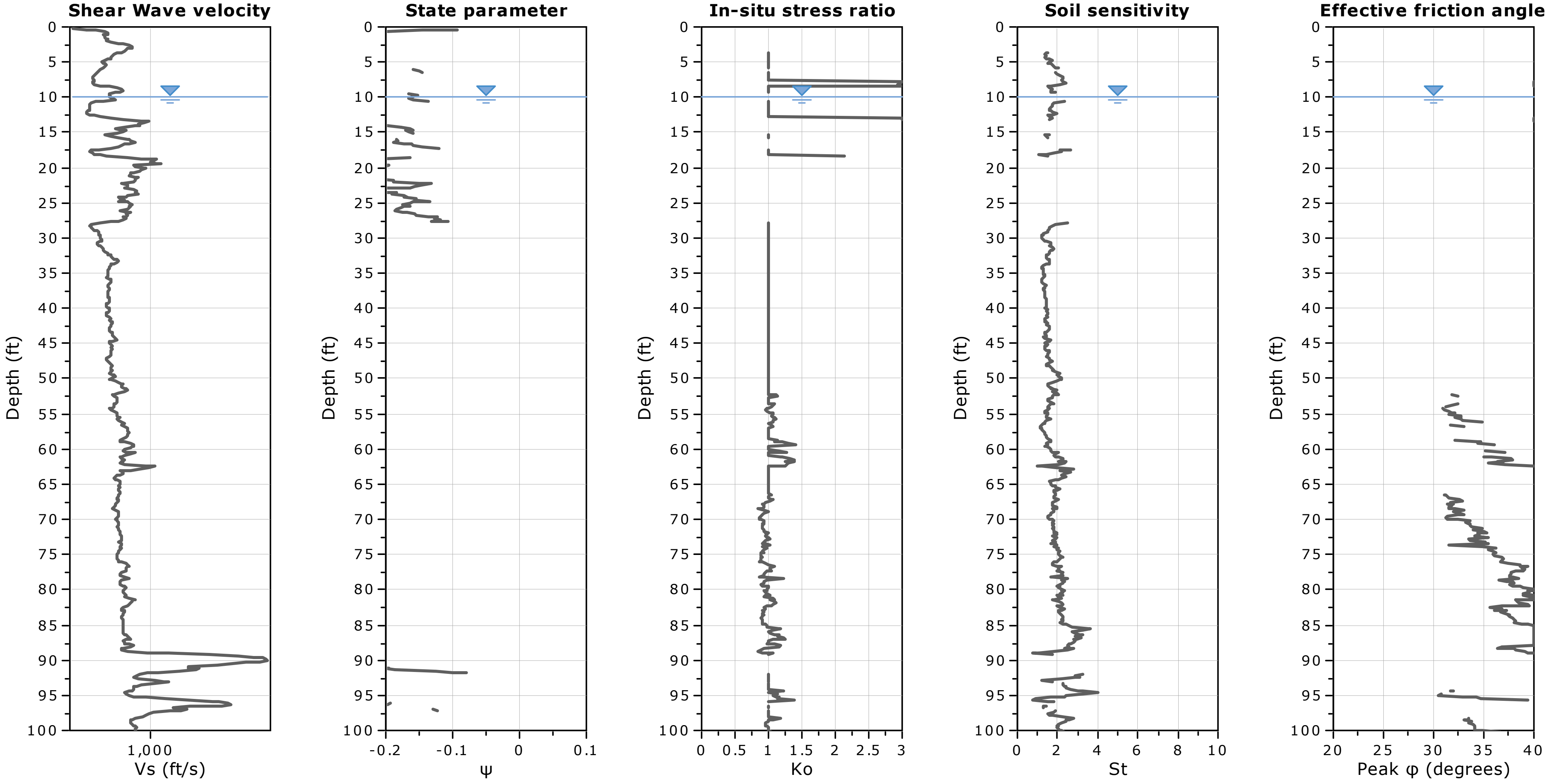

Calculation parameters

Soil Sensitivity factor, N_S: 7.00

CPT: CPT-01

Total depth: 100.39 ft, Date: 11/1/2024

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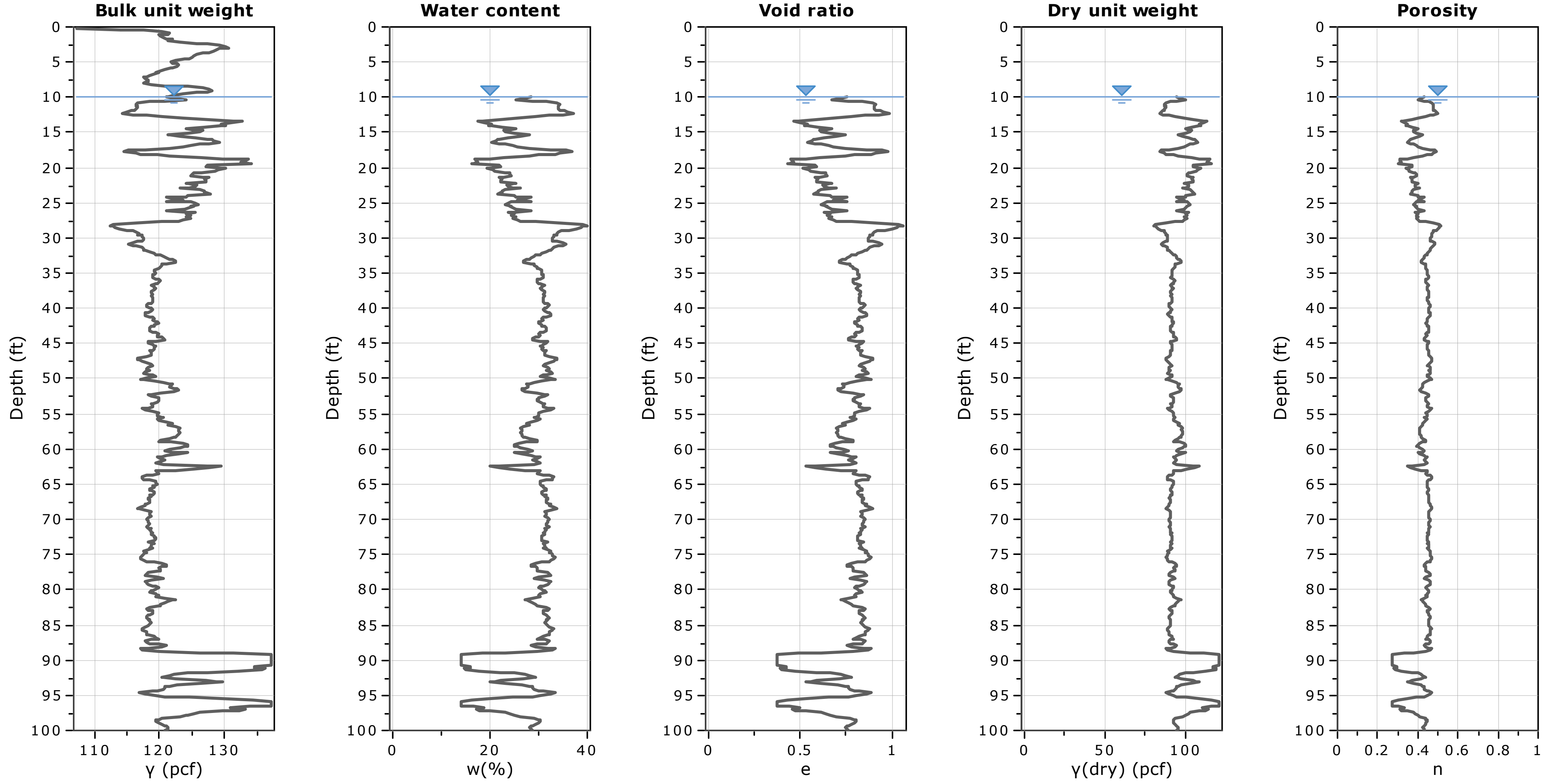

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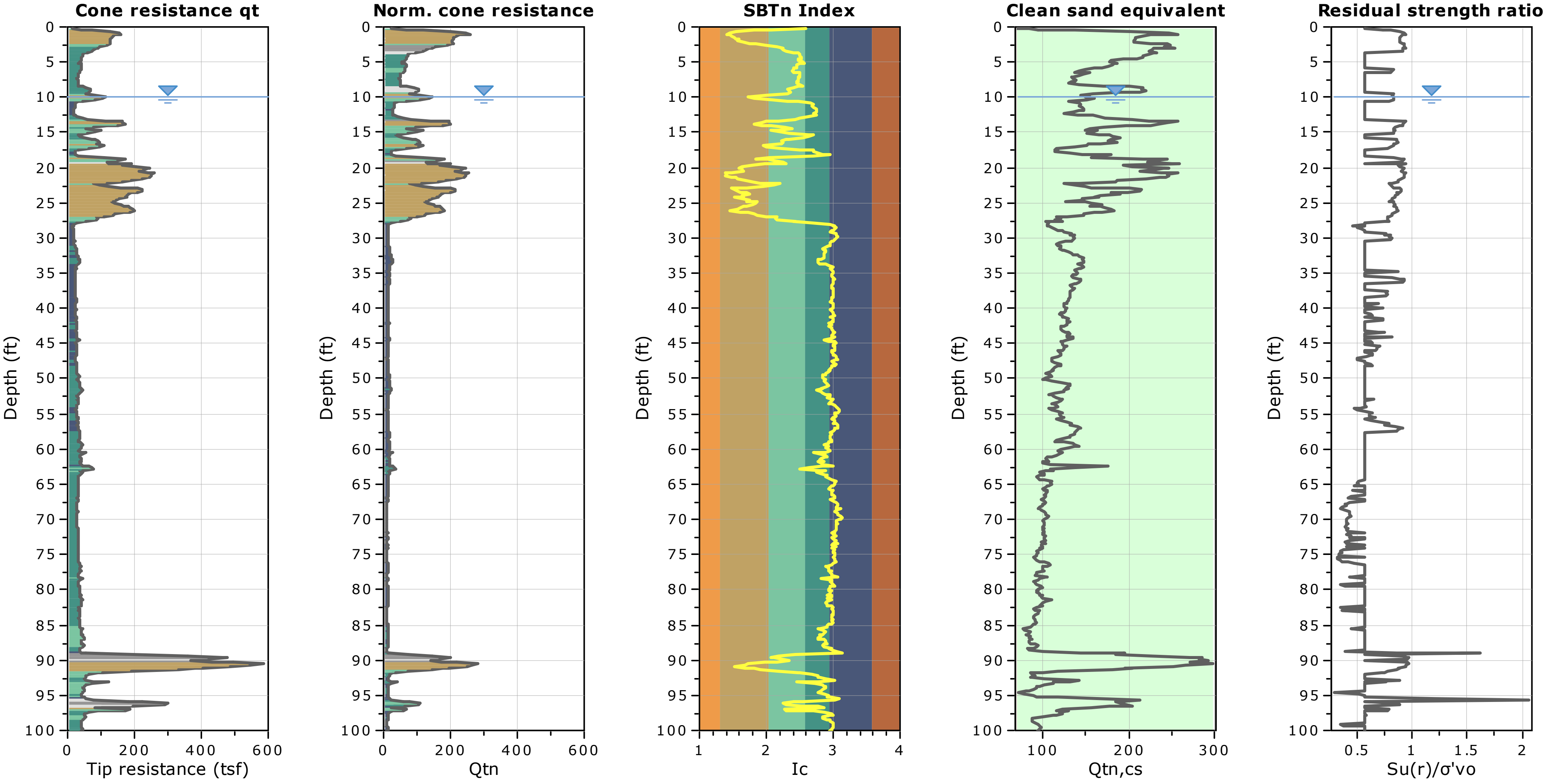
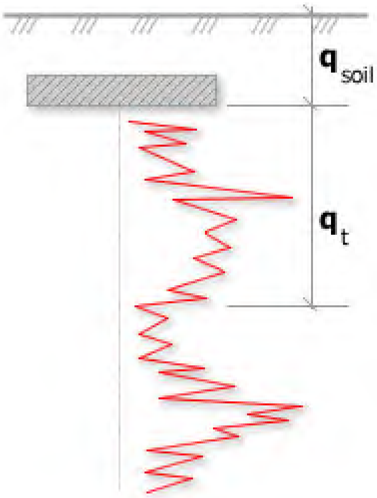
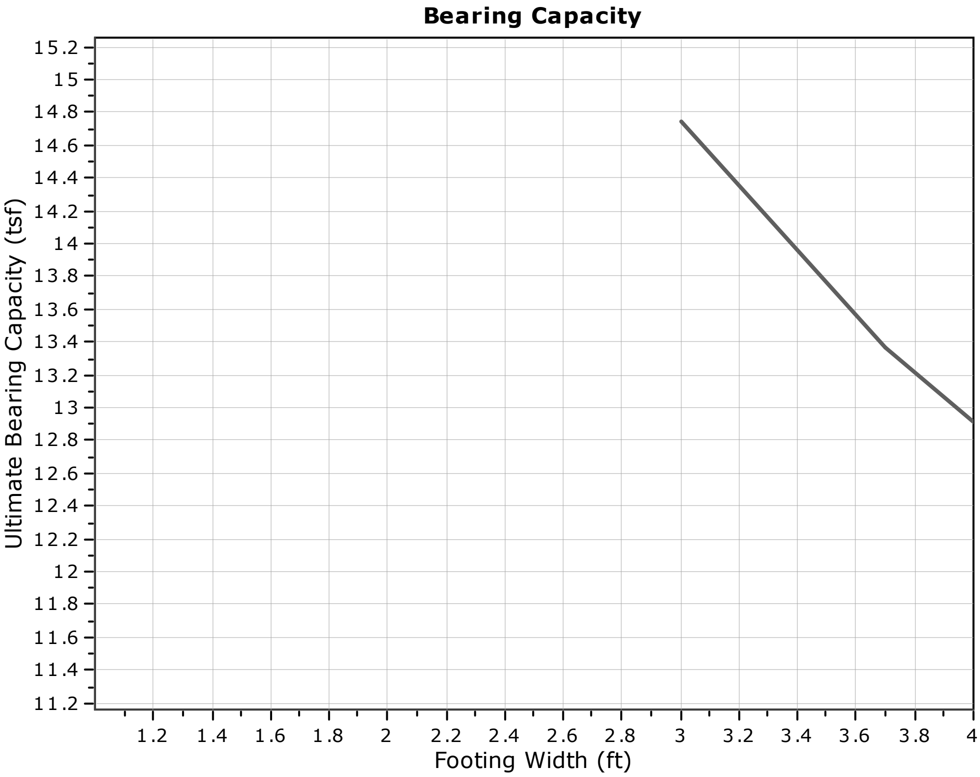




Bearing Capacity calculation is perfromed based on the formula:

$$Q_{ult} = R_k \times q_t + q_{soil}$$

where:
R_k: Bearing capacity factor
q_t: Average corrected cone resistance over calculation depth
q_{soil}: Pressure applied by soil above footing



:: Tabular results ::

No	B (ft)	Start Depth (ft)	End Depth (ft)	Ave. q_t (tsf)	R_k	Soil Press. (tsf)	Ult. bearing cap. (tsf)
1	3.00	1.60	6.10	73.23	0.20	0.10	14.74
2	3.70	1.60	7.15	66.36	0.20	0.10	13.37
3	4.40	1.60	8.20	61.07	0.20	0.10	12.31
4	5.10	1.60	9.25	59.67	0.20	0.10	12.03
5	5.80	1.60	10.30	62.76	0.20	0.10	12.65
6	6.50	1.60	11.35	59.63	0.20	0.10	12.02
7	7.20	1.60	12.40	56.05	0.20	0.10	11.31
8	7.90	1.60	13.45	55.44	0.20	0.10	11.18
9	8.60	1.60	14.50	61.82	0.20	0.10	12.46
10	9.30	1.60	15.55	61.99	0.20	0.10	12.49
11	10.00	1.60	16.60	62.81	0.20	0.10	12.66
12	10.70	1.60	17.65	63.08	0.20	0.10	12.71
13	11.40	1.60	18.70	61.62	0.20	0.10	12.42
14	12.10	1.60	19.75	67.51	0.20	0.10	13.60
15	12.80	1.60	20.80	75.68	0.20	0.10	15.23

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236\right)$$

where g_w = water unit weight

:: Permeability, k (m/s) ::

$I_c < 3.27$ and $I_c > 1.00$ then $k = 10^{0.952\text{-}3.04 \cdot I_c}$

$I_c \leq 4.00$ and $I_c > 3.27$ then $k = 10^{\text{-}4.52\text{-}1.37 \cdot I_c}$

:: N_{SPT} (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268 - 0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268 - 0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT$_n$: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Drained Friction Angle, φ (°) ::

$$\varphi = \varphi'_{cv} + 15.94 \cdot \log(Q_{tn,cs}) - 26.88$$

(applicable only to SBT$_n$: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$
$\alpha = 14$ for $Q_{tn} > 14$
$\alpha = Q_{tn}$ for $Q_{tn} \leq 14$
$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$

If $I_c \geq 2.20$

$$M_{CPT} = 0.03 \cdot (q_t - \sigma_v) \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Small strain shear Modulus, Go (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Shear Wave Velocity, Vs (m/s) ::

$$V_s = \left(\frac{G_0}{\rho}\right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot + 7 \cdot \log(F_r))}\right]^{1.25}$$ or user defined

$$OCR = k_{OCR} \cdot Q_{tn}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_O = (1 - \sin\varphi') \cdot OCR^{\sin\varphi'}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Soil Sensitivity, S_t ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Peak Friction Angle, φ' (°) ::

$$\varphi' = 29.5° \cdot B_q^{0.121} \cdot (0.256 + 0.336 \cdot B_q + \log Q_t)$$

(applicable for $0.10 < B_q < 1.00$)

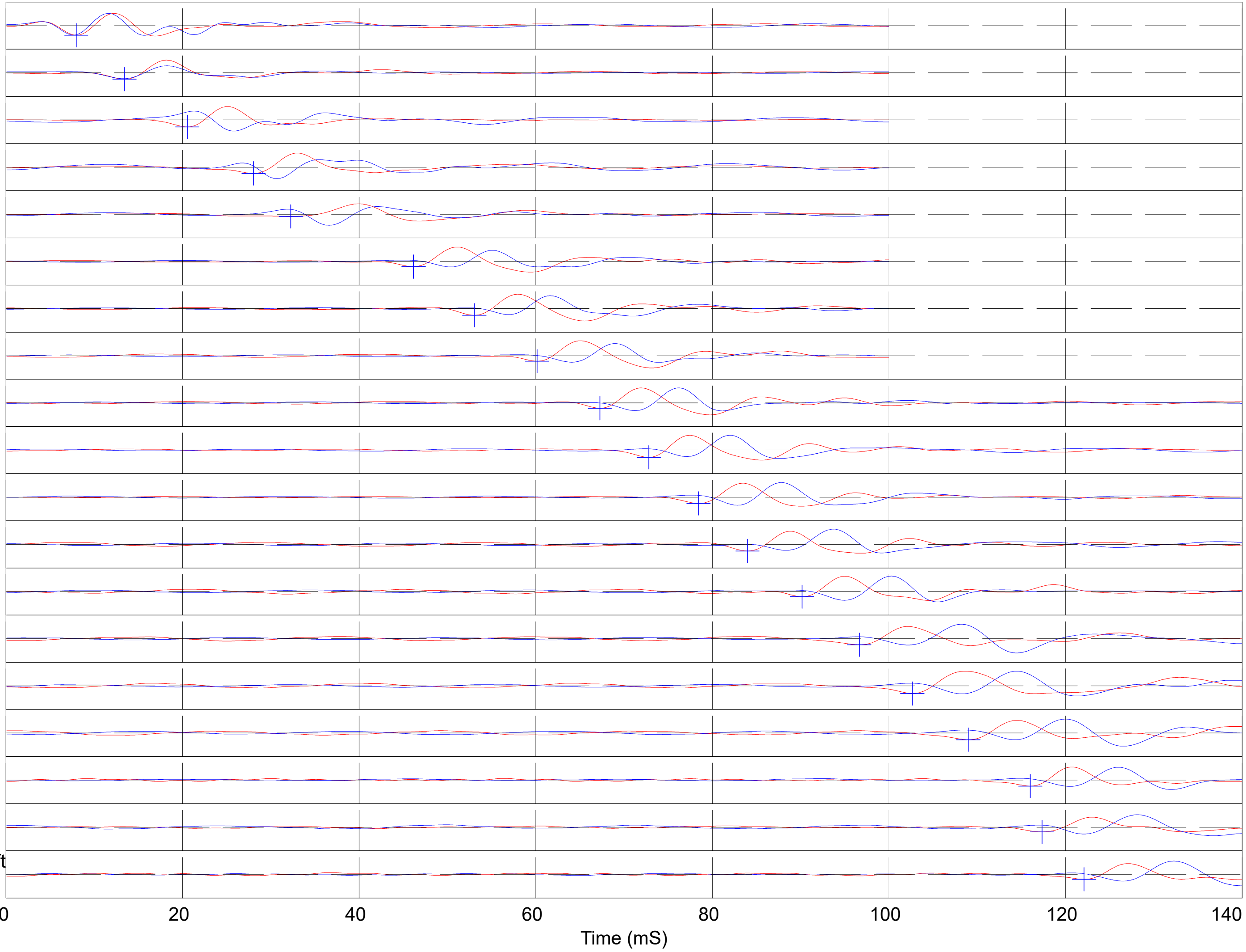
References

- Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 5th Edition, November 2012
- Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)
- N Barounis, J Philpot, Estimation of in-situ water content, void ratio, dry unit weight and porosity using CPT for saturated sands, Proc. 20th NZGS Geotechnical Symposium

CPT-01

Haley & Aldrich Inc.

New Leaf Minto Road Watsonville



Depth 5.09ft Ref*	Arrival 7.97mS Velocity*
Depth 10.01ft Ref 5.09ft	Arrival 13.44mS Velocity 861.46ft/S
Depth 15.06ft Ref 10.01ft	Arrival 20.55mS Velocity 699.92ft/S
Depth 20.05ft Ref 15.06ft	Arrival 28.05mS Velocity 659.85ft/S
Depth 25.16ft Ref 20.05ft	Arrival 32.26mS Velocity 1207.65ft/S
Depth 35.04ft Ref 25.16ft	Arrival 46.17mS Velocity 708.30ft/S
Depth 40.09ft Ref 35.04ft	Arrival 53.04mS Velocity 733.73ft/S
Depth 45.05ft Ref 40.09ft	Arrival 60.15mS Velocity 695.98ft/S
Depth 50.03ft Ref 45.05ft	Arrival 67.26mS Velocity 700.77ft/S
Depth 55.05ft Ref 50.03ft	Arrival 72.81mS Velocity 904.25ft/S
Depth 60.04ft Ref 55.05ft	Arrival 78.43mS Velocity 885.98ft/S
Depth 65.06ft Ref 60.04ft	Arrival 83.98mS Velocity 904.47ft/S
Depth 70.05ft Ref 65.06ft	Arrival 90.15mS Velocity 807.64ft/S
Depth 75.03ft Ref 70.05ft	Arrival 96.63mS Velocity 768.77ft/S
Depth 80.05ft Ref 75.03ft	Arrival 102.65mS Velocity 834.17ft/S
Depth 85.04ft Ref 80.05ft	Arrival 108.98mS Velocity 787.83ft/S
Depth 93.08ft Ref 85.04ft	Arrival 116.01mS Velocity 1142.93ft/S
Depth 95.05ft Ref 93.08ft	Arrival 117.34mS Velocity 1481.86ft/S
Depth 100.07ft Ref 95.05ft	Arrival 122.10mS Velocity 1053.12ft/S

Hammer to Rod String Distance (ft): 2.17
* = Not Determined

COMMENT:

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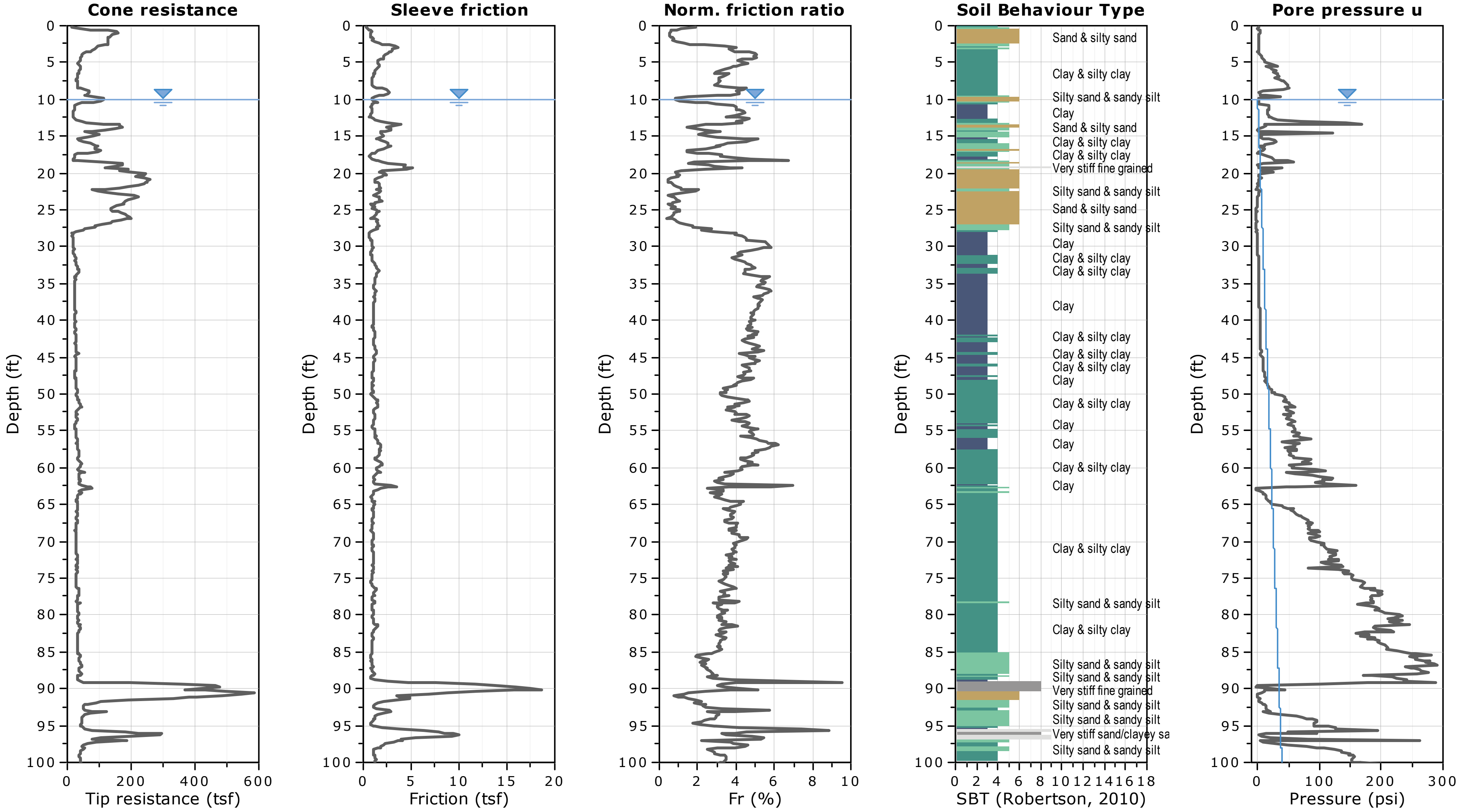
Total depth: 100.39 ft, Date: 11/1/2024

Cone Type: 15cm²

Cone Operator: JM-IY

Project: New Leaf Minto Road Watsonville

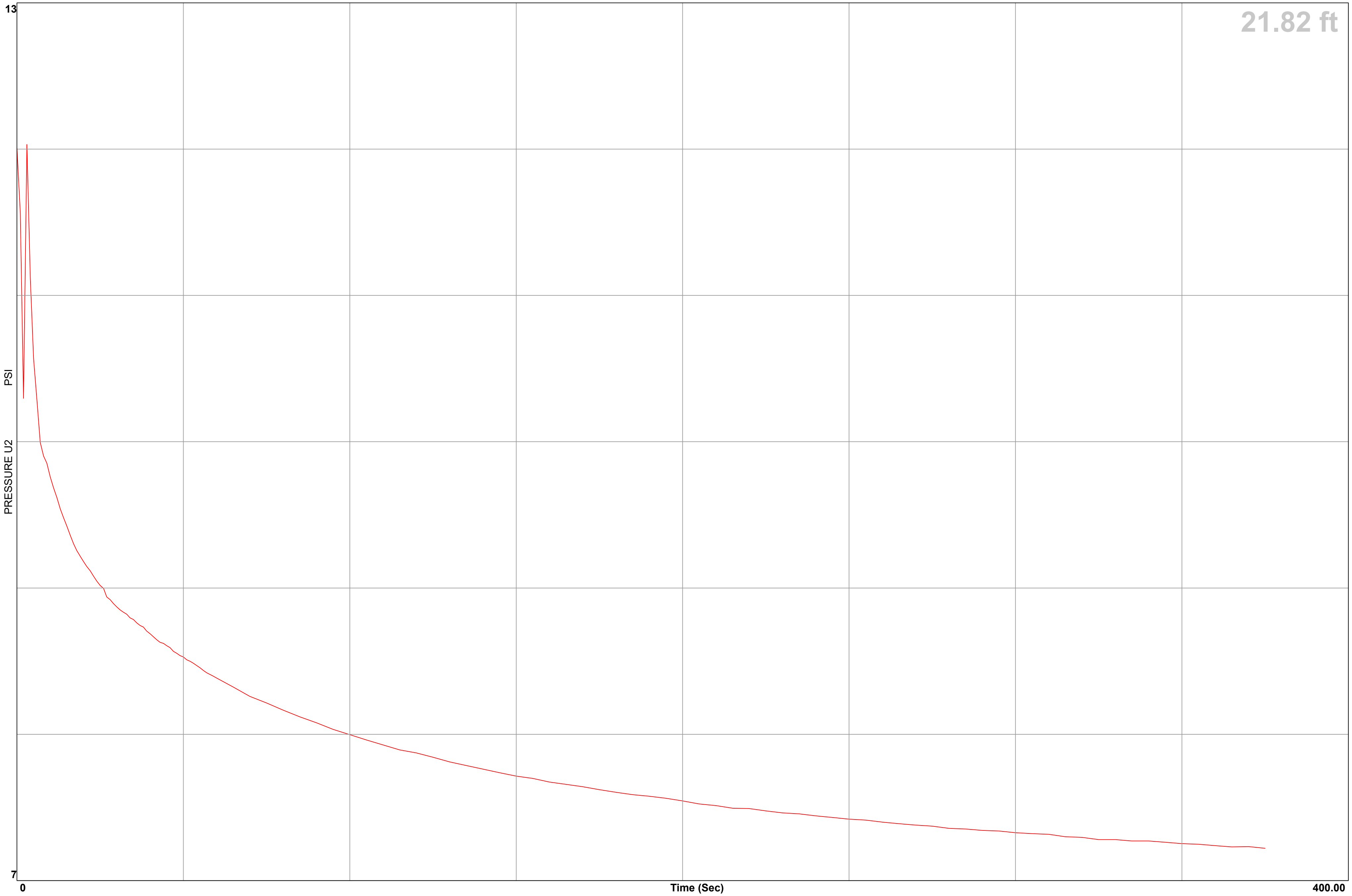
Location: 90 Minto Road, Watsonville, CA 95076



Haley & Aldrich Inc.

Location	New Leaf Minto Road Watsonville	Operator	JM-IY
Job Number	021059-003	Cone Number	DDG1589
Hole Number	CPT-02	Date and Time	11/1/2024 11:06:08 AM
Equilized Pressure	7.2	EST GW Depth During Test	5.1

GPS



Haley & Aldrich Inc.

https://www.haleyaldrich.com/

CPT: CPT-02

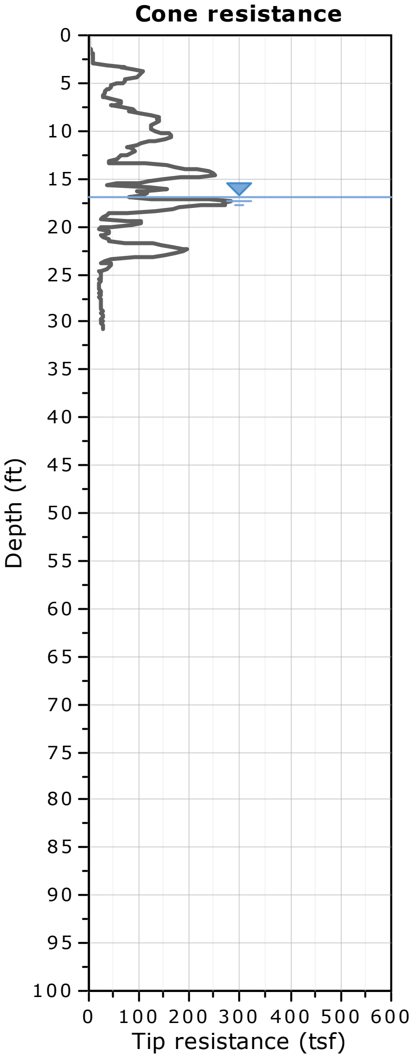
Total depth: 30.68 ft, Date: 11/1/2024

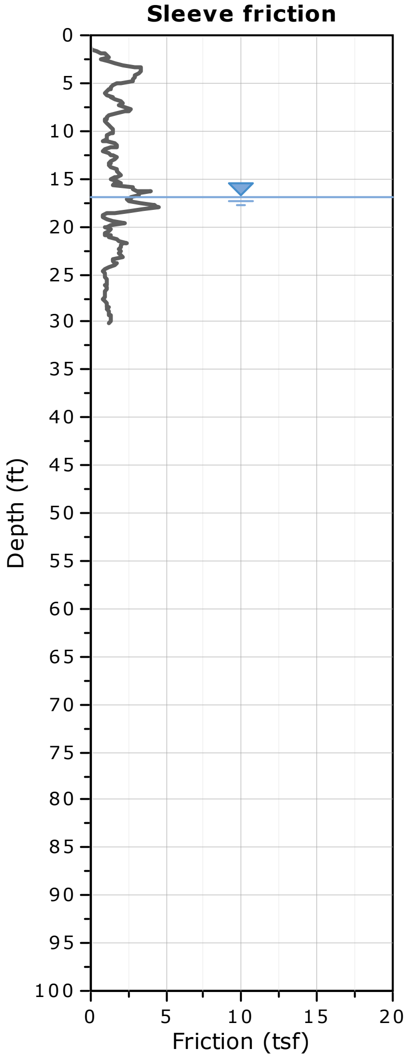
Cone Type: 15cm²

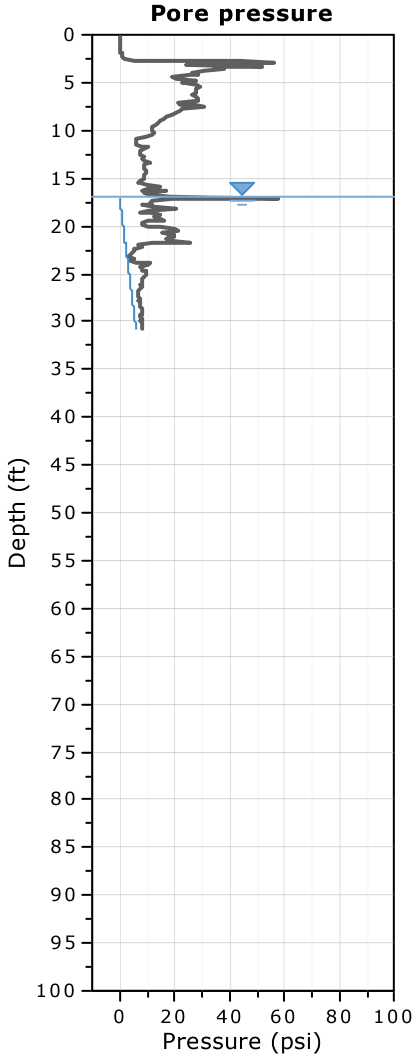
Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**

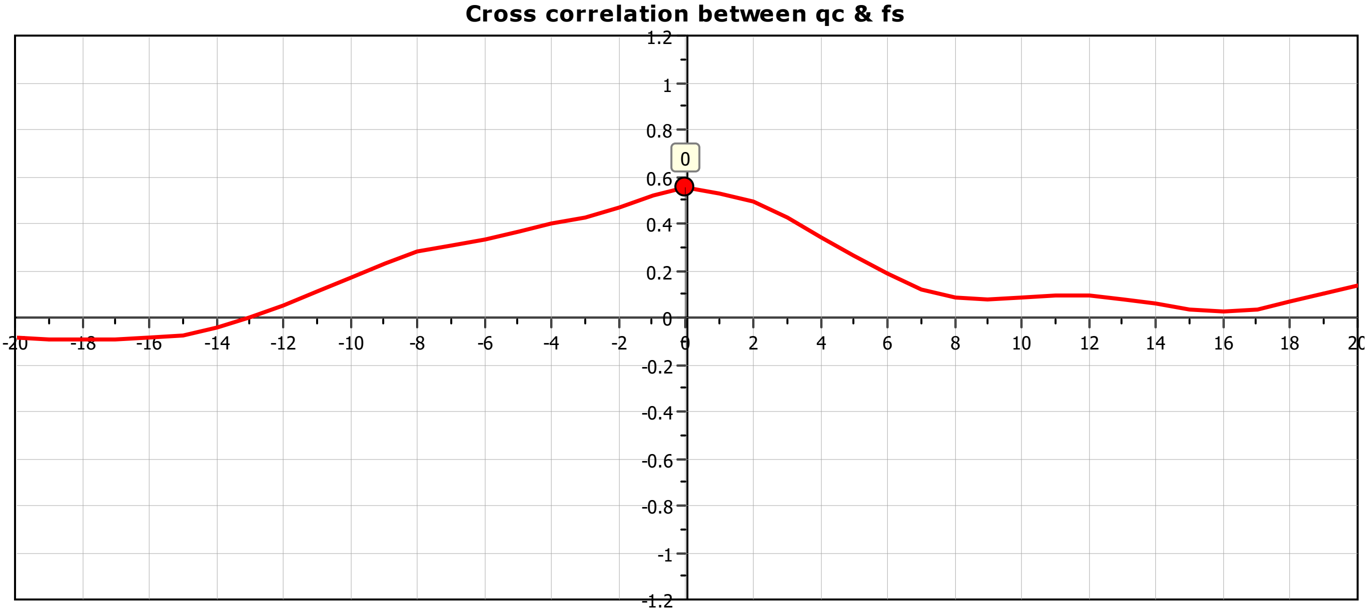
Location: **90 Minto Road, Watsonville, CA 95076**







The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



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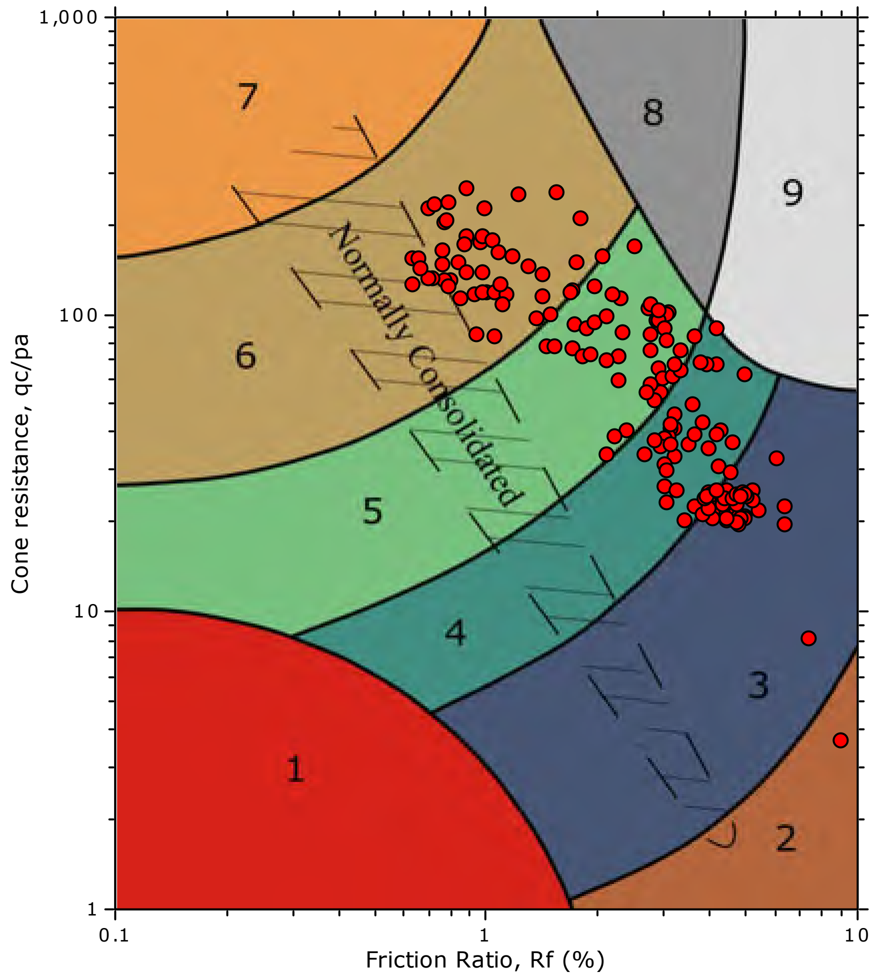
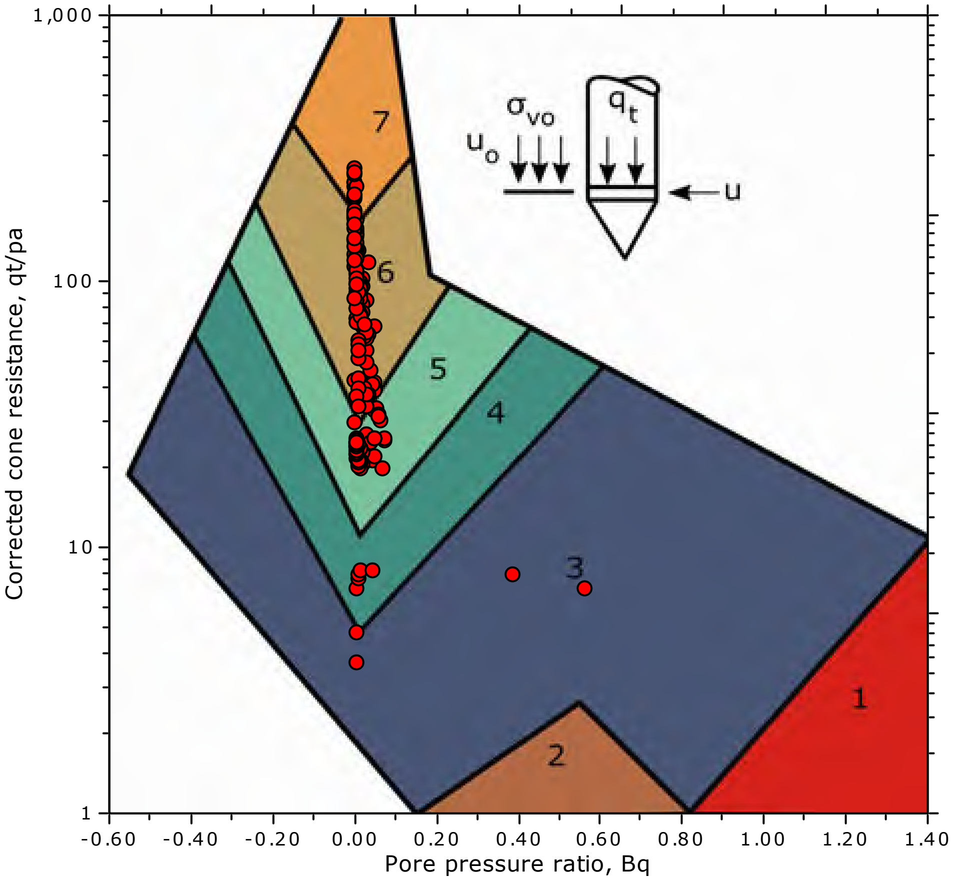
Project: **New Leaf Minto Road Watsonville**
Location: **90 Minto Road, Watsonville, CA 95076**

CPT: CPT-02
Total depth: 30.68 ft, Date: 11/1/2024
Cone Type: 15cm²
Cone Operator: JM-IY

SBT - Bq plots

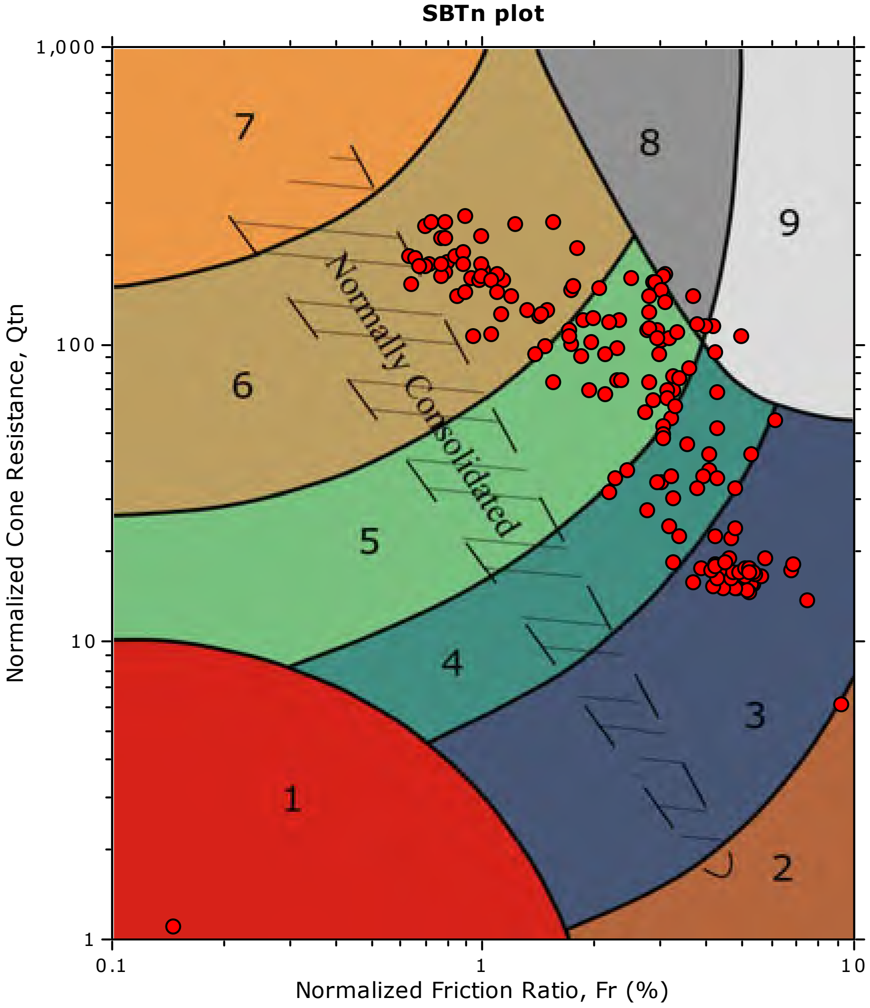

SBT plot

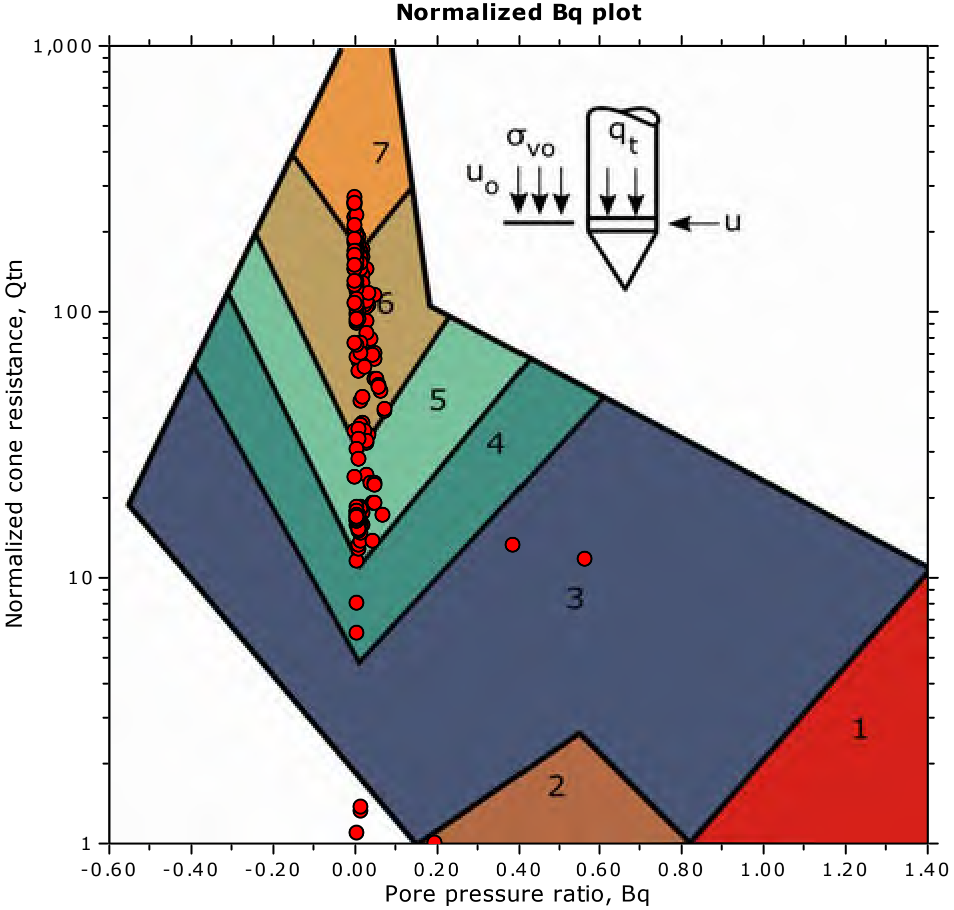

Bq plot

SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravelly sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

SBT - Bq plots (normalized)





SBTn legend

🟥 1. Sensitive fine grained
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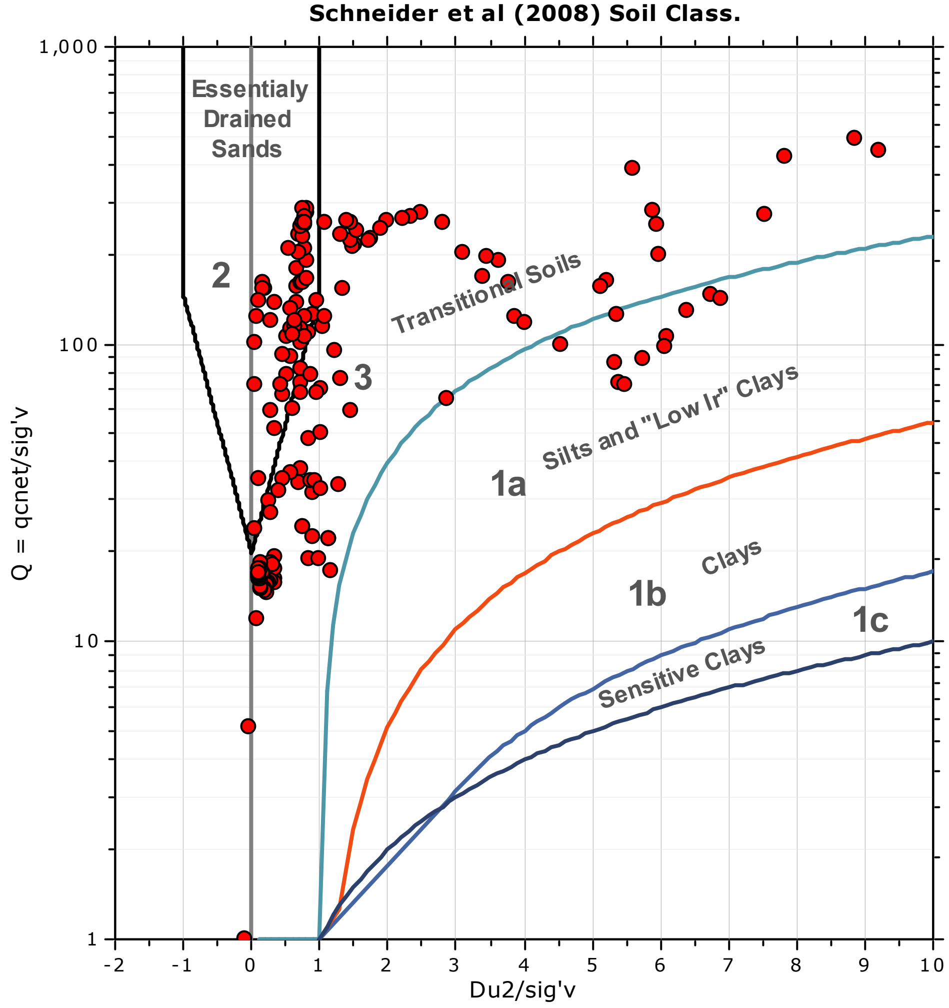
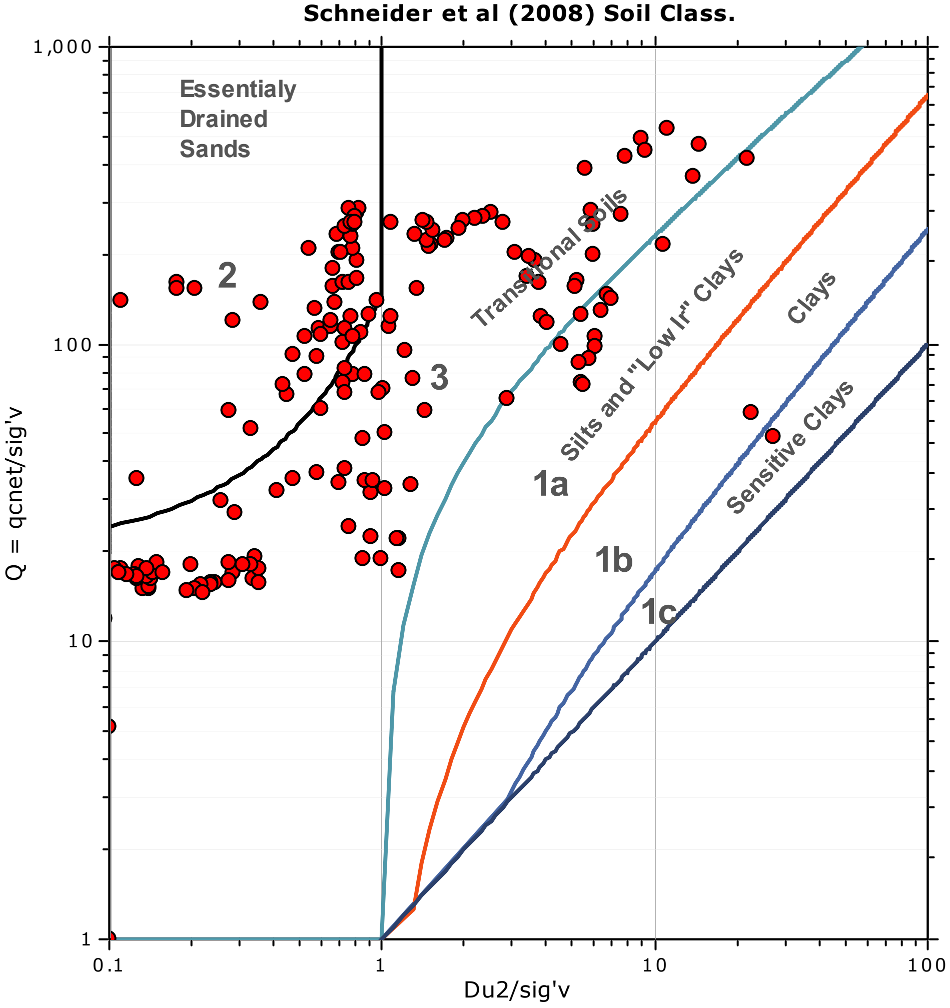
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟫 6. Clean sand to silty sand
🟧 7. Gravelly sand to sand
⬜ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

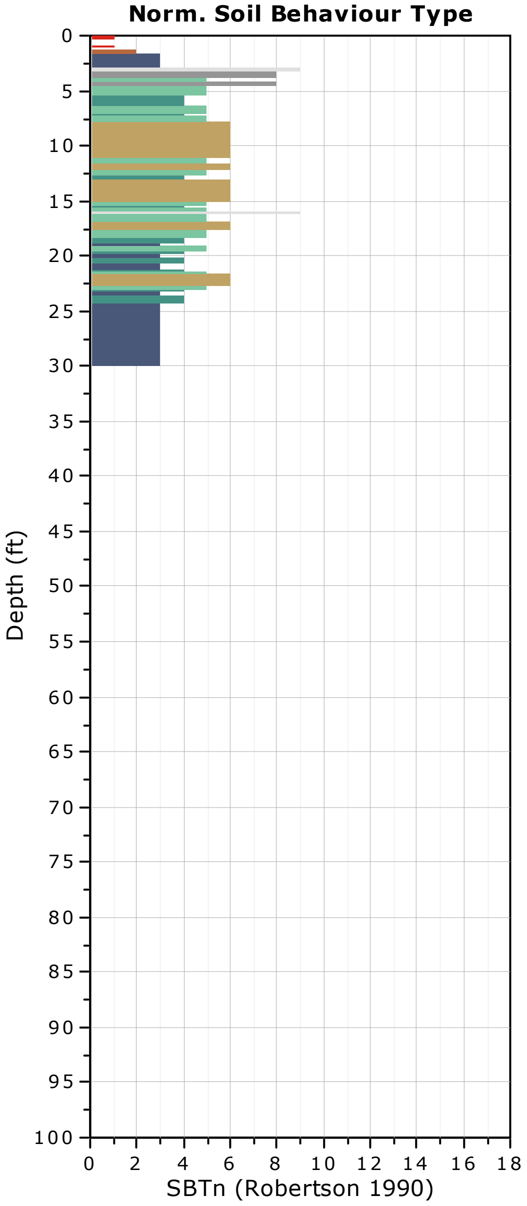
Bq plots (Schneider)

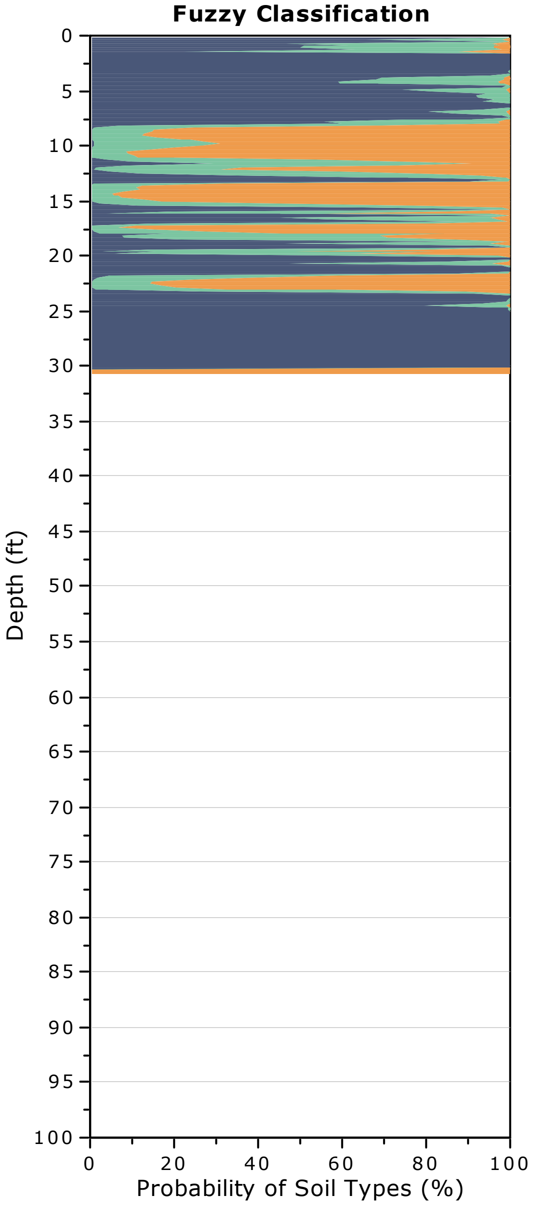




CPT: CPT-02
Total depth: 30.68 ft, Date: 11/1/2024
Cone Type: 15cm²
Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**
Location: **90 Minto Road, Watsonville, CA 95076**





Fuzzy classifiaction legend
- ■ Highly probable clayey soil
- ■ Highly probable mixture soil
- ■ Highly probable sandy soil

Haley & Aldrich Inc.

https://www.haleyaldrich.com/

CPT: CPT-02

Total depth: 30.68 ft, Date: 11/1/2024

Cone Type: 15cm²

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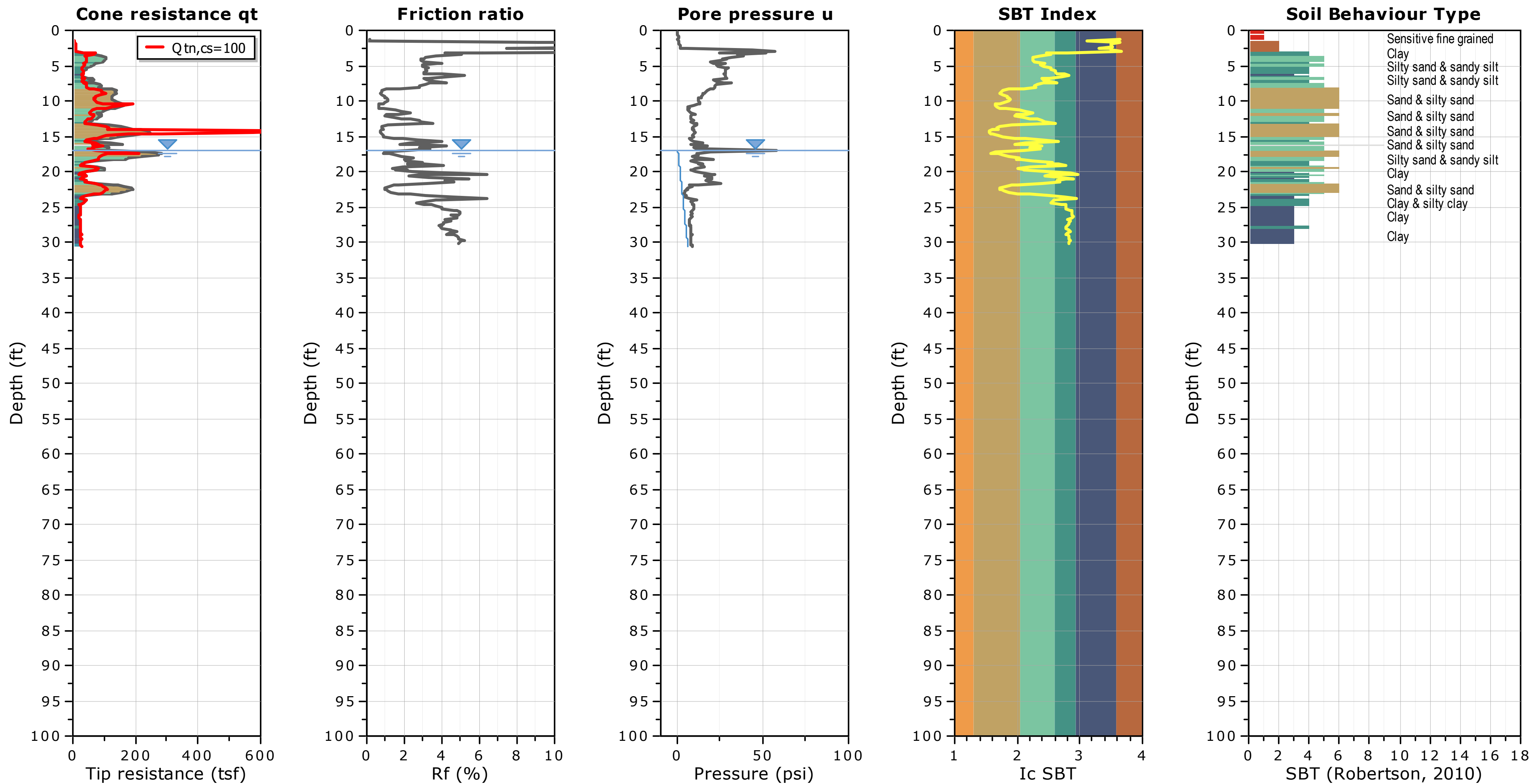

SBT legend

- 🟥 1. Sensitive fine grained
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- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬜ 8. Very stiff sand to clayey sand
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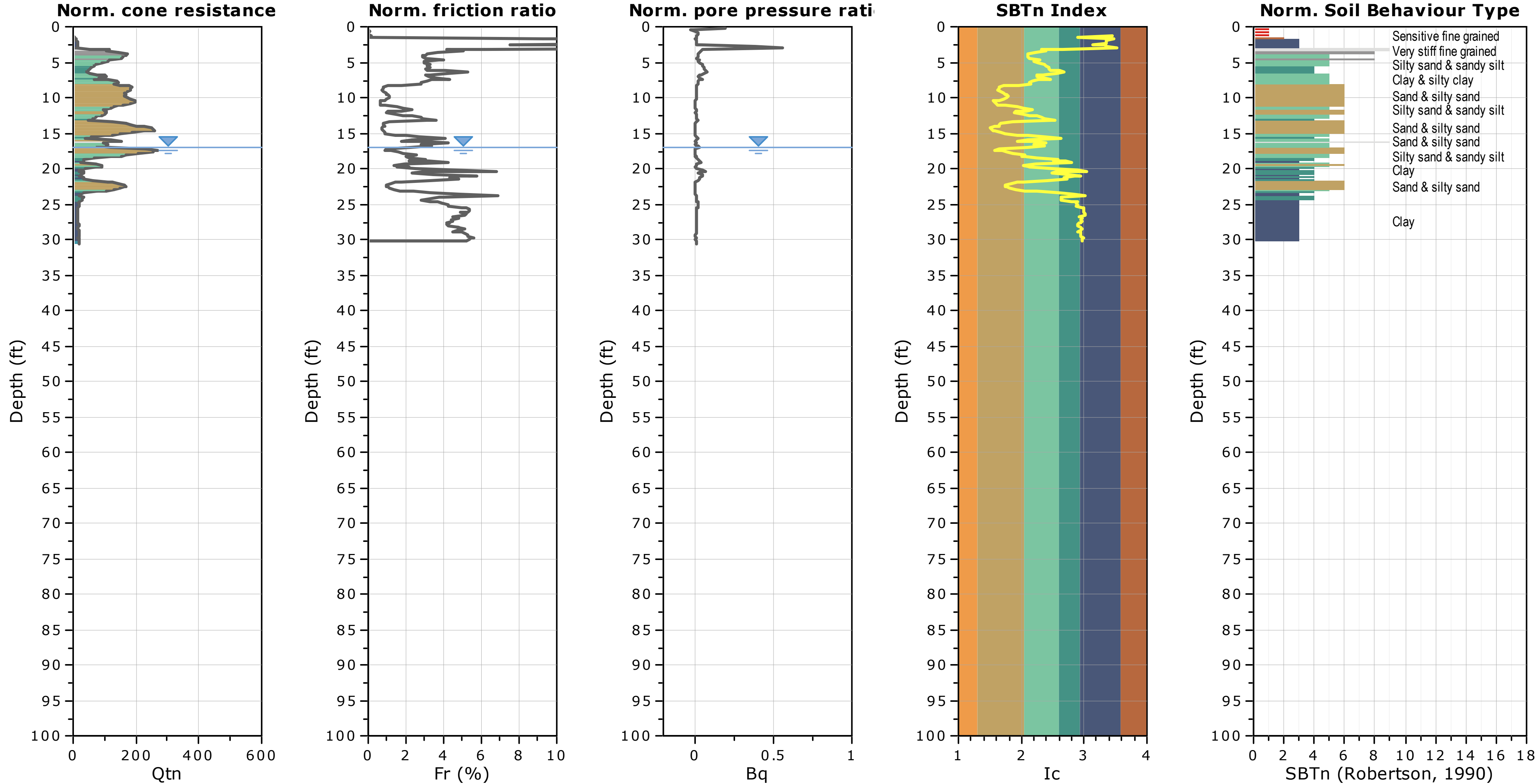

SBTn legend

🟥 1. Sensitive fine grained

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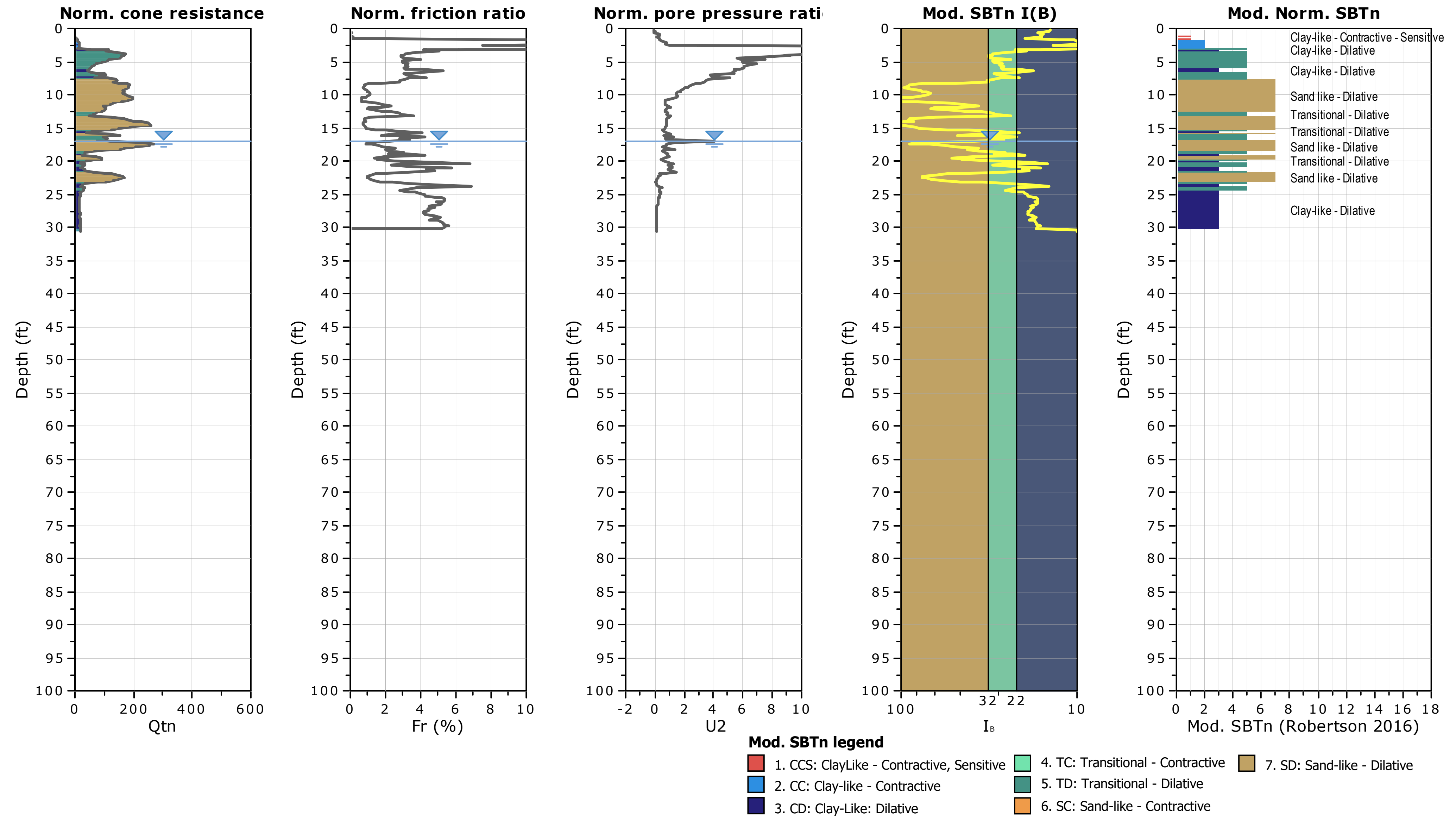
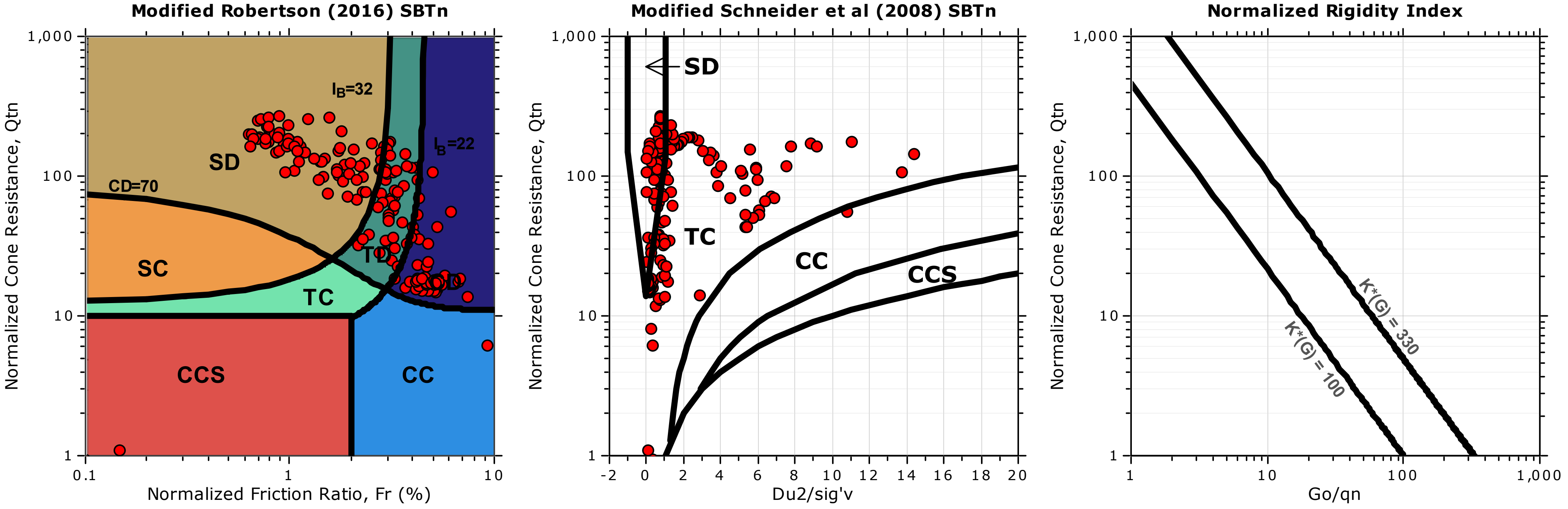


Updated SBTn plots



CCS: Clay-like - Contractive - Sensitive
CC: Clay-like - Contractive
CD: Clay-like - Dilative
TC: Transitional - Contractive
TD: Transitional - Dilative
SC: Sand-like - Contractive
SD: Sand-like - Dilative

K(G) > 330: Soils with significant microstructure (e.g. age/cementation)

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CPT: CPT-02

Total depth: 30.68 ft, Date: 11/1/2024

Cone Type: 15cm²

Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**

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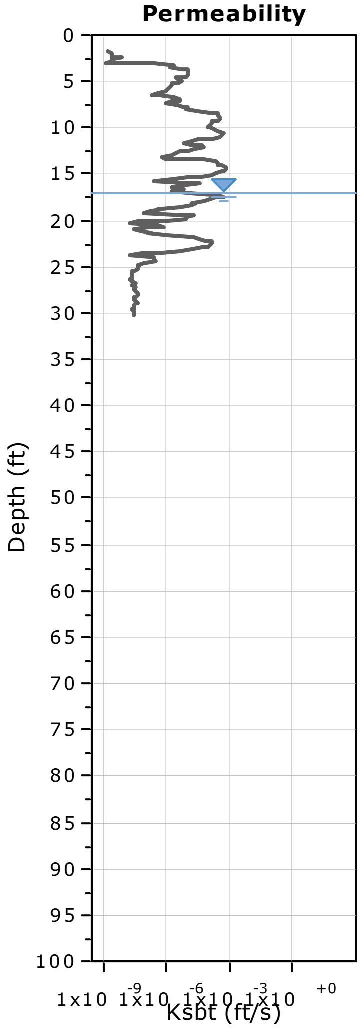
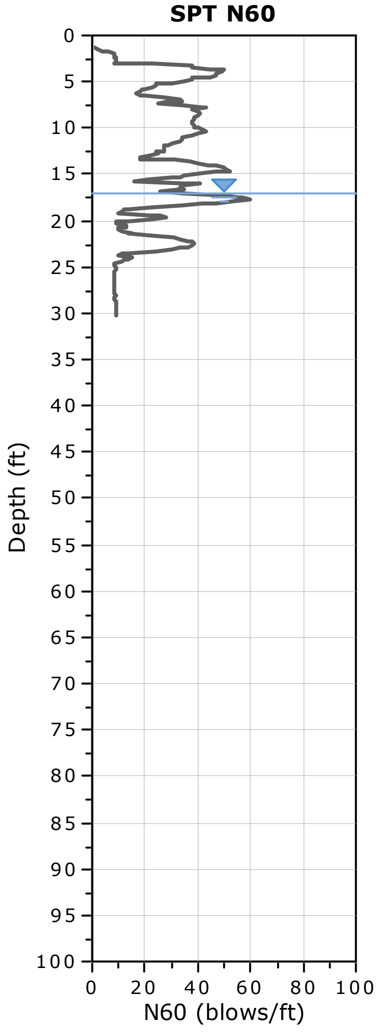
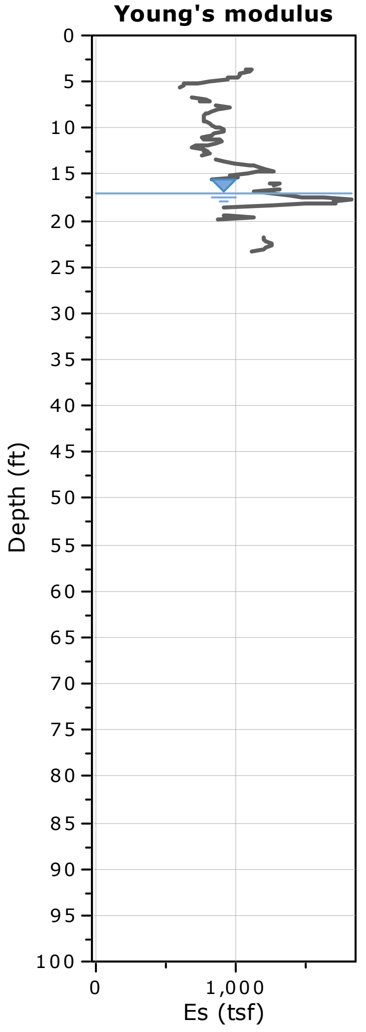


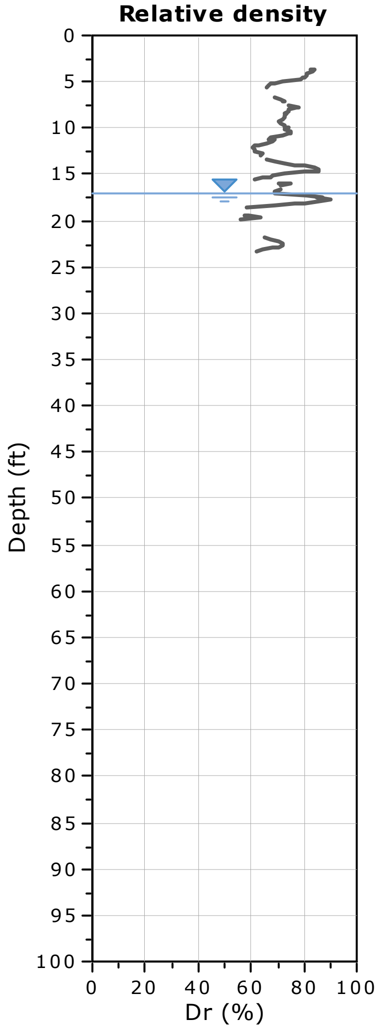
Permeability



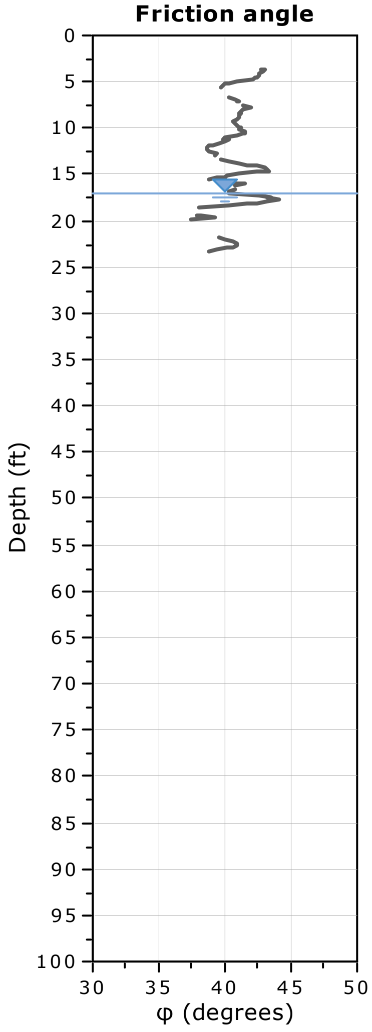
SPT N60



Young's modulus



Relative density



Friction angle

Calculation parameters

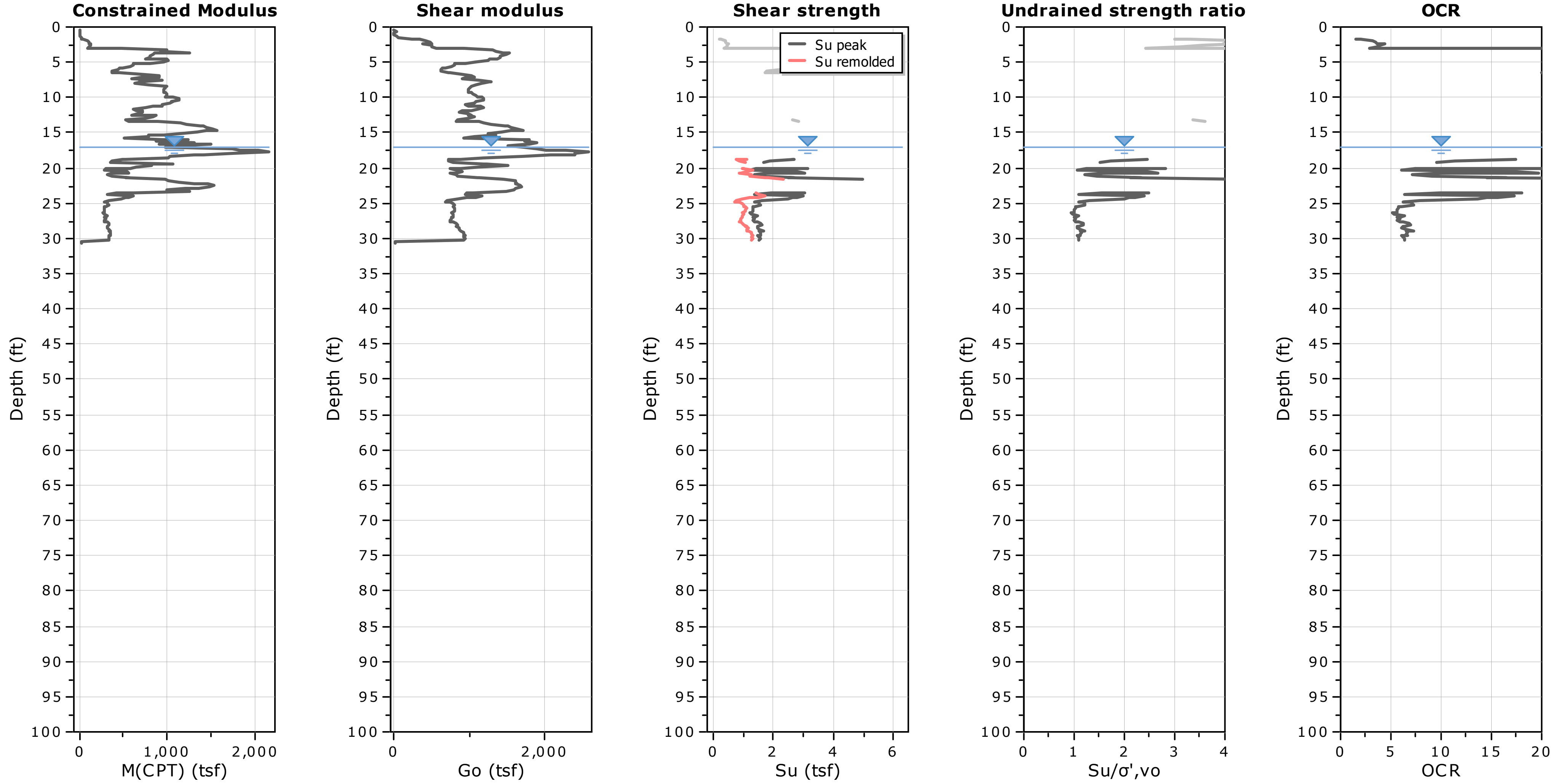
Permeability: Based on SBT_n

SPT N_{60}: Based on I_c and q_t

Young's modulus: Based on variable alpha using I_c (Robertson, 2009)

Relative density constant, C_{Dr}: 350.0

Phi: Based on Kulhawy & Mayne (1990)



Calculation parameters

Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable *alpha* using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: Auto

OCR factor for clays, N_{kt}: Auto
●— Flat Dilatometer Test data

Haley & Aldrich Inc.

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CPT: CPT-02

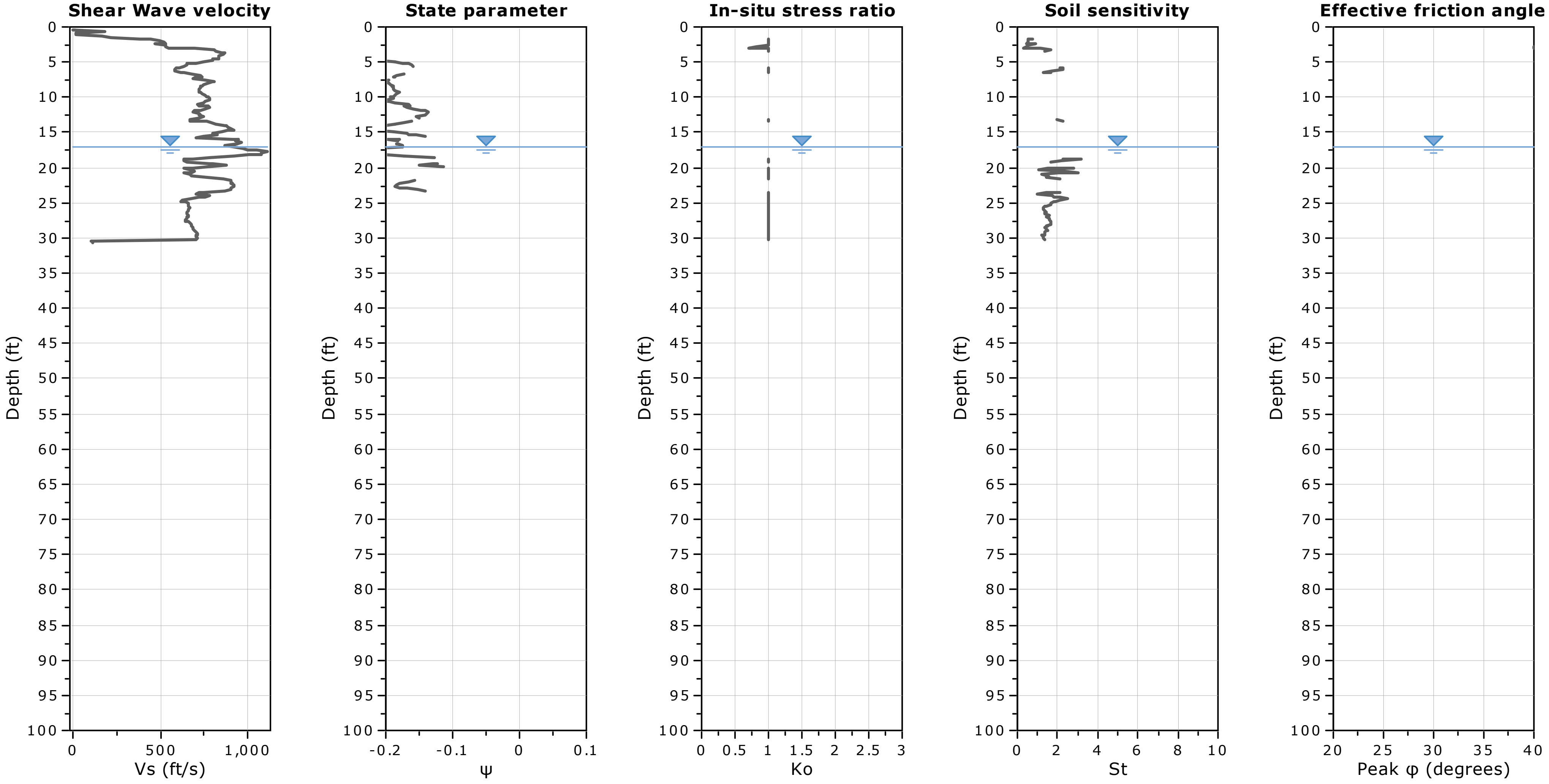
Total depth: 30.68 ft, Date: 11/1/2024

Cone Type: 15cm²

Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**

Location: **90 Minto Road, Watsonville, CA 95076**



Calculation parameters

Soil Sensitivity factor, N_S: 7.00

CPT: CPT-02

Total depth: 30.68 ft, Date: 11/1/2024

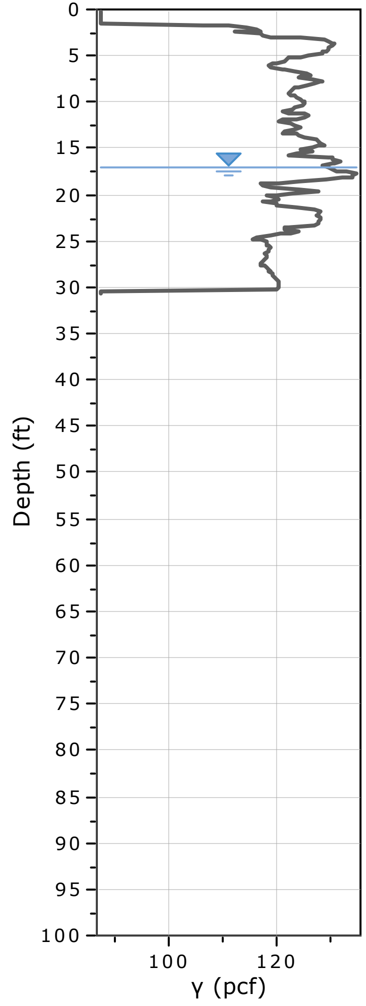
Cone Type: 15cm²

Cone Operator: JM-IY

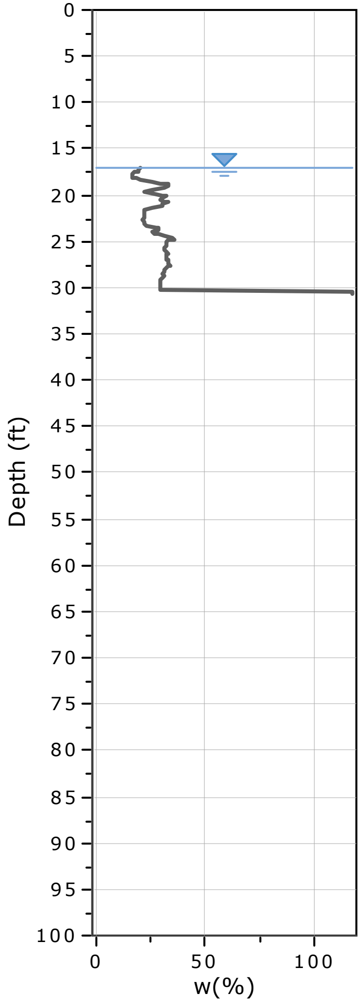
Project: New Leaf Minto Road Watsonville

Location: 90 Minto Road, Watsonville, CA 95076

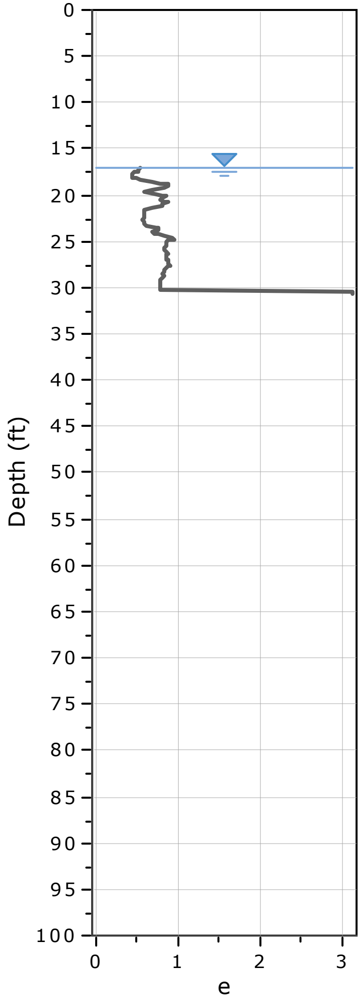
Bulk unit weight



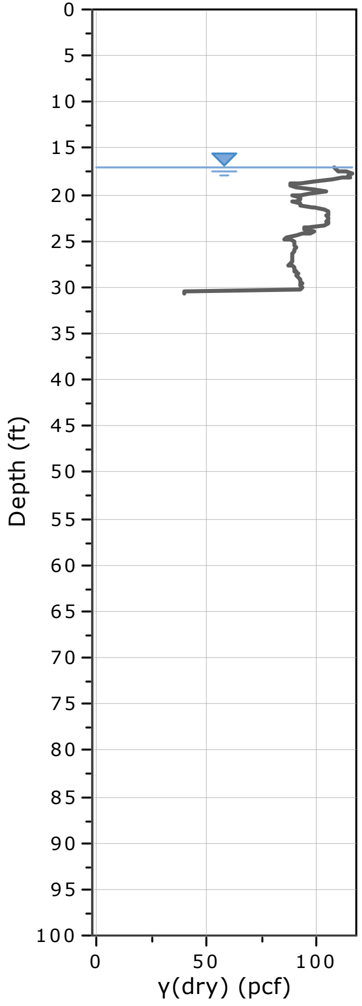
Water content



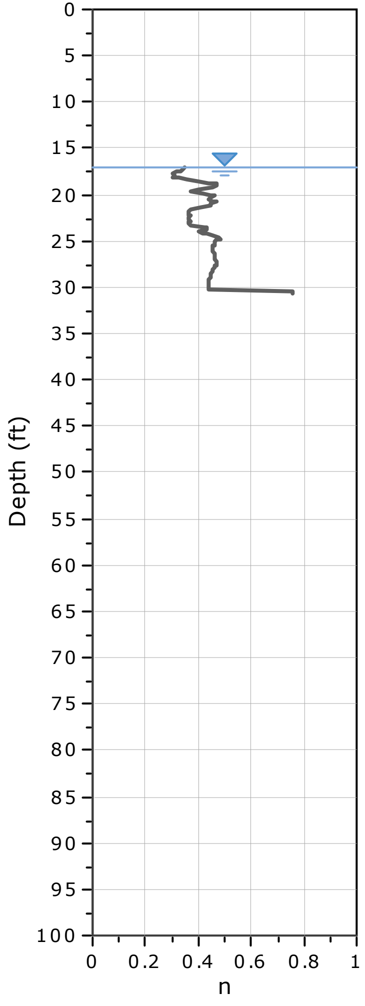
Void ratio



Dry unit weight



Porosity



Haley & Aldrich Inc.

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CPT: CPT-02

Total depth: 30.68 ft, Date: 11/1/2024

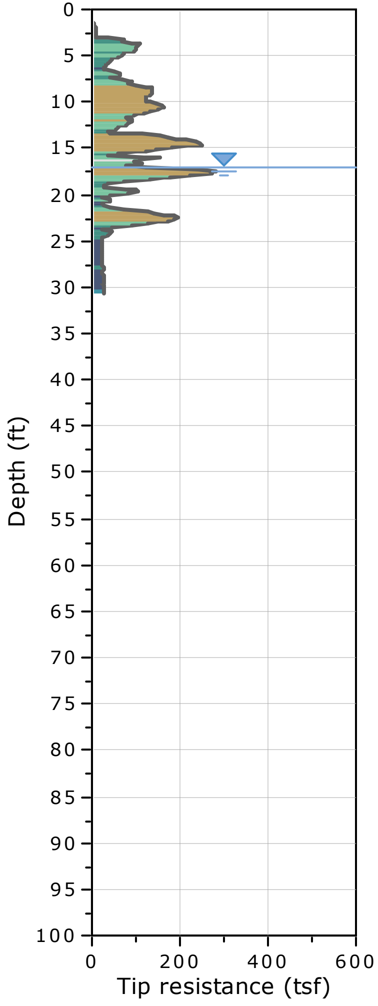
Cone Type: 15cm²

Cone Operator: JM-IY

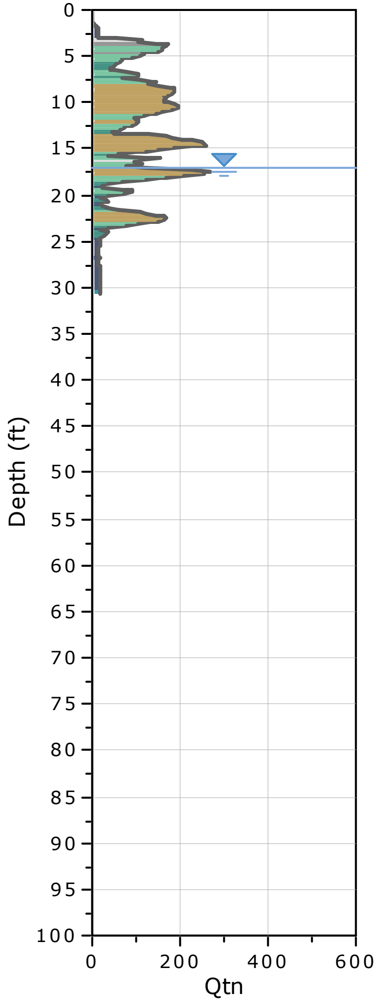
Project: **New Leaf Minto Road Watsonville**

Location: **90 Minto Road, Watsonville, CA 95076**

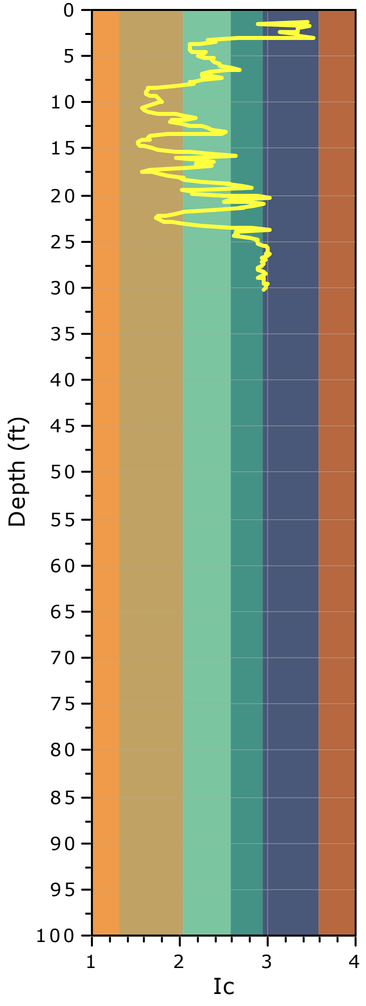
Cone resistance qt



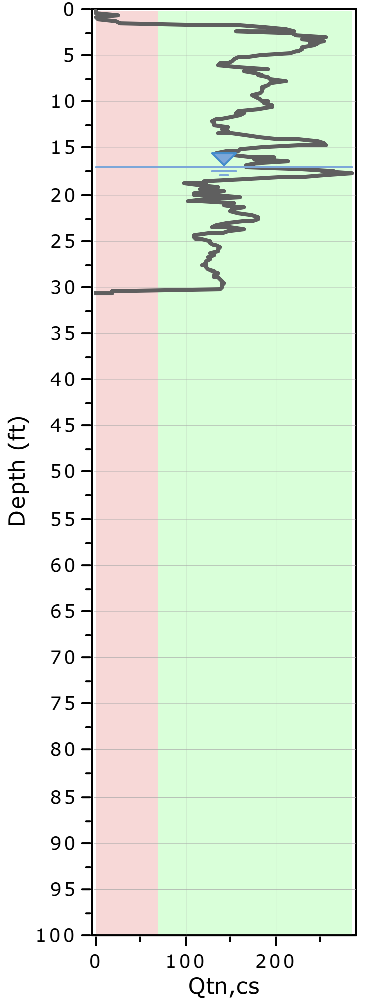
Norm. cone resistance



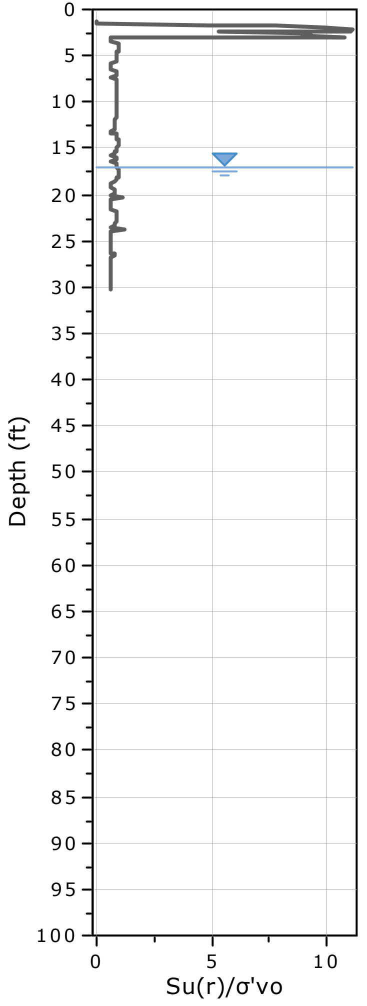
SBTn Index



Clean sand equivalent

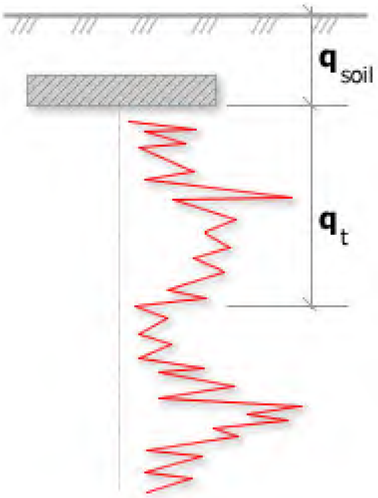


Residual strength ratio



CPT: CPT-02
Total depth: 30.68 ft, Date: 11/1/2024
Cone Type: 15cm²
Cone Operator: JM-IY

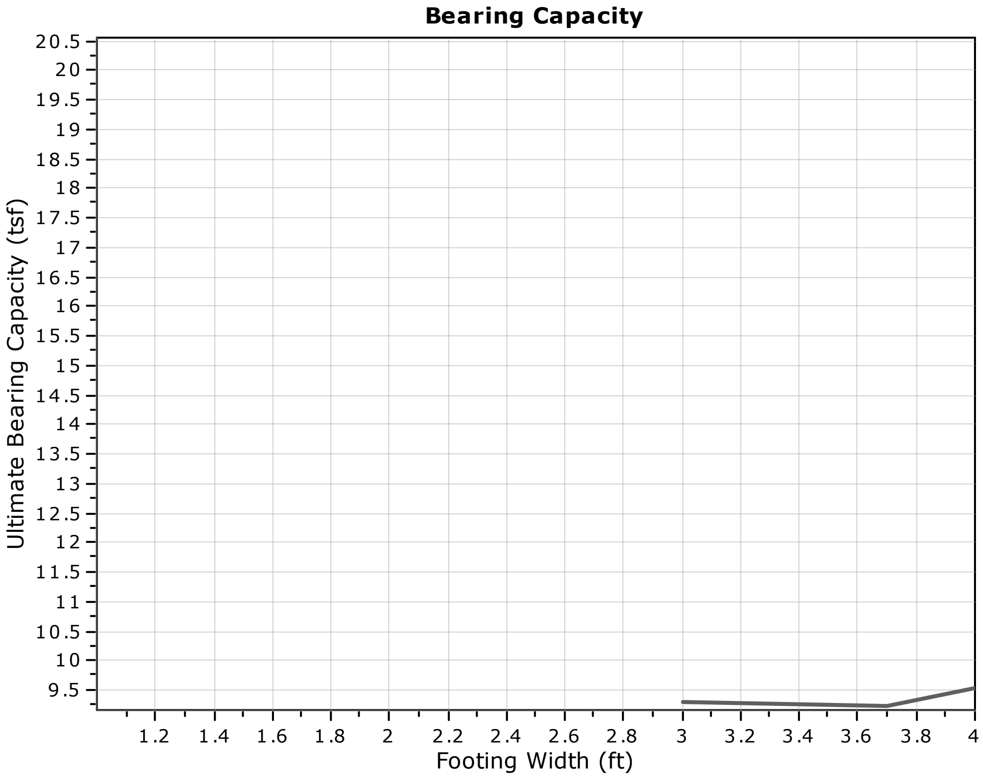
Project: **New Leaf Minto Road Watsonville**
Location: **90 Minto Road, Watsonville, CA 95076**



Bearing Capacity calculation is perfromed based on the formula:

$$Q_{ult} = R_k \times q_t + q_{soil}$$

where:
R_k: Bearing capacity factor
q_t: Average corrected cone resistance over calculation depth
q_{soil}: Pressure applied by soil above footing



:: Tabular results ::

No	B (ft)	Start Depth (ft)	End Depth (ft)	Ave. q_t (tsf)	R_k	Soil Press. (tsf)	Ult. bearing cap. (tsf)
1	3.00	1.60	6.10	46.01	0.20	0.10	9.30
2	3.70	1.60	7.15	45.70	0.20	0.10	9.24
3	4.40	1.60	8.20	49.23	0.20	0.10	9.94
4	5.10	1.60	9.25	60.89	0.20	0.10	12.27
5	5.80	1.60	10.30	68.44	0.20	0.10	13.78
6	6.50	1.60	11.35	76.79	0.20	0.10	15.45
7	7.20	1.60	12.40	77.58	0.20	0.10	15.61
8	7.90	1.60	13.45	75.76	0.20	0.10	15.25
9	8.60	1.60	14.50	85.75	0.20	0.10	17.25
10	9.30	1.60	15.55	91.57	0.20	0.10	18.41
11	10.00	1.60	16.60	92.11	0.20	0.10	18.52
12	10.70	1.60	17.65	97.63	0.20	0.10	19.62
13	11.40	1.60	18.70	101.98	0.20	0.10	20.49
14	12.10	1.60	19.75	99.26	0.20	0.10	19.95
15	12.80	1.60	20.80	96.32	0.20	0.10	19.36

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236\right)$$

where g_w = water unit weight

:: Permeability, k (m/s) ::

$$I_c < 3.27 \text{ and } I_c > 1.00 \text{ then } k = 10^{0.952-3.04 \cdot I_c}$$

$$I_c \leq 4.00 \text{ and } I_c > 3.27 \text{ then } k = 10^{-4.52-1.37 \cdot I_c}$$

:: N$_{SPT}$ (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT$_n$: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Drained Friction Angle, φ (°) ::

$$\varphi = \varphi'_{cv} + 15.94 \cdot \log(Q_{tn,cs}) - 26.88$$

(applicable only to SBT$_n$: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$
$\alpha = 14$ for $Q_{tn} > 14$
$\alpha = Q_{tn}$ for $Q_{tn} \leq 14$
$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$

If $I_c \geq 2.20$

$$M_{CPT} = 0.03 \cdot (q_t - \sigma_v) \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Small strain shear Modulus, Go (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Shear Wave Velocity, Vs (m/s) ::

$$V_s = \left(\frac{G_0}{\rho}\right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot +7 \cdot \log(F_r))}\right]^{1.25} \text{ or user defined}$$

$$OCR = k_{OCR} \cdot Q_{tn}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_O = (1 - \sin\varphi') \cdot OCR^{\sin\varphi'}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Soil Sensitivity, S$_t$ ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Peak Friction Angle, φ' (°) ::

$$\varphi' = 29.5^\circ \cdot B_q^{0.121} \cdot (0.256 + 0.336 \cdot B_q + \log Q_t)$$

(applicable for $0.10 < B_q < 1.00$)

References

- Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 5th Edition, November 2012
- Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)
- N Barounis, J Philpot, Estimation of in-situ water content, void ratio, dry unit weight and porosity using CPT for saturated sands, Proc. 20th NZGS Geotechnical Symposium

Haley & Aldrich Inc.

https://www.haleyaldrich.com/

CPT: CPT-02

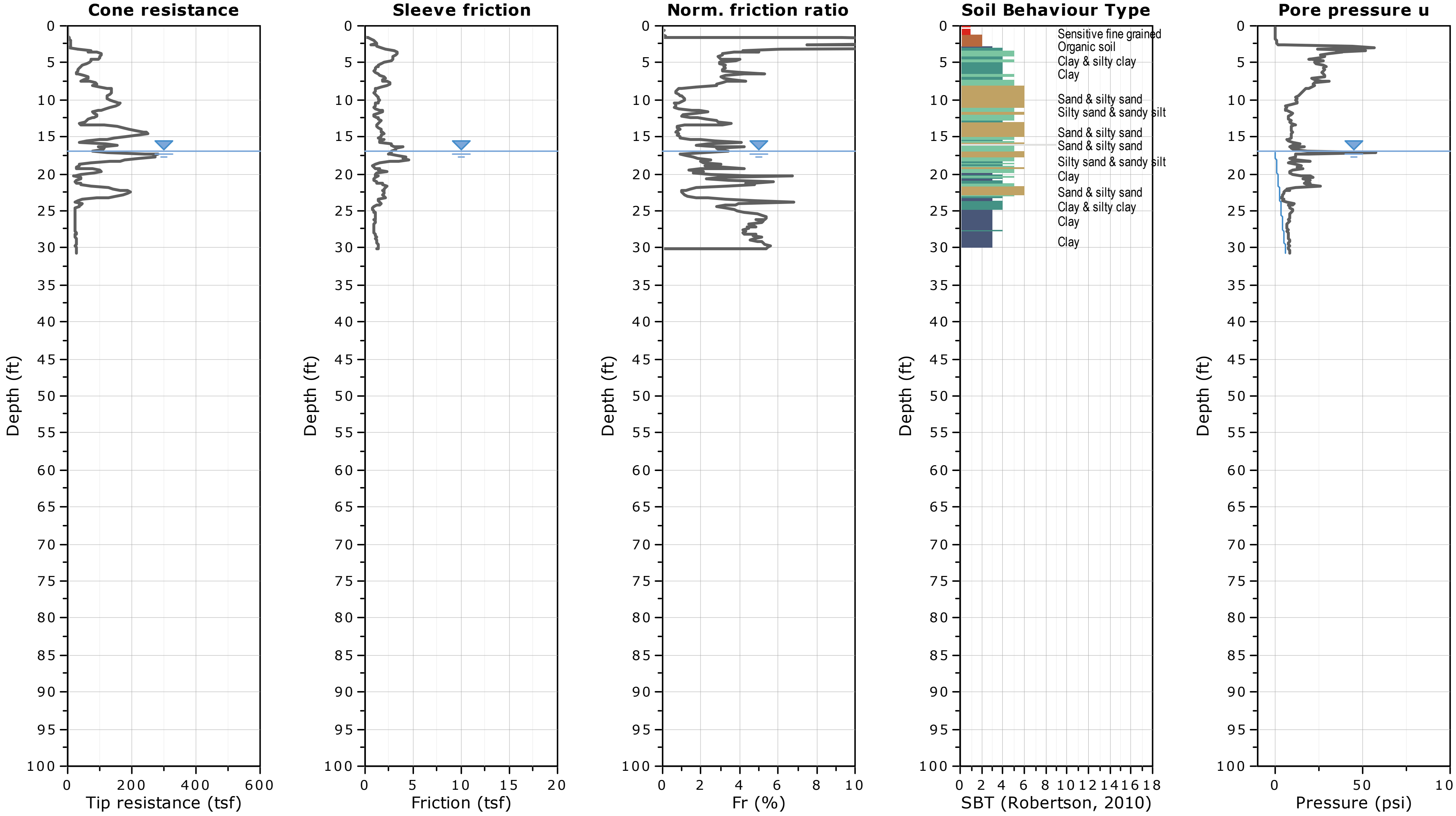
Total depth: 30.68 ft, Date: 11/1/2024

Cone Type: 15cm²

Cone Operator: JM-IY

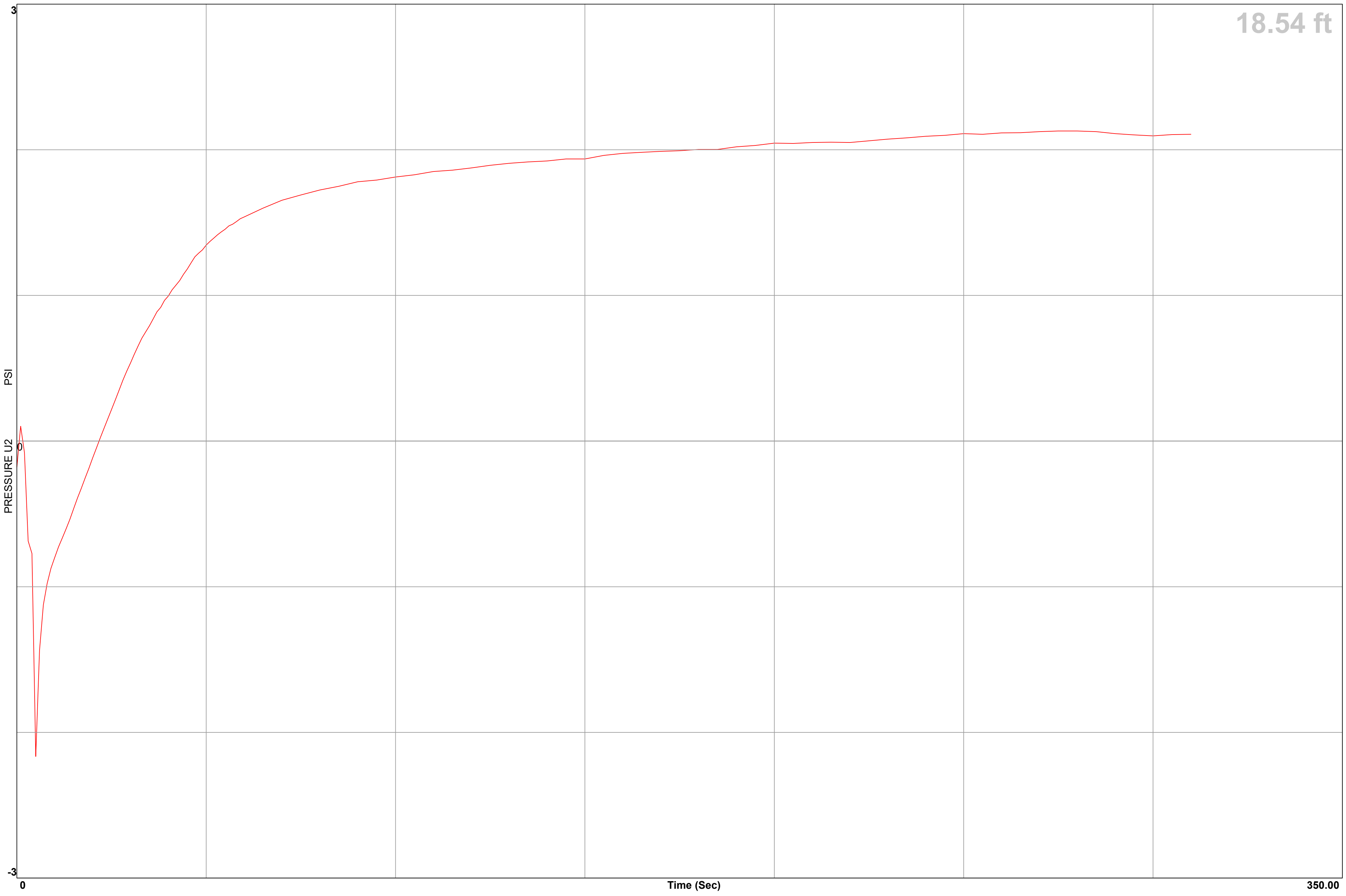
Project: New Leaf Minto Road Watsonville

Location: 90 Minto Road, Watsonville, CA 95076



Haley & Aldrich Inc.

Location	New Leaf Minto Road Watsonville	**Operator**	JM-IY		
Job Number	021059-003	**Cone Number**	DDG1589	**GPS**	
Hole Number	CPT-03	**Date and Time**	11/1/2024 11:53:37 AM		
Equilized Pressure	2.1	**EST GW Depth During Test**	13.6		



Haley & Aldrich Inc.

https://www.haleyaldrich.com/

CPT: CPT-03

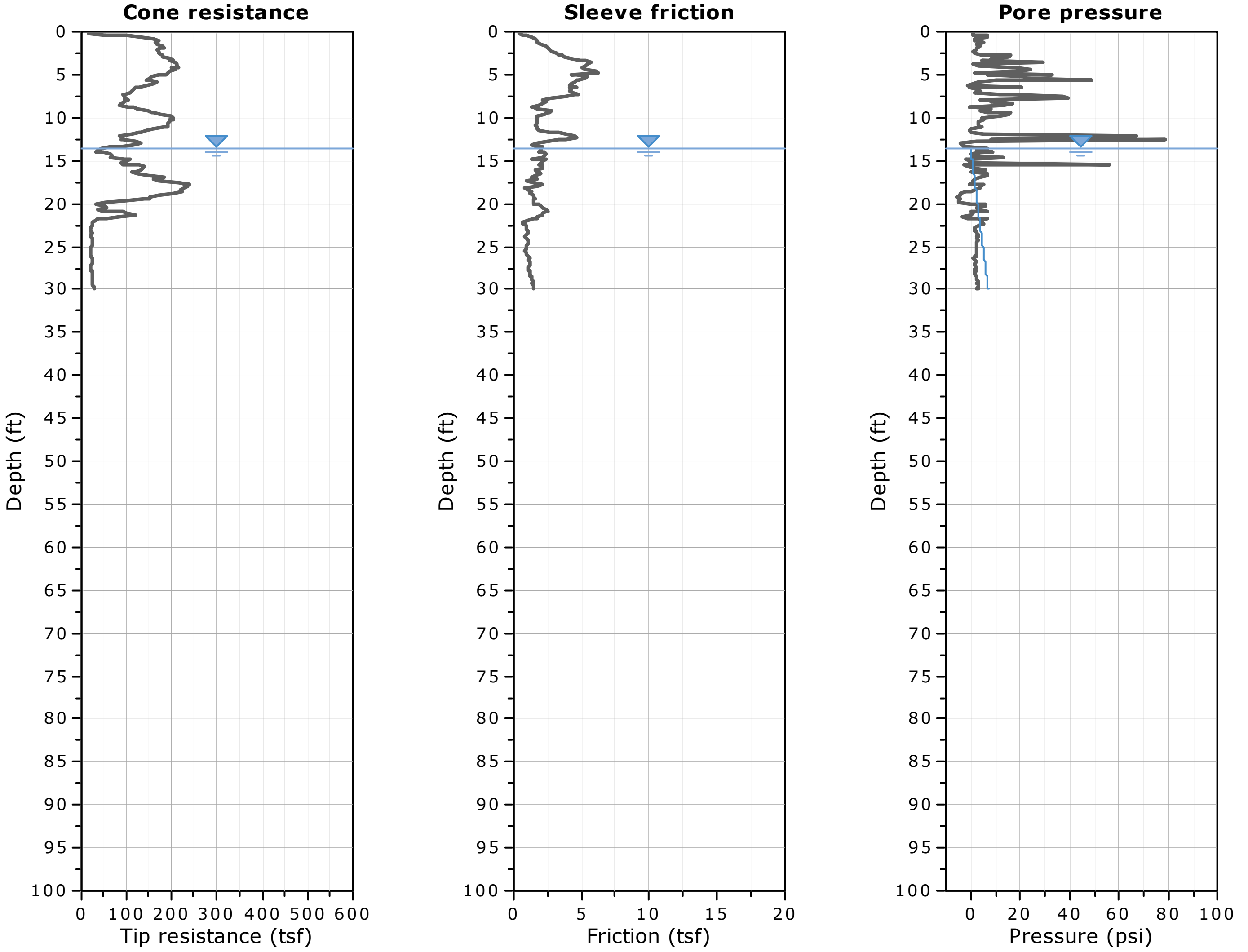
Total depth: 30.02 ft, Date: 11/1/2024

Cone Type: 15cm²

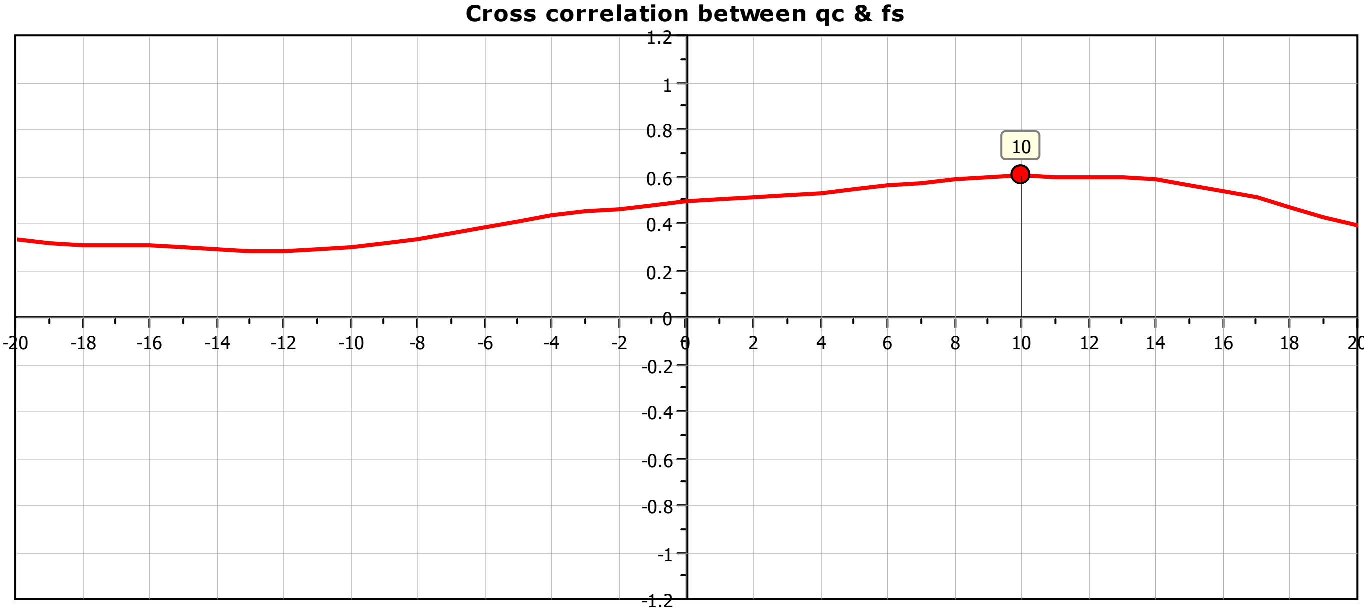
Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**

Location: **90 Minto Road, Watsonville, CA 95076**

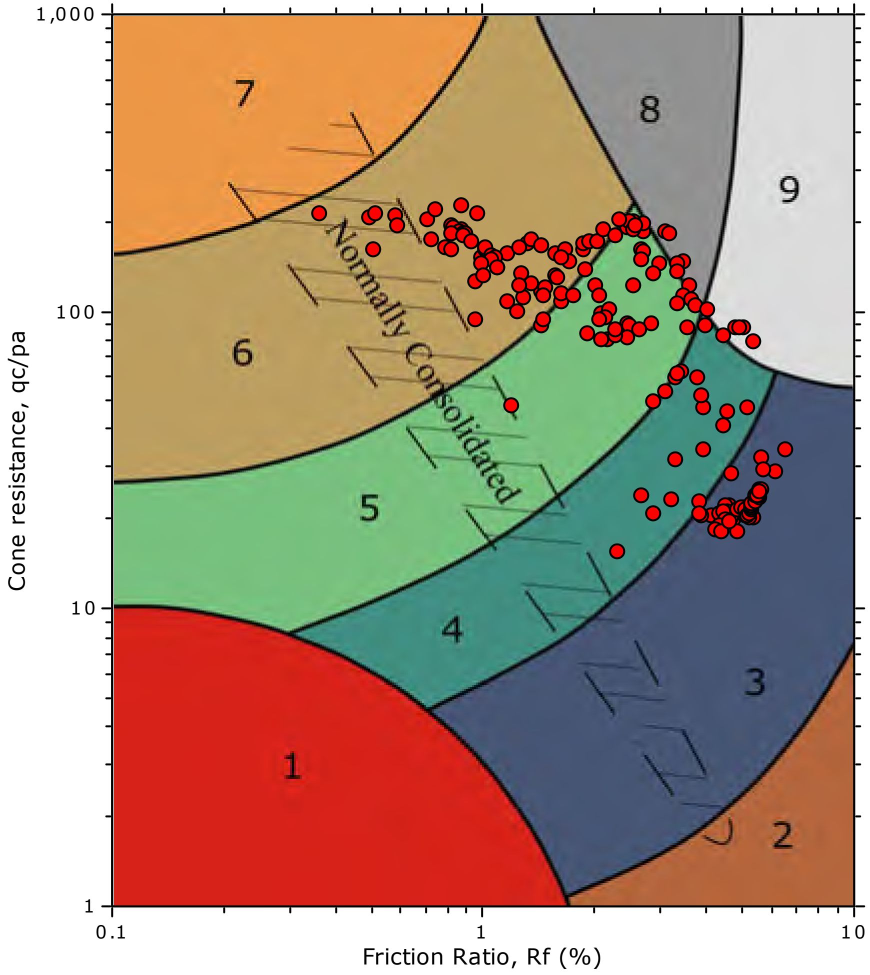


The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).

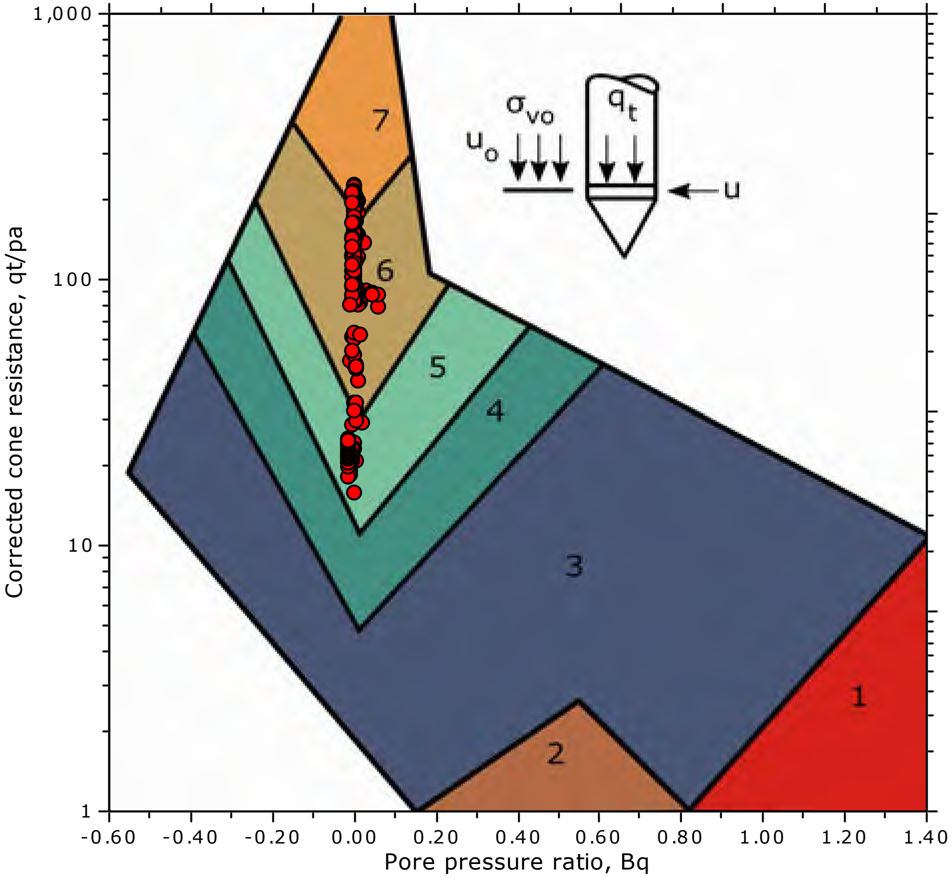


SBT - Bq plots

SBT plot



Bq plot



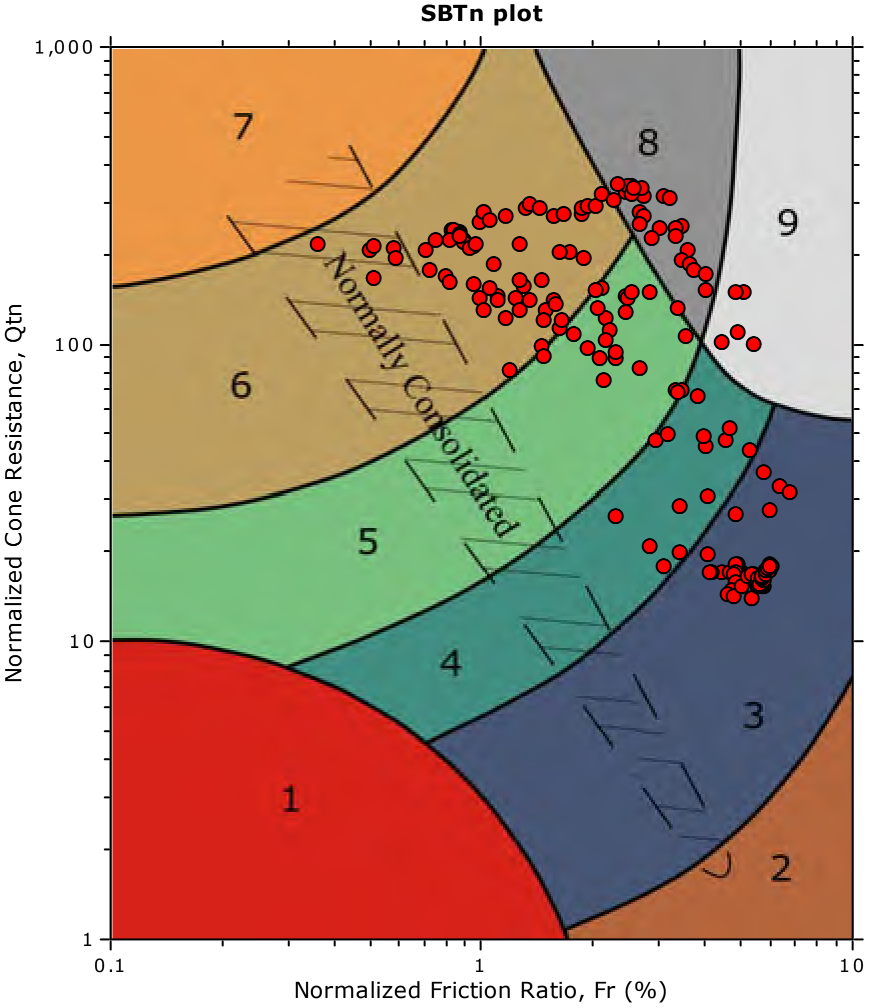
SBT legend

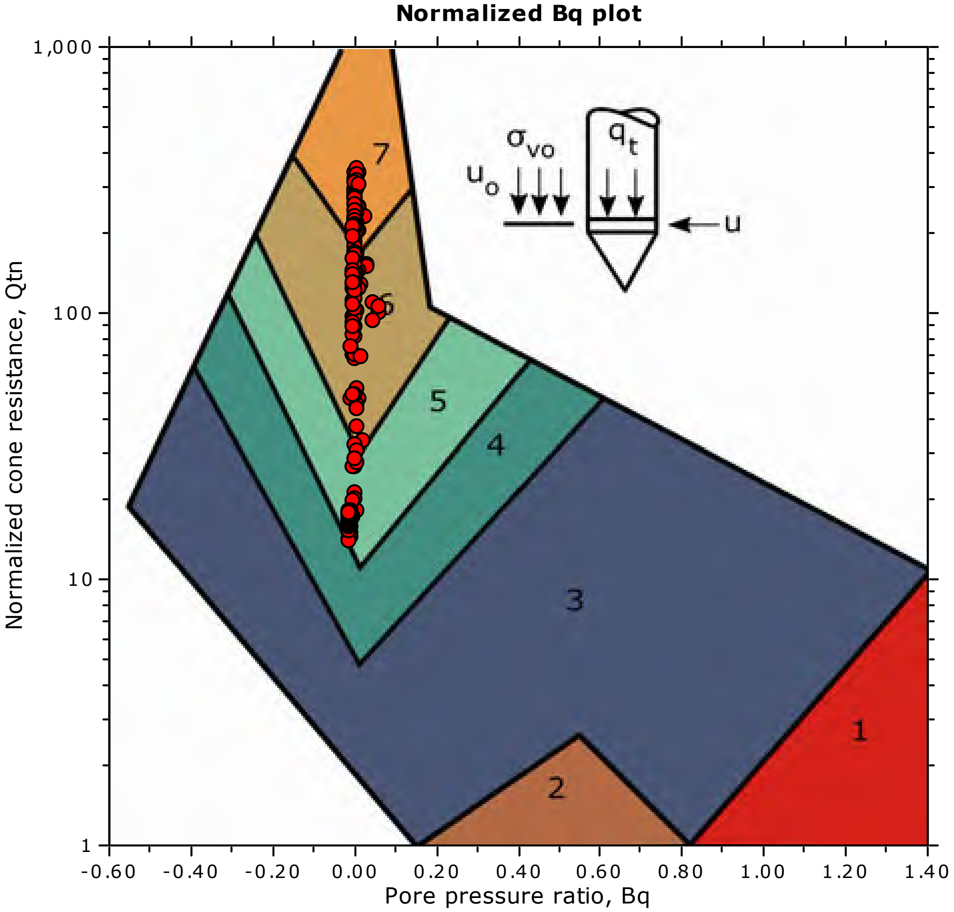
🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟫 6. Clean sand to silty sand
🟧 7. Gravelly sand to sand
⬛ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

Project: **New Leaf Minto Road Watsonville**

Location: **90 Minto Road, Watsonville, CA 95076**

SBT - Bq plots (normalized)

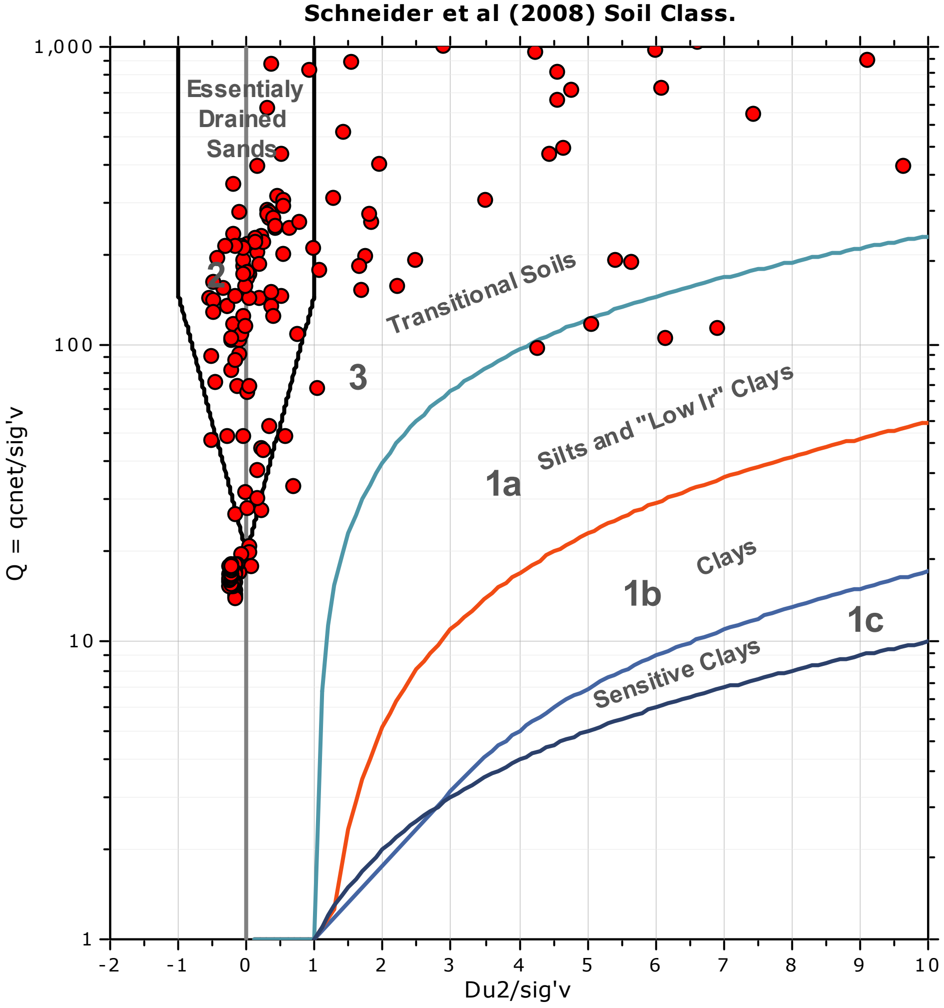


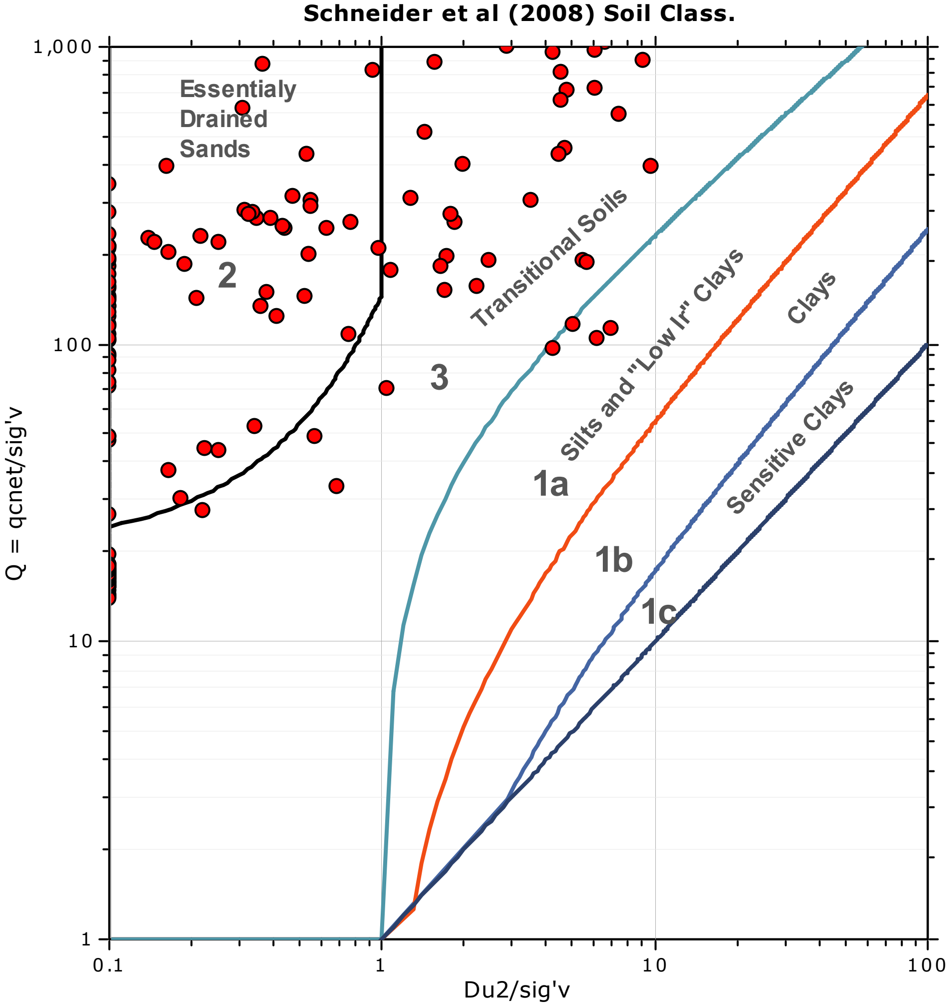


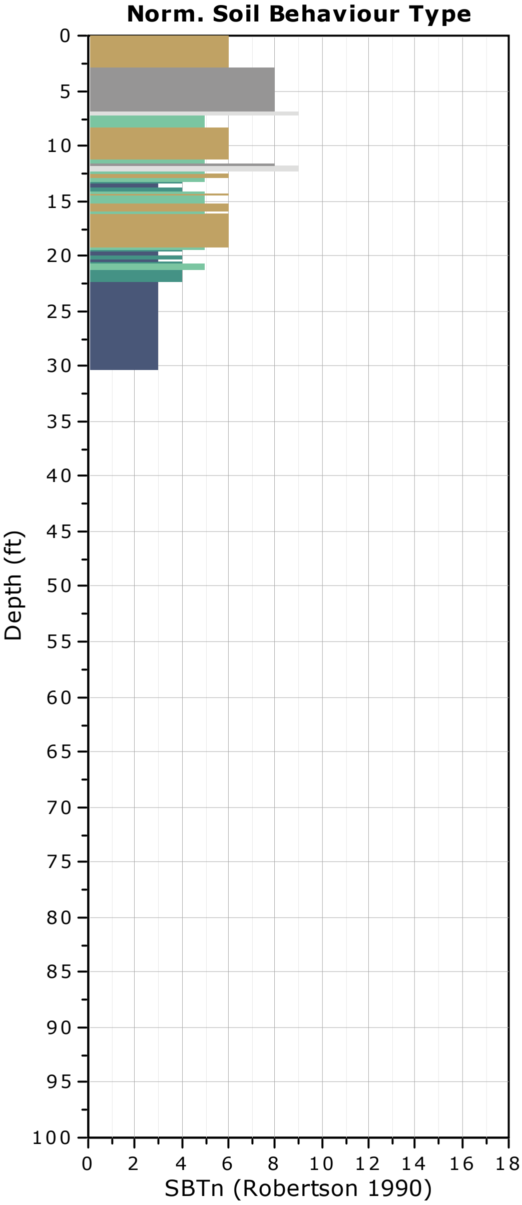
SBTn legend

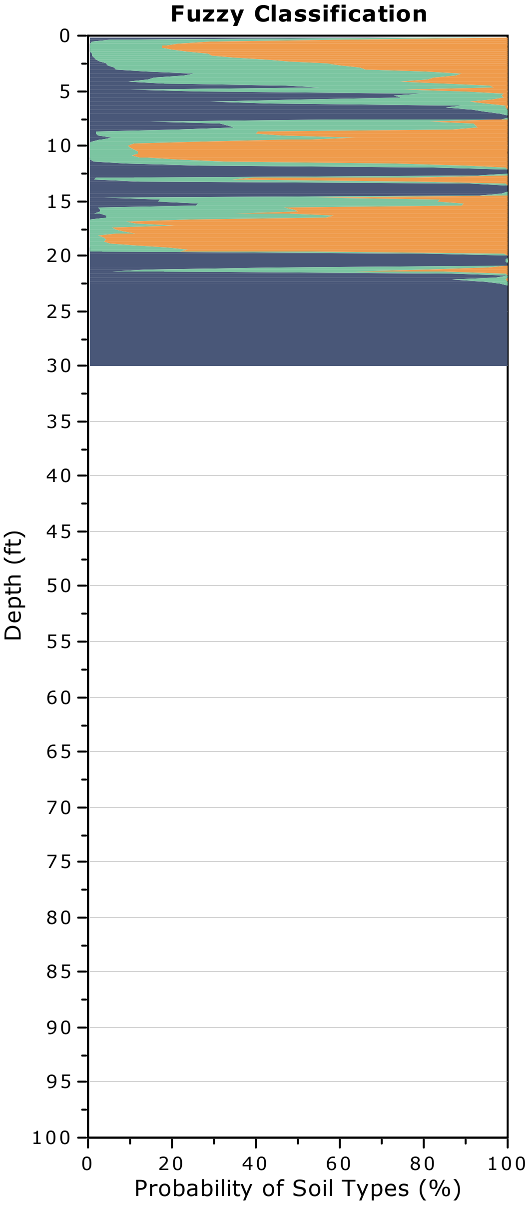
🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟫 6. Clean sand to silty sand
🟧 7. Gravelly sand to sand
⬛ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

Bq plots (Schneider)









Fuzzy classifiaction legend

- 🟦 Highly probable clayey soil
- 🟩 Highly probable mixture soil
- 🟧 Highly probable sandy soil

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CPT: CPT-03

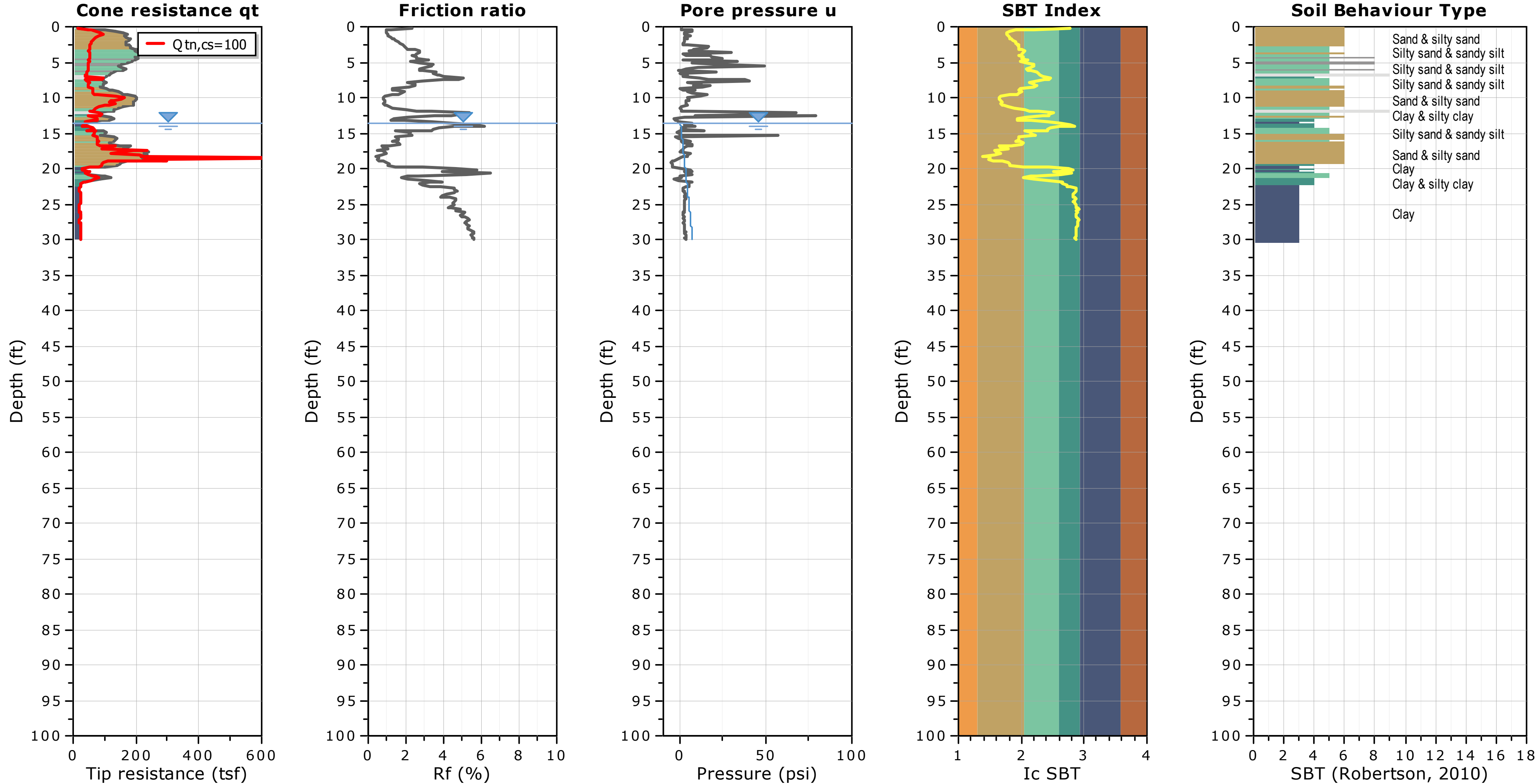
Total depth: 30.02 ft, Date: 11/1/2024

Cone Type: 15cm²

Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**

Location: **90 Minto Road, Watsonville, CA 95076**



SBT legend

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

CPT: CPT-03

Total depth: 30.02 ft, Date: 11/1/2024

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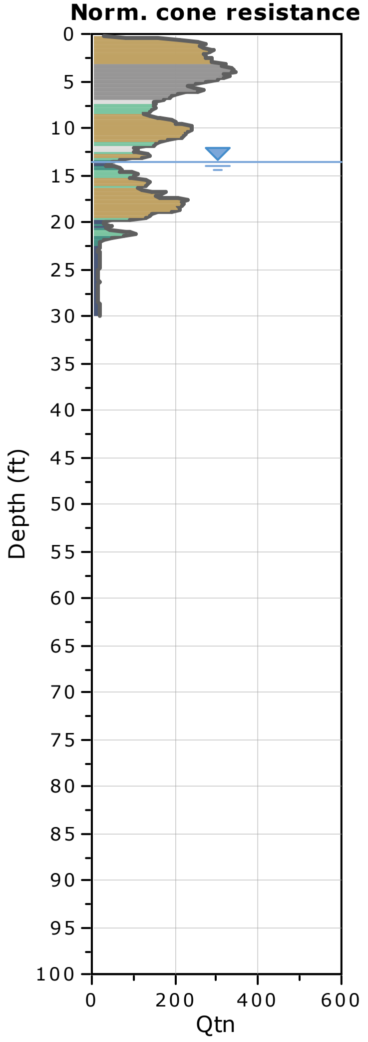
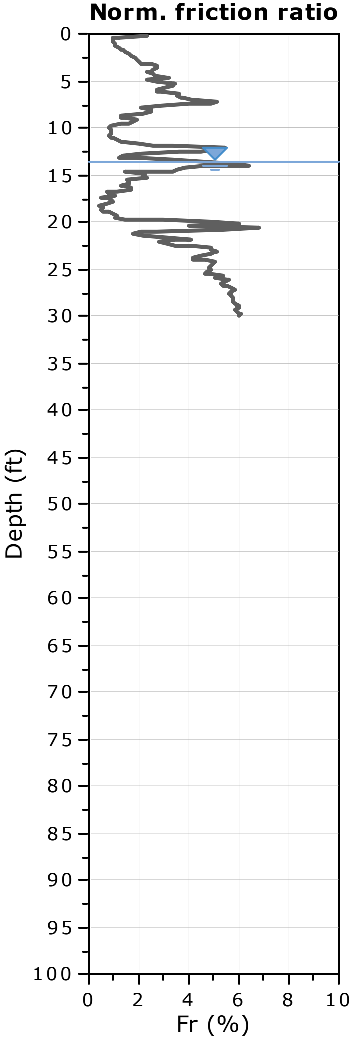
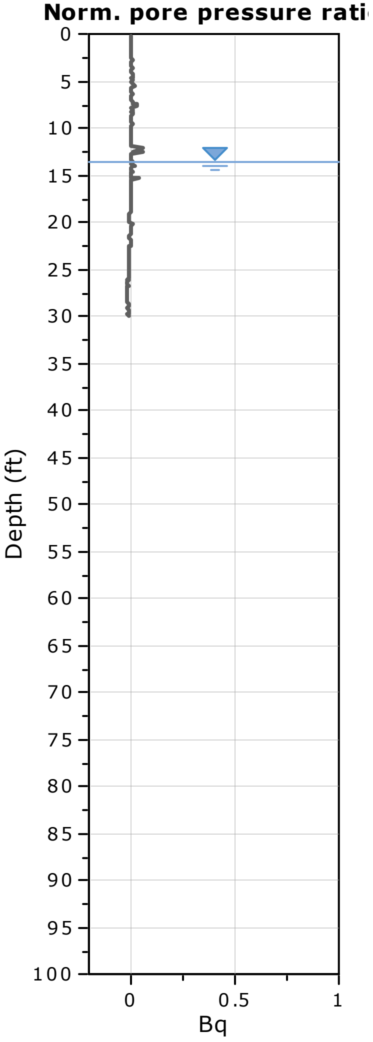
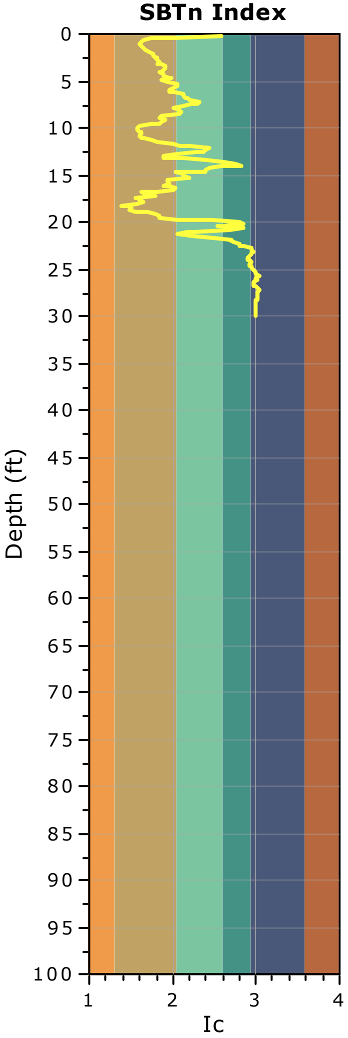
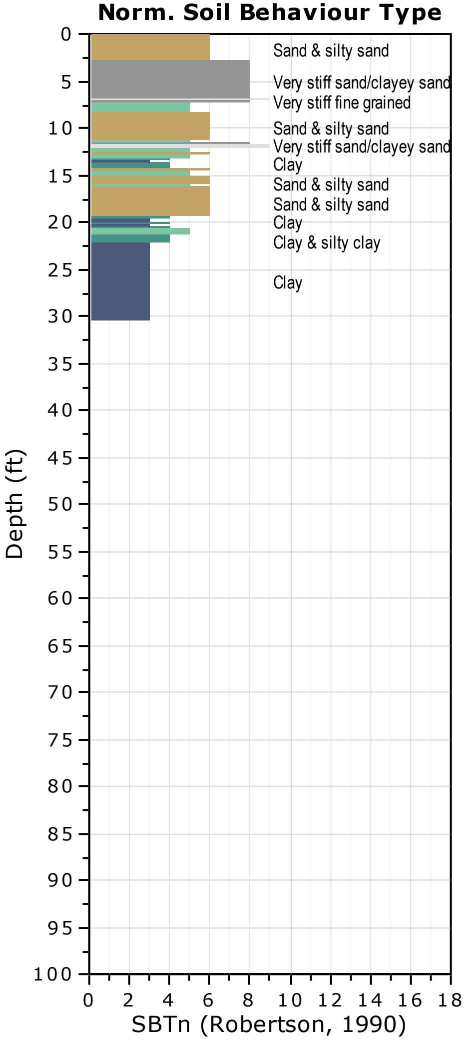

Norm. cone resistance


Norm. friction ratio


Norm. pore pressure rati


SBTn Index


Norm. Soil Behaviour Type

SBTn legend

🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟨 6. Clean sand to silty sand
🟧 7. Gravely sand to sand
⬛ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

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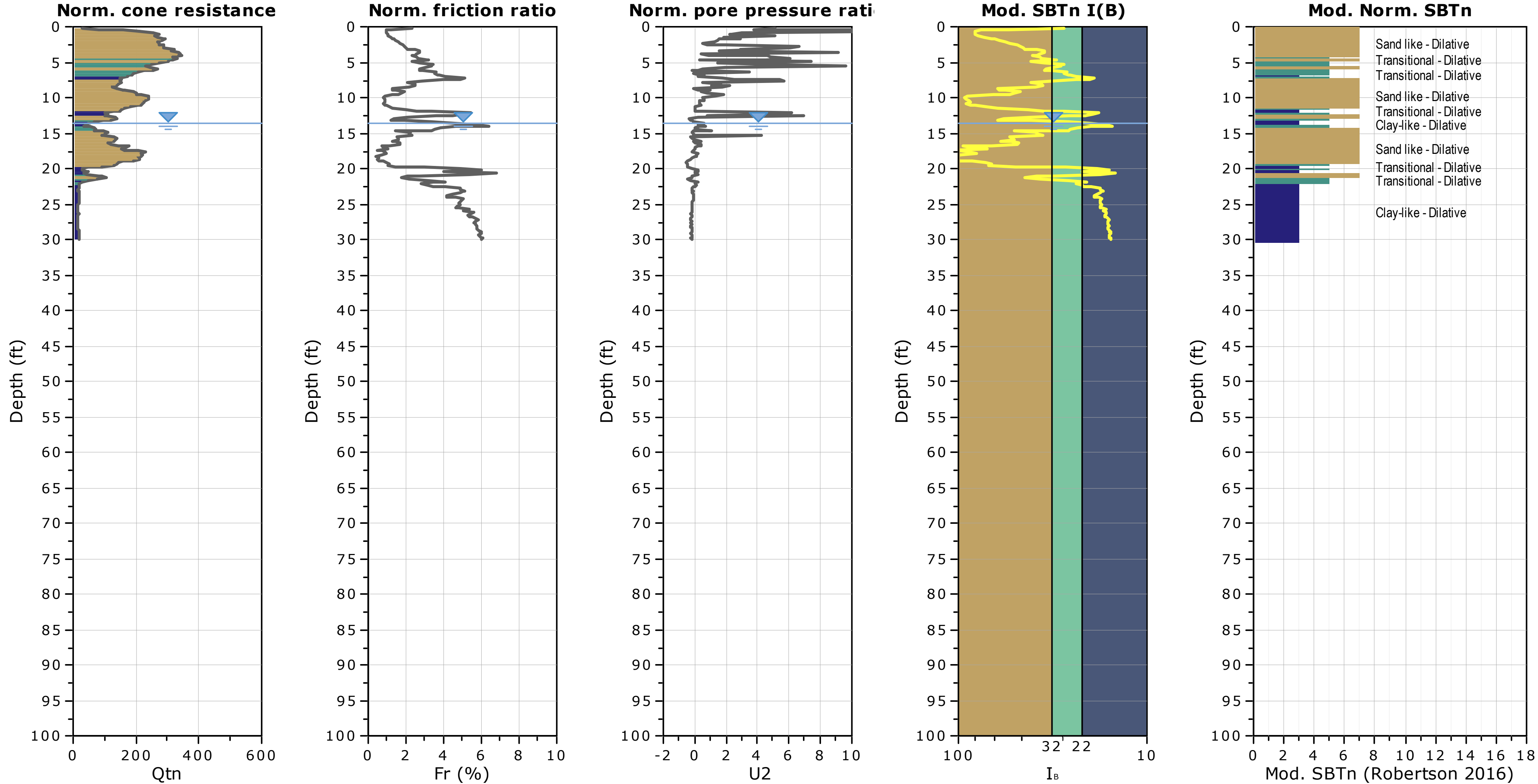


Mod. SBTn legend

🟥 1. CCS: ClayLike - Contractive, Sensitive

🟦 2. CC: Clay-like - Contractive

■ 3. CD: Clay-Like: Dilative

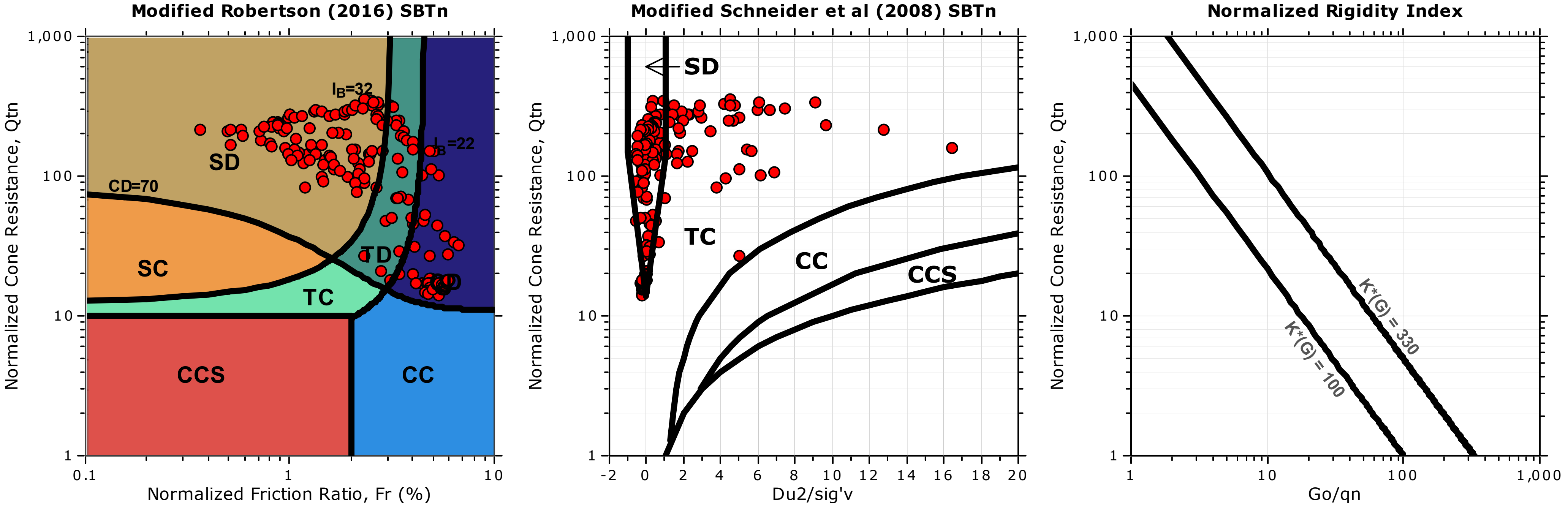
🟩 4. TC: Transitional - Contractive

🟩 5. TD: Transitional - Dilative

🟧 6. SC: Sand-like - Contractive

🟫 7. SD: Sand-like - Dilative

Updated SBTn plots



CCS: Clay-like - Contractive - Sensitive
CC: Clay-like - Contractive
CD: Clay-like - Dilative
TC: Transitional - Contractive
TD: Transitional - Dilative
SC: Sand-like - Contractive
SD: Sand-like - Dilative

K(G) > 330: Soils with significant microstructure (e.g. age/cementation)

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CPT: CPT-03

Total depth: 30.02 ft, Date: 11/1/2024

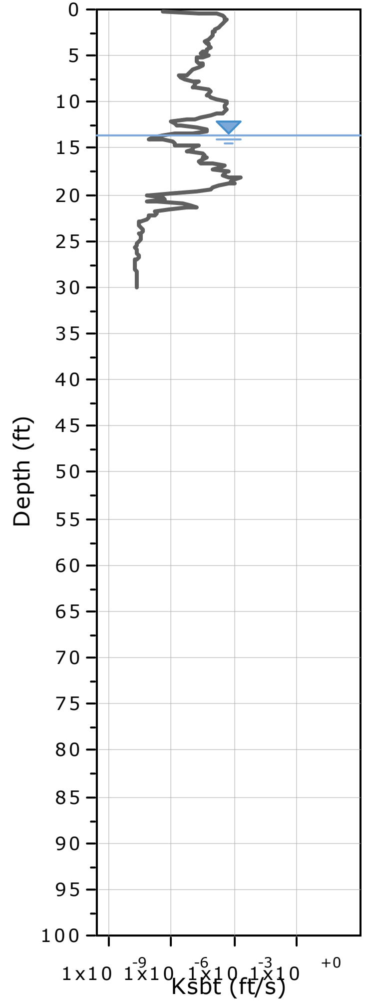
Cone Type: 15cm²

Cone Operator: JM-IY

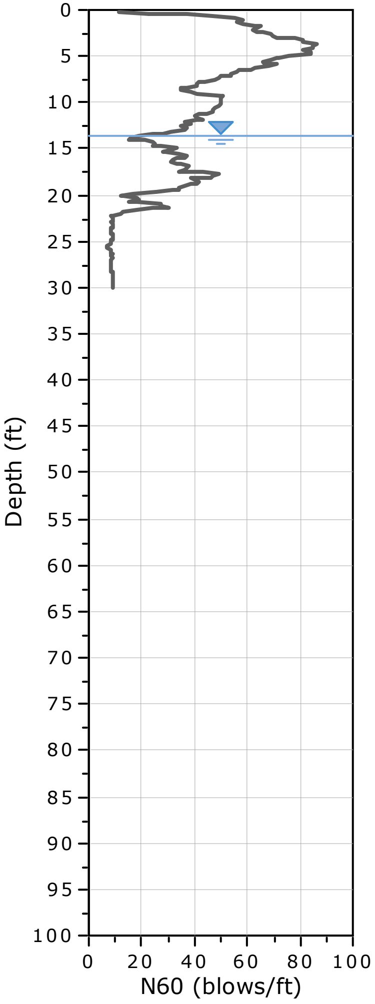
Project: **New Leaf Minto Road Watsonville**

Location: **90 Minto Road, Watsonville, CA 95076**

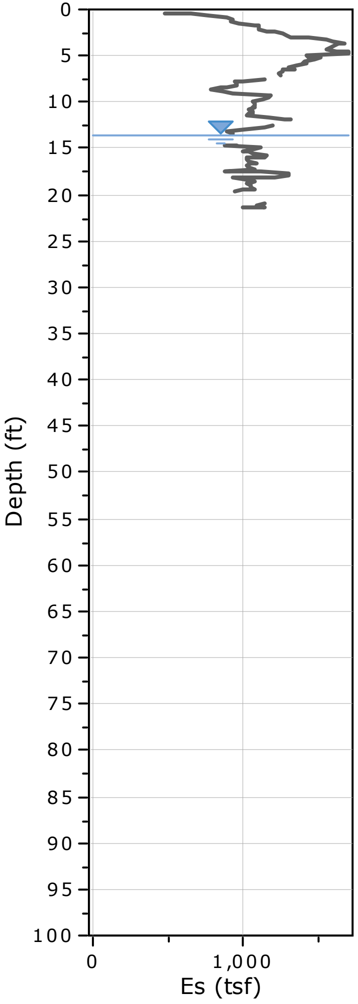
Permeability



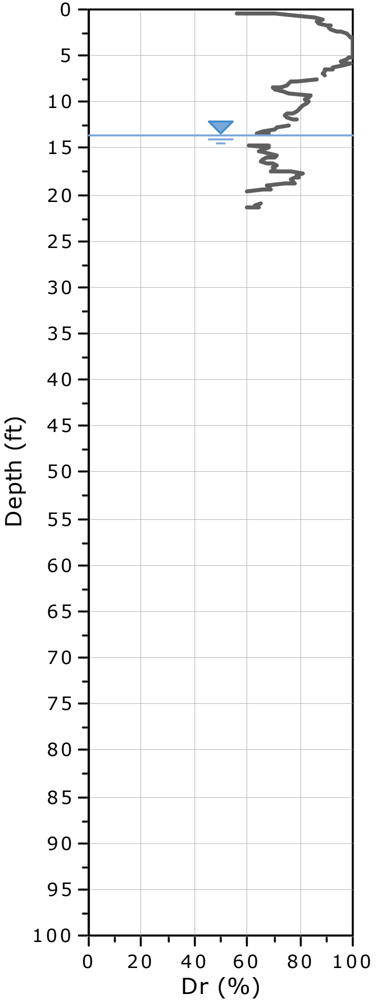
SPT N60



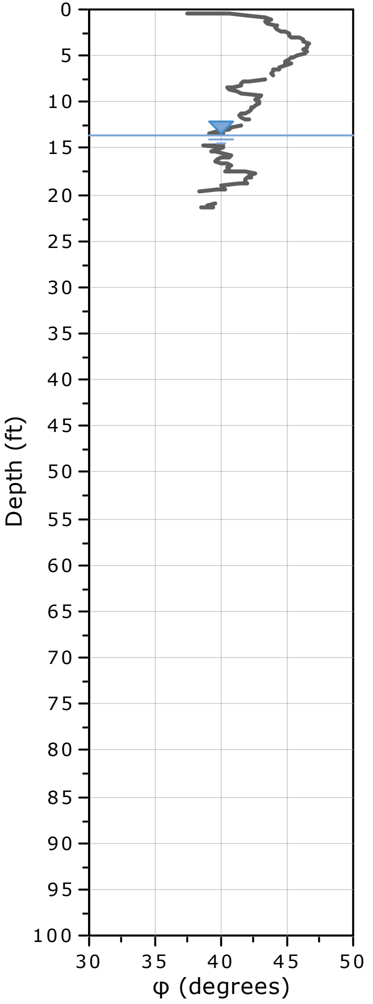
Young's modulus



Relative density



Friction angle



Calculation parameters

Permeability: Based on SBT_n

SPT N_{60}: Based on I_c and q_t

Young's modulus: Based on variable alpha using I_c (Robertson, 2009)

Relative density constant, C_{Dr}: 350.0

Phi: Based on Kulhawy & Mayne (1990)

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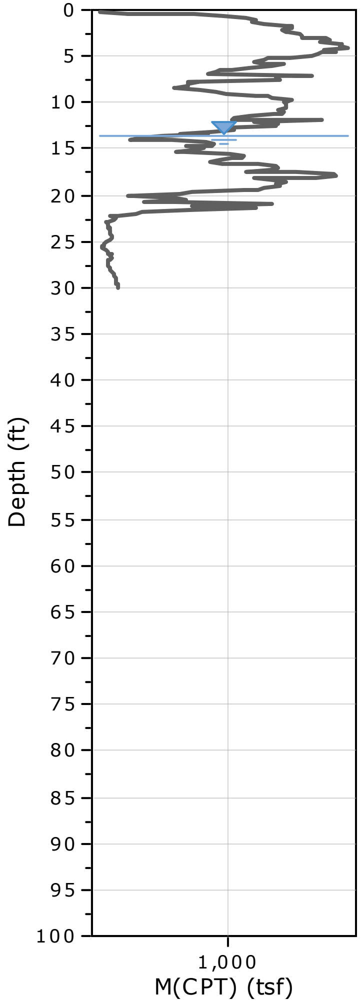
CPT: CPT-03

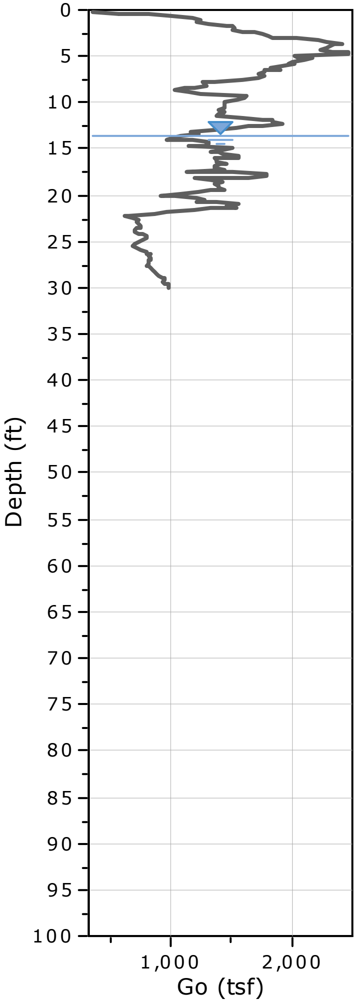
Total depth: 30.02 ft, Date: 11/1/2024
Cone Type: 15cm²
Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**
Location: **90 Minto Road, Watsonville, CA 95076**

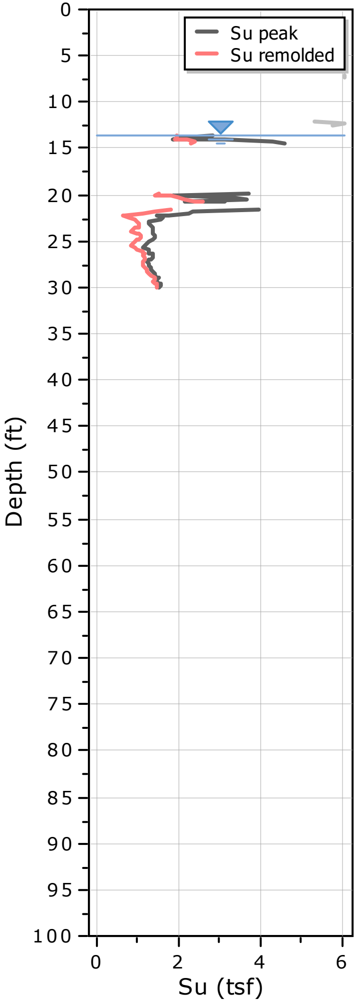
Constrained Modulus



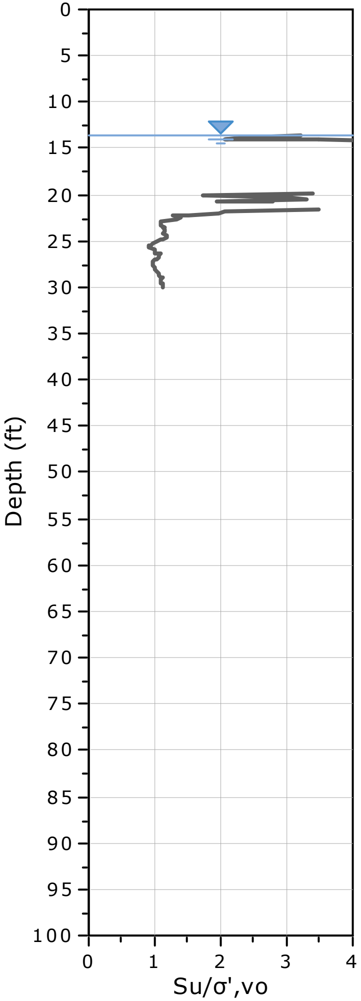
Shear modulus



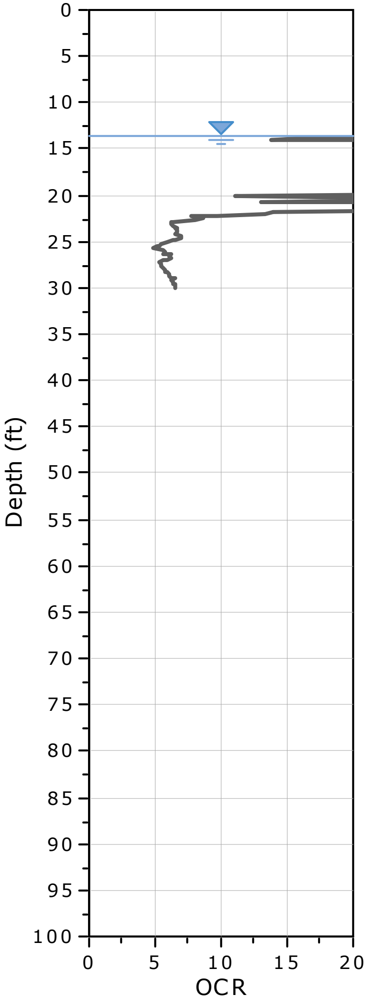
Shear strength



Undrained strength ratio



OCR



Calculation parameters

Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable *alpha* using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: Auto

OCR factor for clays, N_{kt}: Auto
●— Flat Dilatometer Test data

CPT: CPT-03

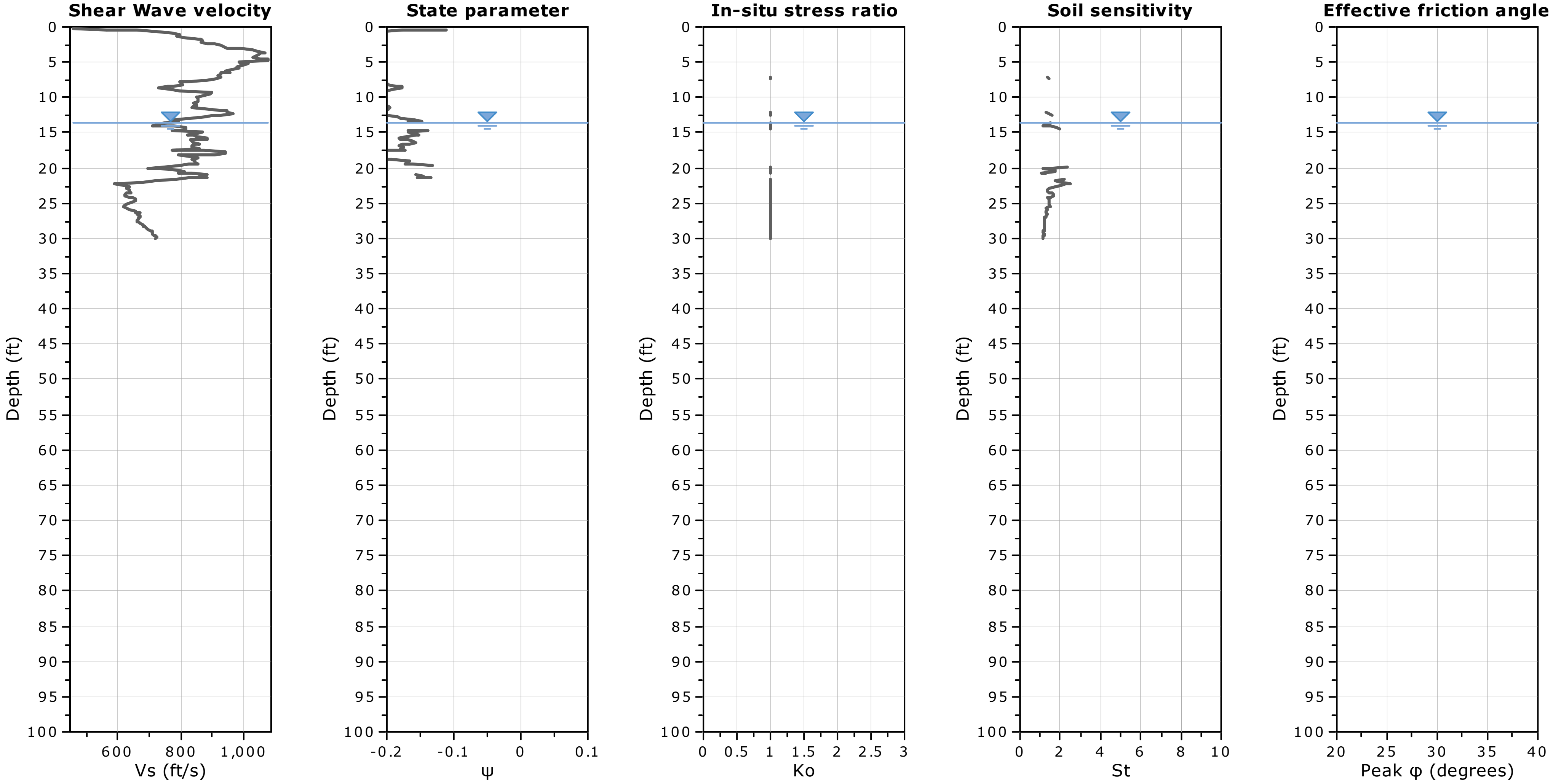
Total depth: 30.02 ft, Date: 11/1/2024

Cone Type: 15cm²

Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**

Location: **90 Minto Road, Watsonville, CA 95076**



Calculation parameters

Soil Sensitivity factor, N_S: 7.00

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CPT: CPT-03

Total depth: 30.02 ft, Date: 11/1/2024

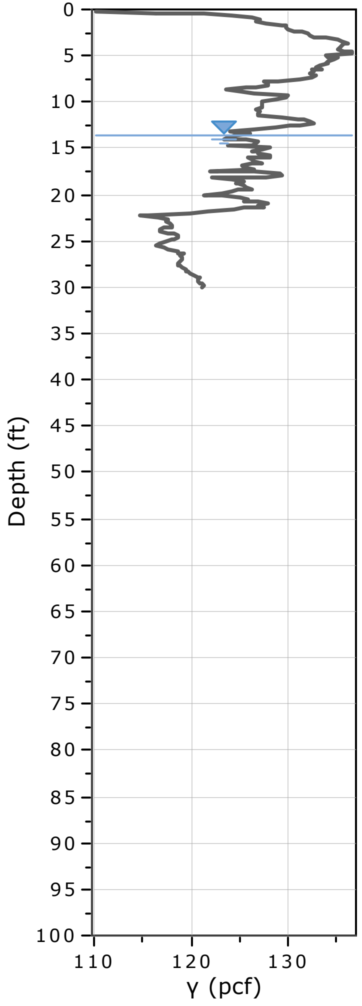
Cone Type: 15cm²

Cone Operator: JM-IY

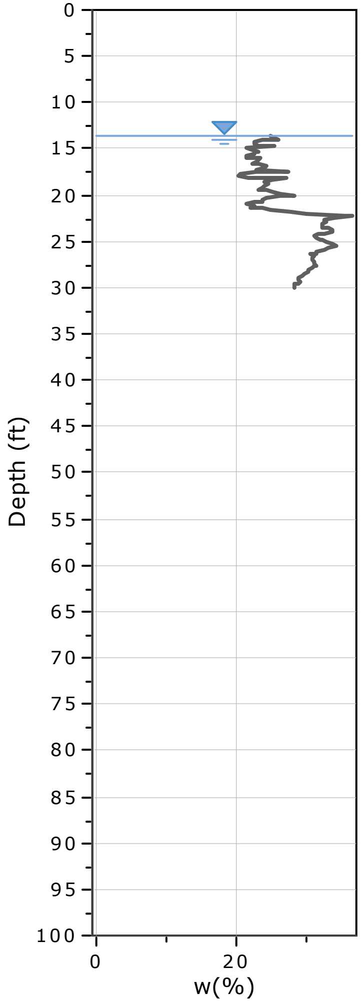
Project: **New Leaf Minto Road Watsonville**

Location: **90 Minto Road, Watsonville, CA 95076**

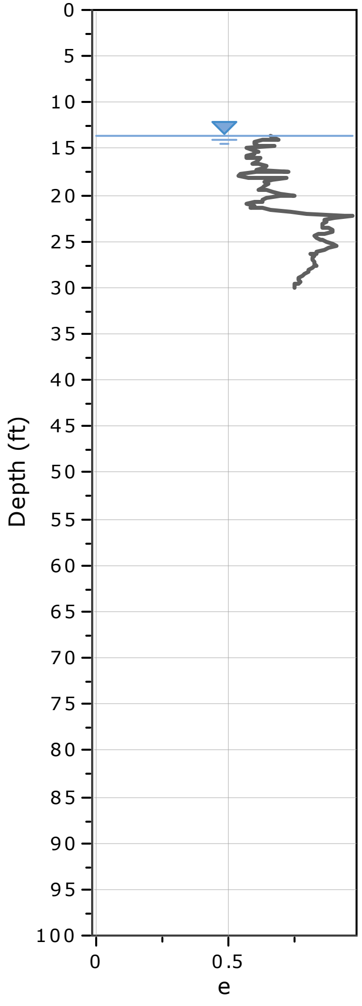
Bulk unit weight



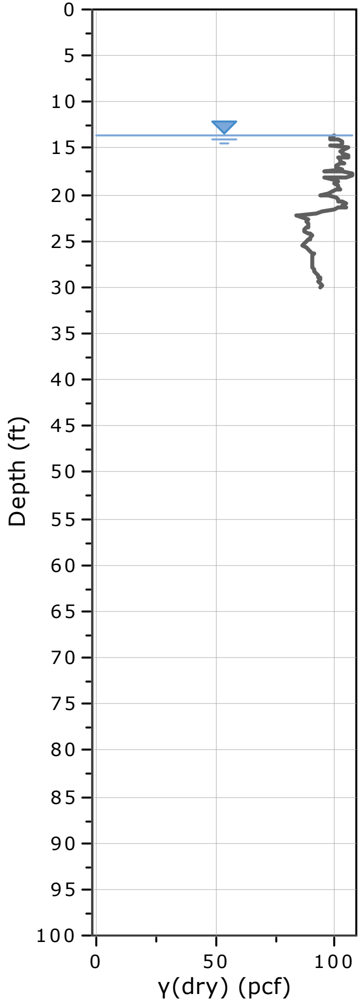
Water content



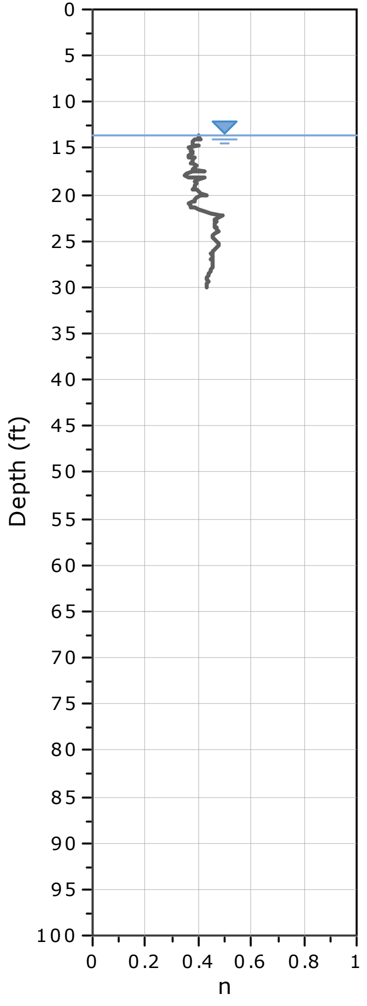
Void ratio



Dry unit weight



Porosity



Haley & Aldrich Inc.

https://www.haleyaldrich.com/

CPT: CPT-03

Total depth: 30.02 ft, Date: 11/1/2024

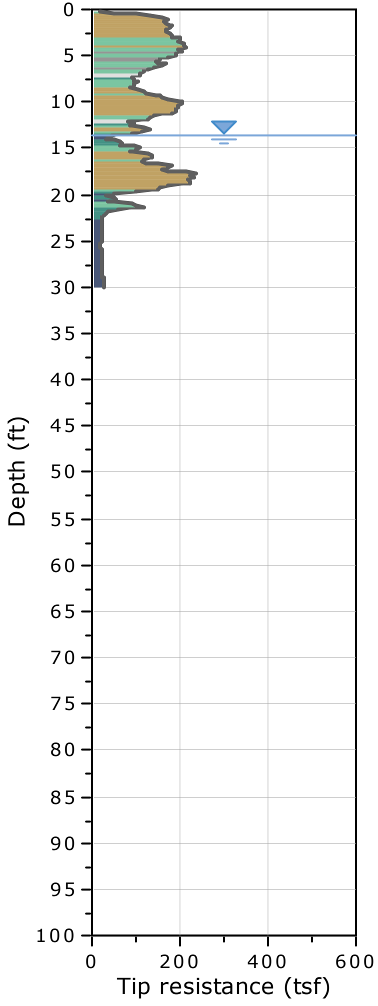
Cone Type: 15cm²

Cone Operator: JM-IY

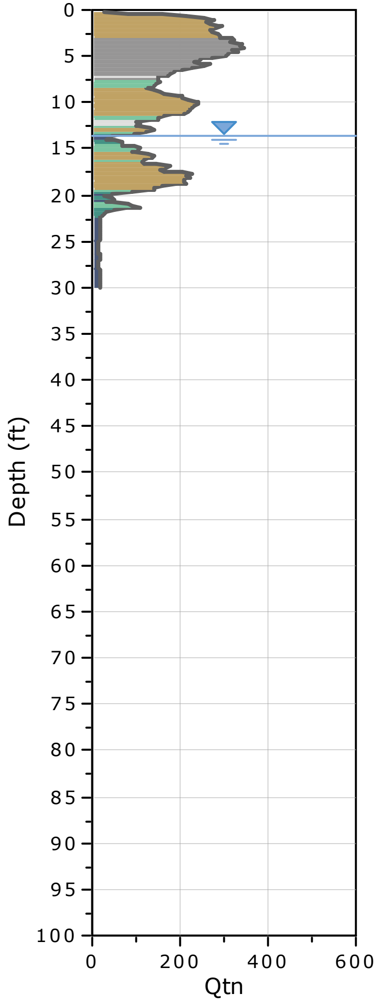
Project: **New Leaf Minto Road Watsonville**

Location: **90 Minto Road, Watsonville, CA 95076**

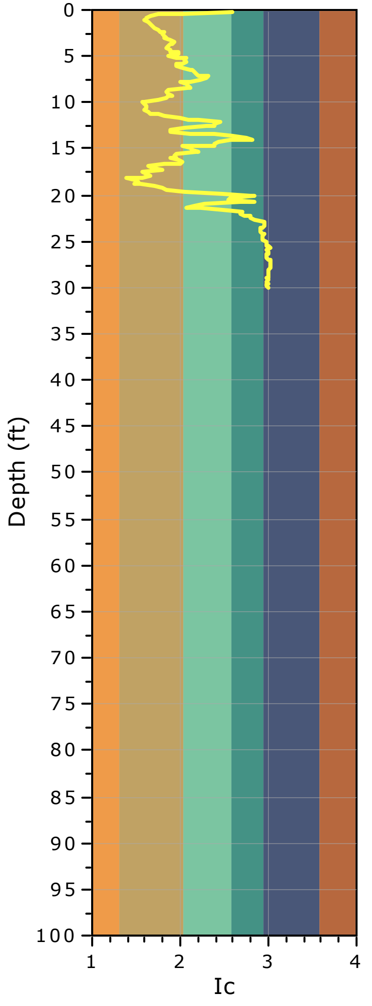
Cone resistance qt



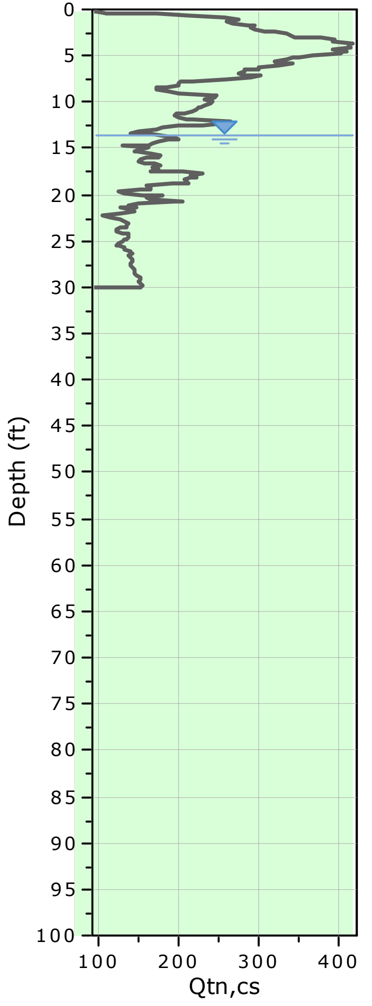
Norm. cone resistance



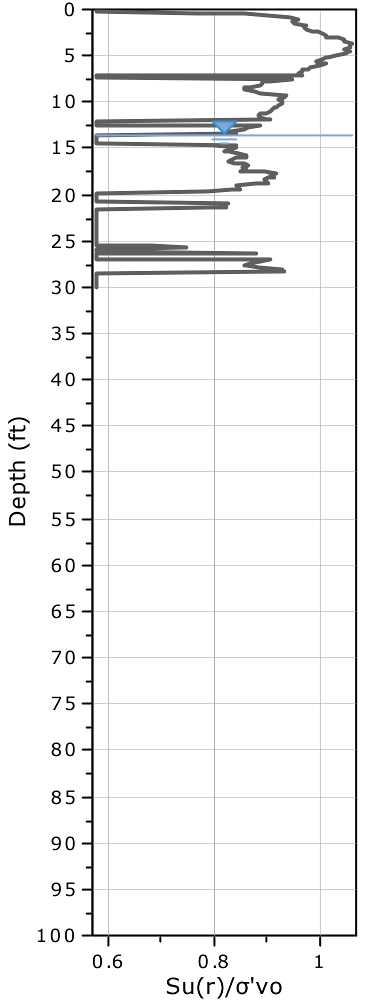
SBTn Index



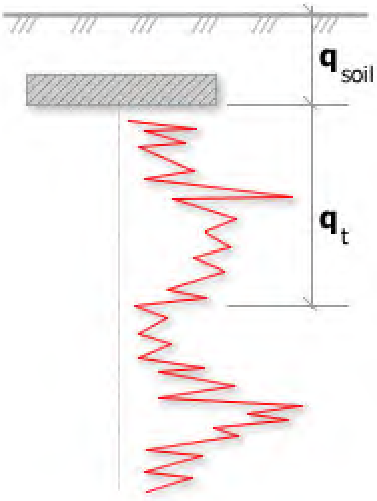
Clean sand equivalent



Residual strength ratio



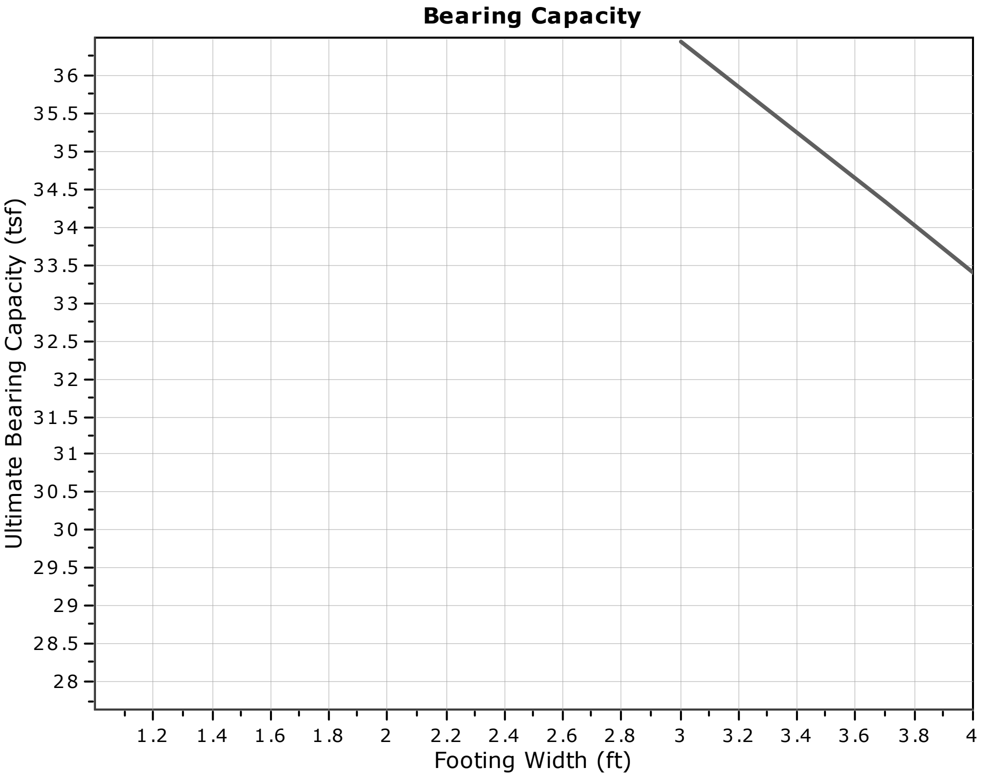
Project: **New Leaf Minto Road Watsonville**
Location: **90 Minto Road, Watsonville, CA 95076**



Bearing Capacity calculation is perfromed based on the formula:

$$Q_{ult} = R_k \times q_t + q_{soil}$$

where:
R_k: Bearing capacity factor
q_t: Average corrected cone resistance over calculation depth
q_{soil}: Pressure applied by soil above footing



:: Tabular results ::

No	B (ft)	Start Depth (ft)	End Depth (ft)	Ave. q_t (tsf)	R_k	Soil Press. (tsf)	Ult. bearing cap. (tsf)
1	3.00	1.60	6.10	181.74	0.20	0.10	36.44
2	3.70	1.60	7.15	171.25	0.20	0.10	34.35
3	4.40	1.60	8.20	160.23	0.20	0.10	32.14
4	5.10	1.60	9.25	152.65	0.20	0.10	30.63
5	5.80	1.60	10.30	155.94	0.20	0.10	31.28
6	6.50	1.60	11.35	159.45	0.20	0.10	31.99
7	7.20	1.60	12.40	155.46	0.20	0.10	31.19
8	7.90	1.60	13.45	151.35	0.20	0.10	30.37
9	8.60	1.60	14.50	142.48	0.20	0.10	28.59
10	9.30	1.60	15.55	138.90	0.20	0.10	27.88
11	10.00	1.60	16.60	137.97	0.20	0.10	27.69
12	10.70	1.60	17.65	140.18	0.20	0.10	28.13
13	11.40	1.60	18.70	145.12	0.20	0.10	29.12
14	12.10	1.60	19.75	146.22	0.20	0.10	29.34
15	12.80	1.60	20.80	140.94	0.20	0.10	28.28

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236\right)$$

where $g_w =$ water unit weight

:: Permeability, k (m/s) ::

$$I_c < 3.27 \text{ and } I_c > 1.00 \text{ then } k = 10^{0.952\text{-}3.04 \cdot I_c}$$

$$I_c \leq 4.00 \text{ and } I_c > 3.27 \text{ then } k = 10^{-4.52\text{-}1.37 \cdot I_c}$$

:: N$_{SPT}$ (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268 - 0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268 - 0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT$_n$: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Drained Friction Angle, φ (°) ::

$$\varphi = \varphi'_{cv} + 15.94 \cdot \log(Q_{tn,cs}) - 26.88$$

(applicable only to SBT$_n$: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$
$\alpha = 14$ for $Q_{tn} > 14$
$\alpha = Q_{tn}$ for $Q_{tn} \leq 14$
$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$

If $I_c \geq 2.20$

$$M_{CPT} = 0.03 \cdot (q_t - \sigma_v) \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Small strain shear Modulus, Go (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Shear Wave Velocity, Vs (m/s) ::

$$V_s = \left(\frac{G_0}{\rho}\right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot + 7 \cdot \log(F_r))}\right]^{1.25} \text{ or user defined}$$

$$OCR = k_{OCR} \cdot Q_{tn}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_O = (1 - \sin\varphi') \cdot OCR^{\sin\varphi'}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Soil Sensitivity, S$_t$ ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Peak Friction Angle, φ' (°) ::

$$\varphi' = 29.5° \cdot B_q^{0.121} \cdot (0.256 + 0.336 \cdot B_q + \log Q_t)$$

(applicable for $0.10 < B_q < 1.00$)

References

- Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 5th Edition, November 2012
- Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)
- N Barounis, J Philpot, Estimation of in-situ water content, void ratio, dry unit weight and porosity using CPT for saturated sands, Proc. 20th NZGS Geotechnical Symposium

CPT: CPT-03

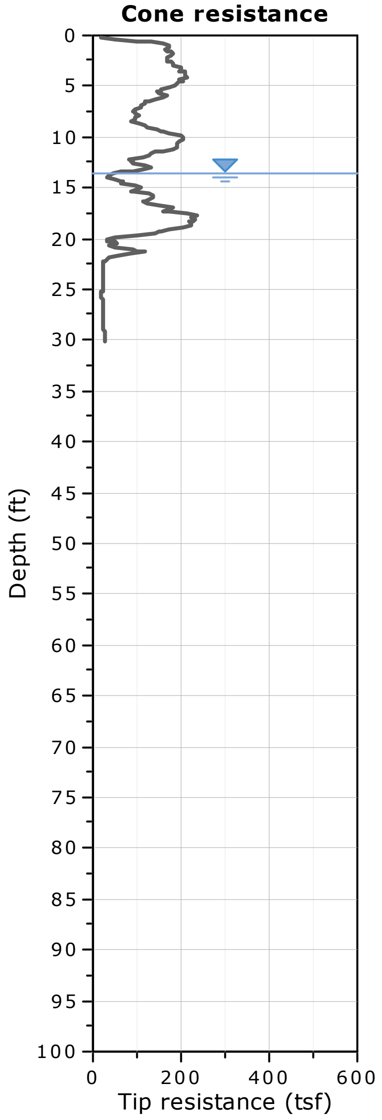
Total depth: 30.02 ft, Date: 11/1/2024

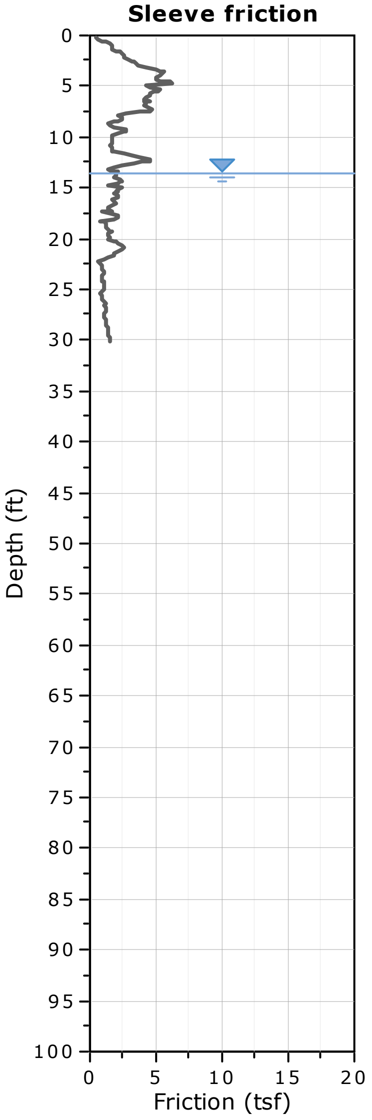
Cone Type: 15cm²

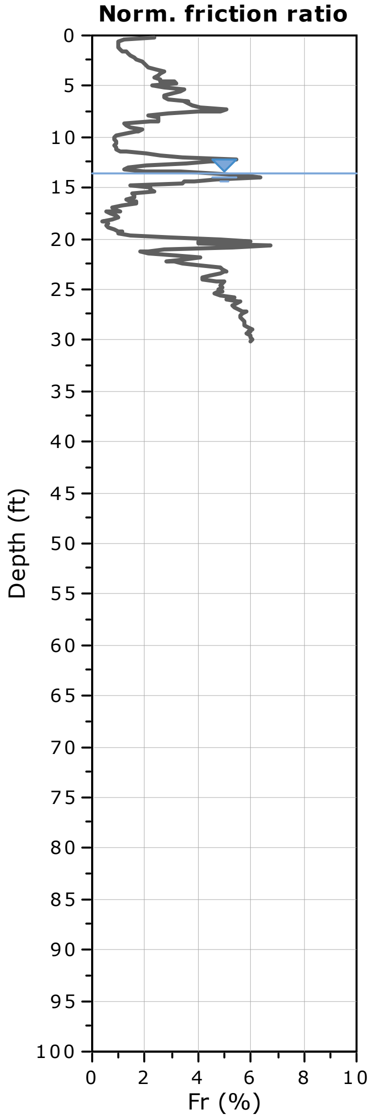
Cone Operator: JM-IY

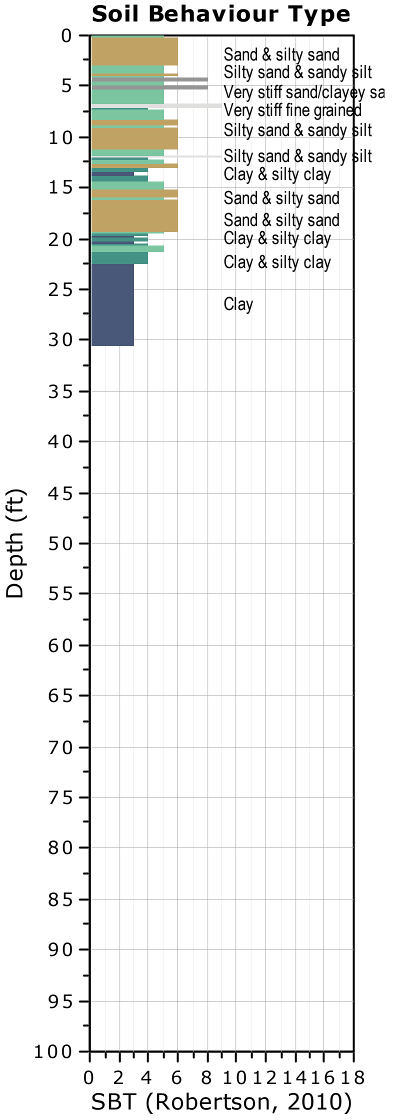
Project: New Leaf Minto Road Watsonville

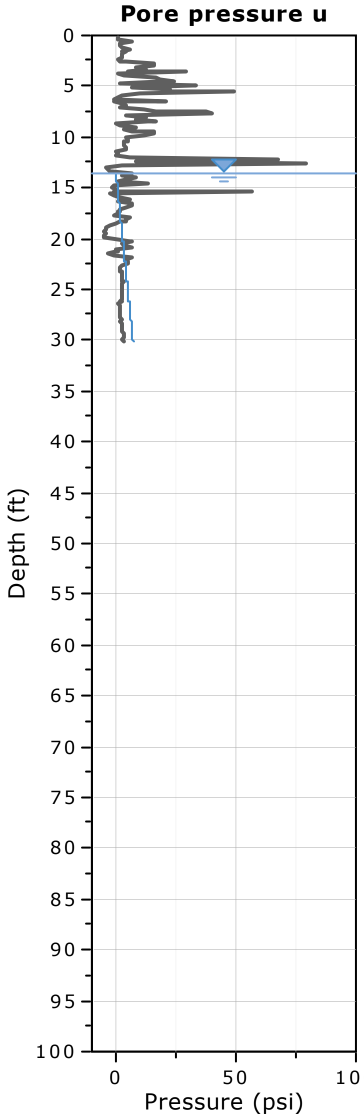
Location: 90 Minto Road, Watsonville, CA 95076











CPT: CPT-04

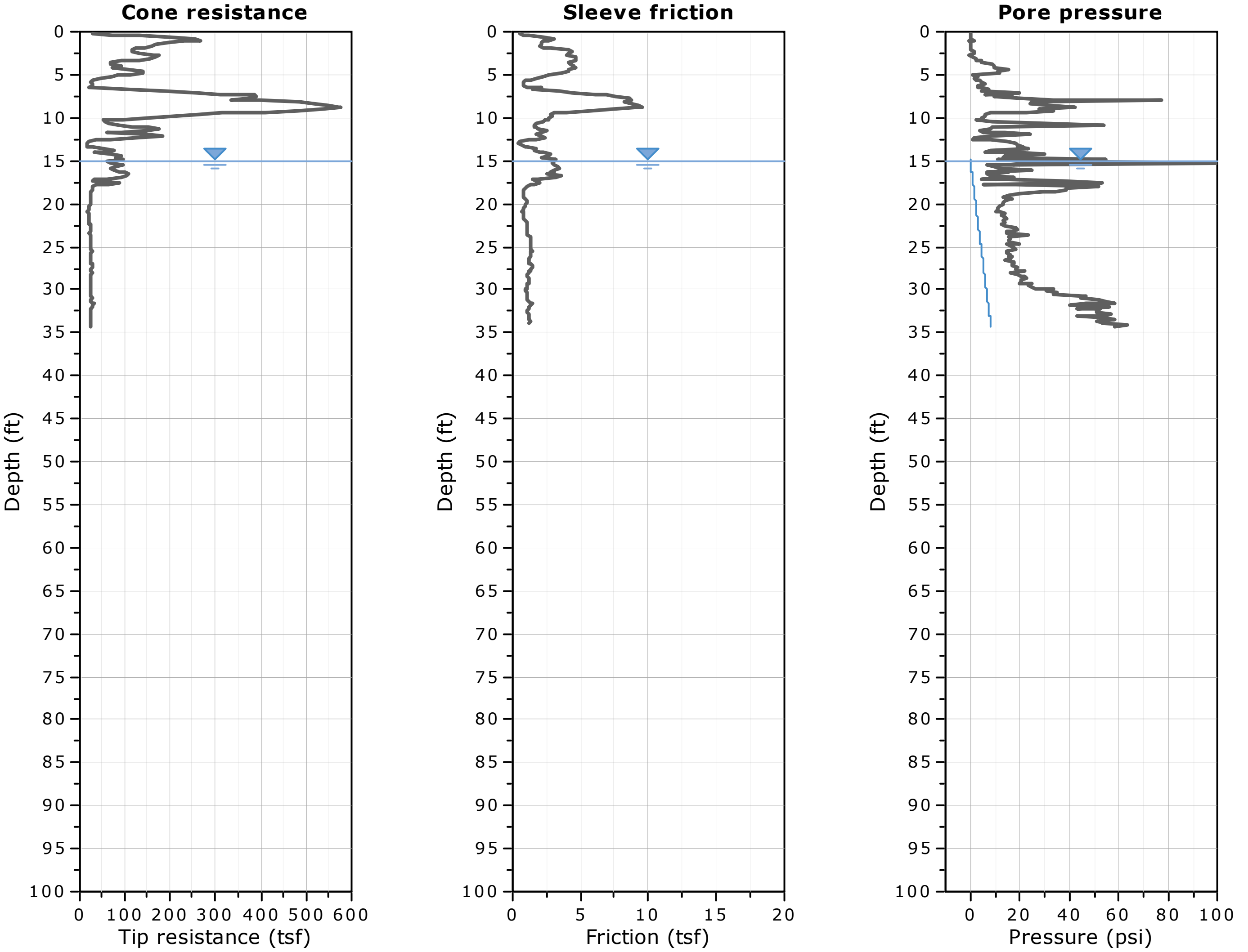
Total depth: 34.28 ft, Date: 11/1/2024

Cone Type: 15cm²

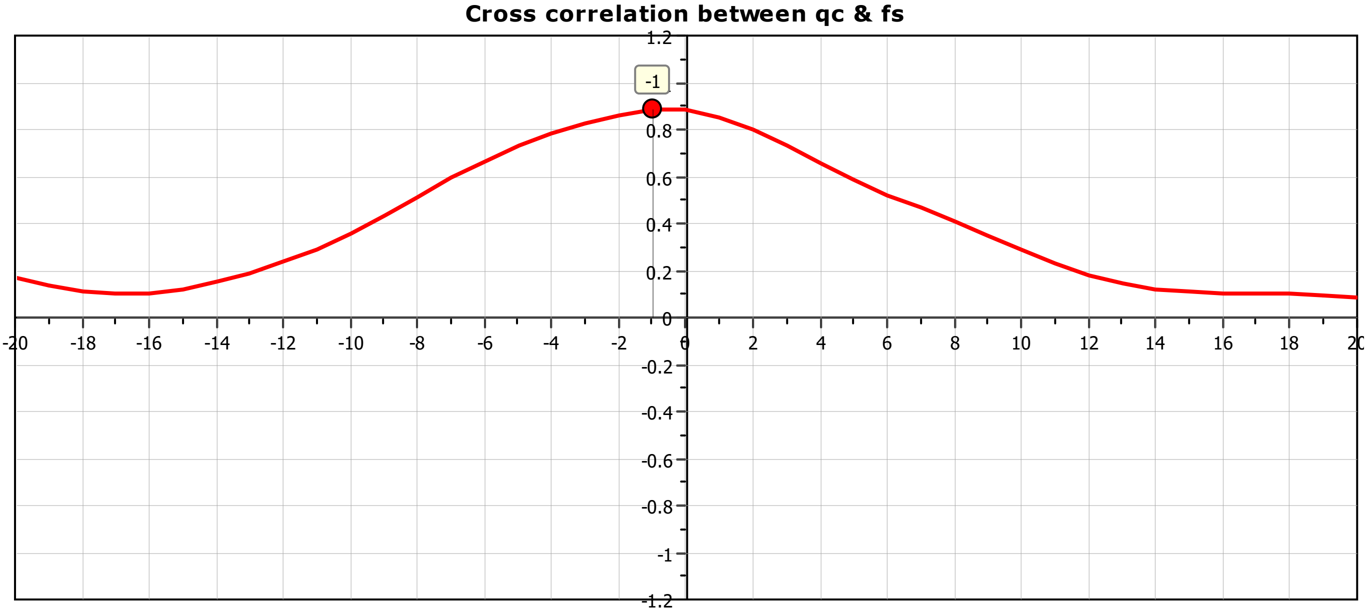
Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**

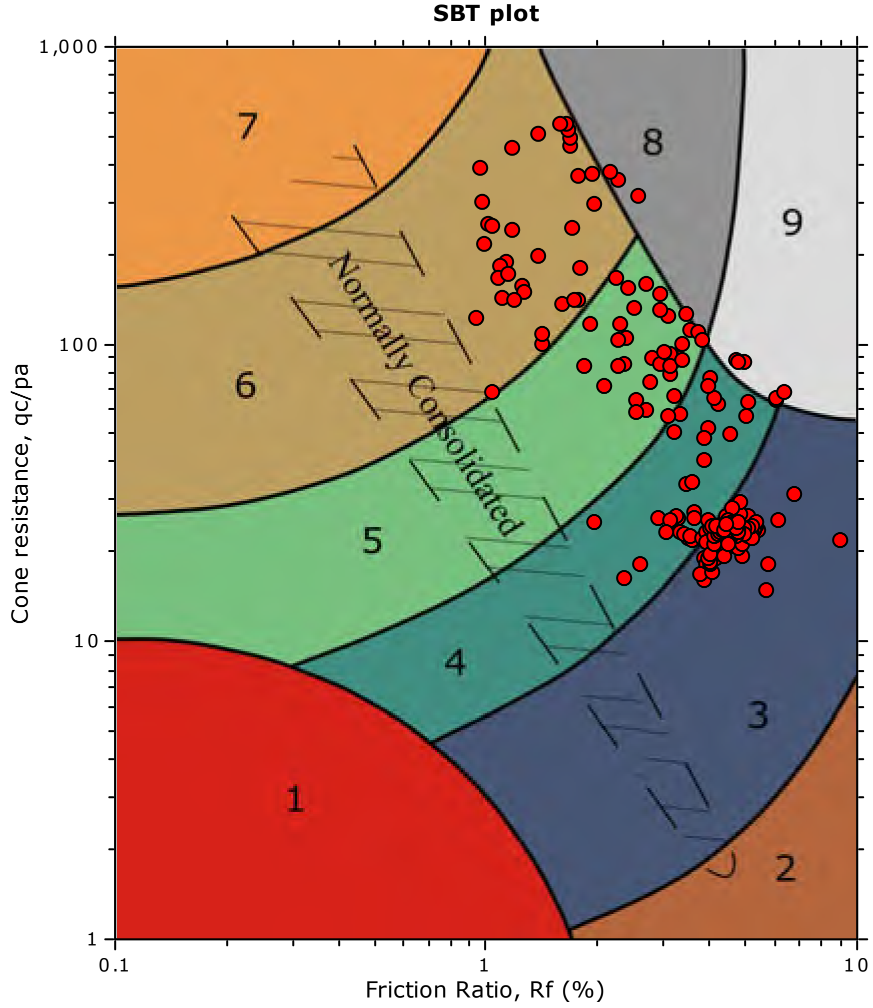
Location: **90 Minto Road, Watsonville, CA 95076**



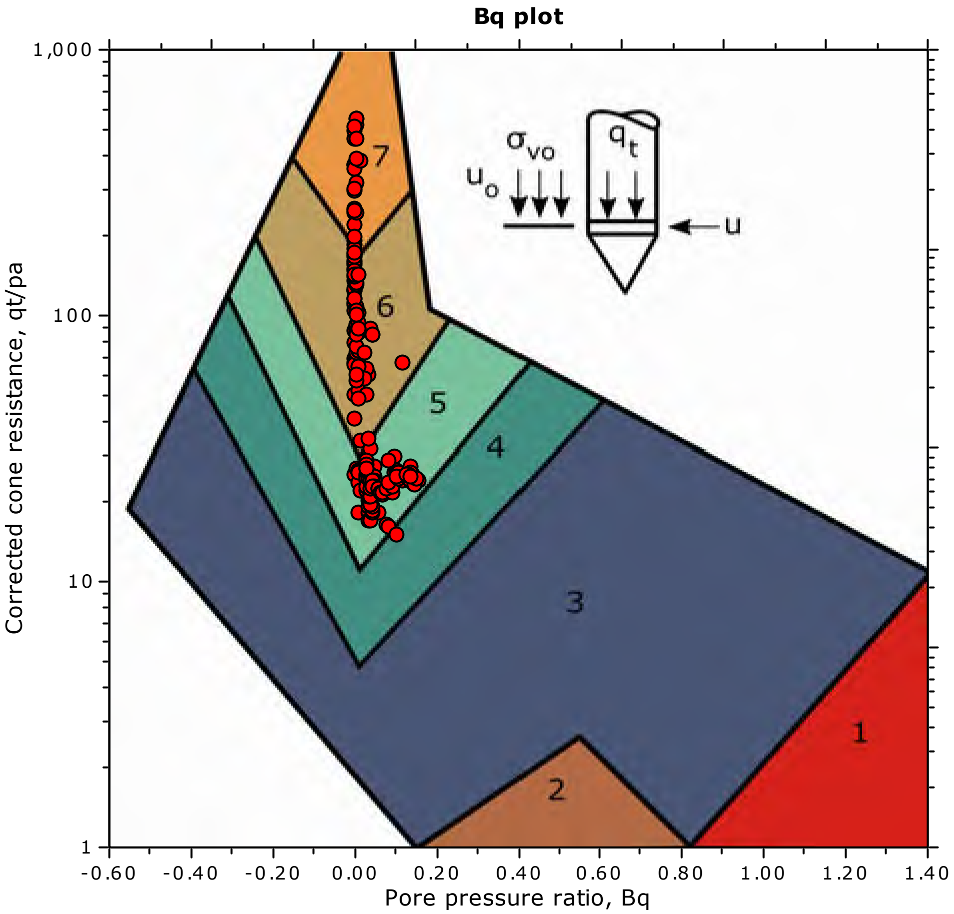
The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



SBT - Bq plots



SBT plot

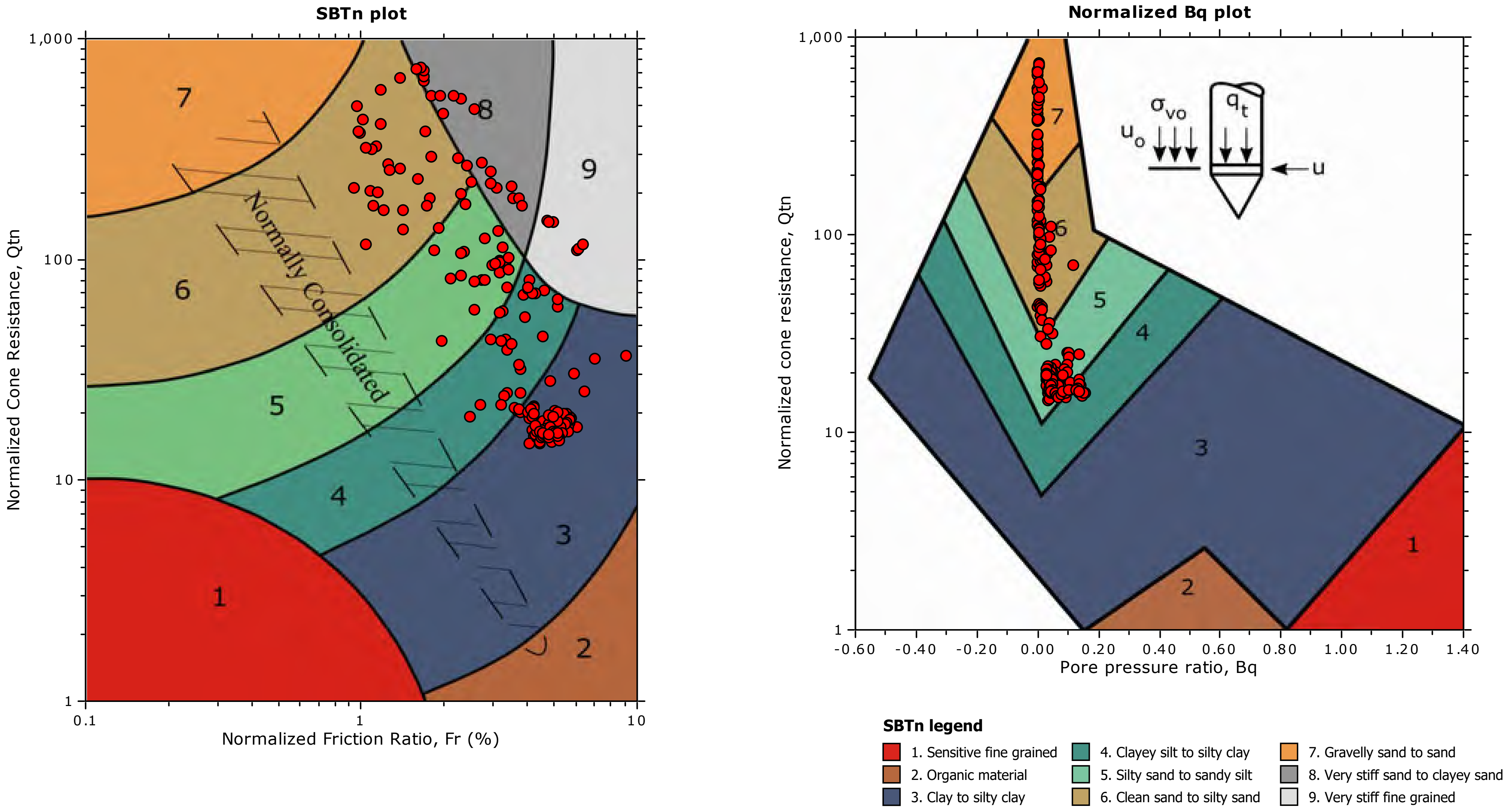


Bq plot

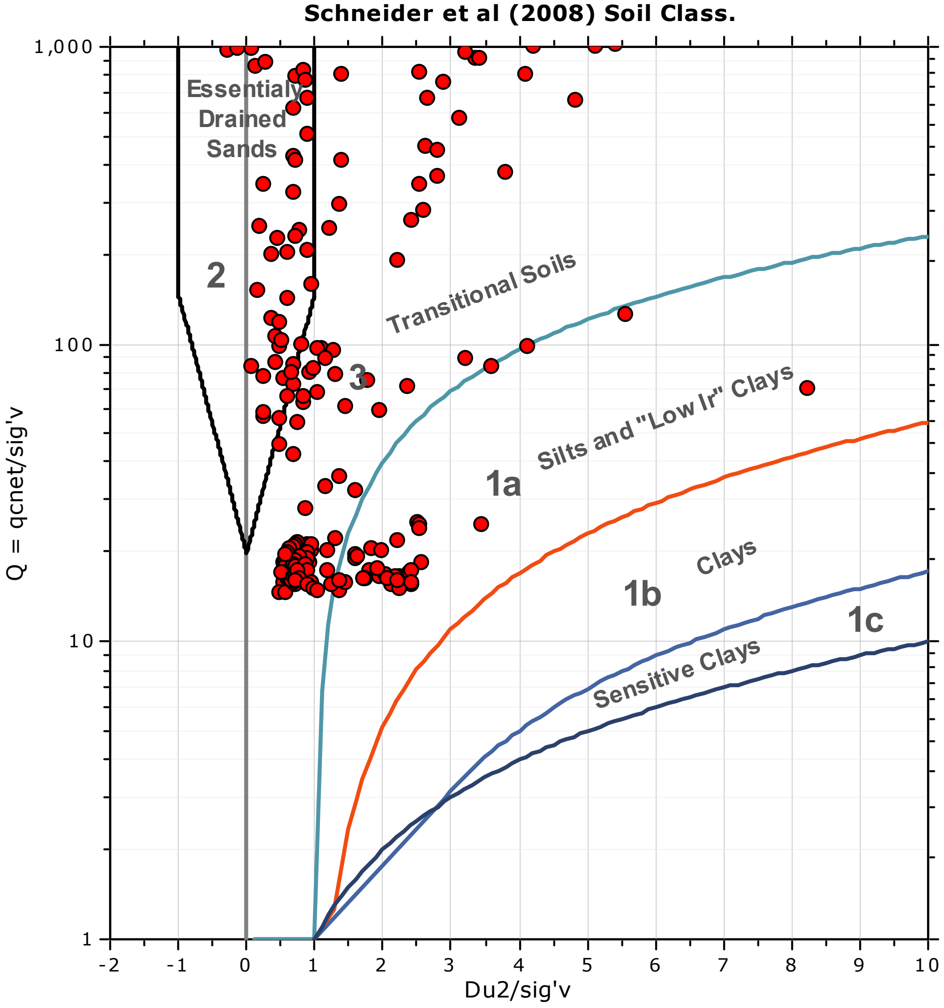
SBT legend

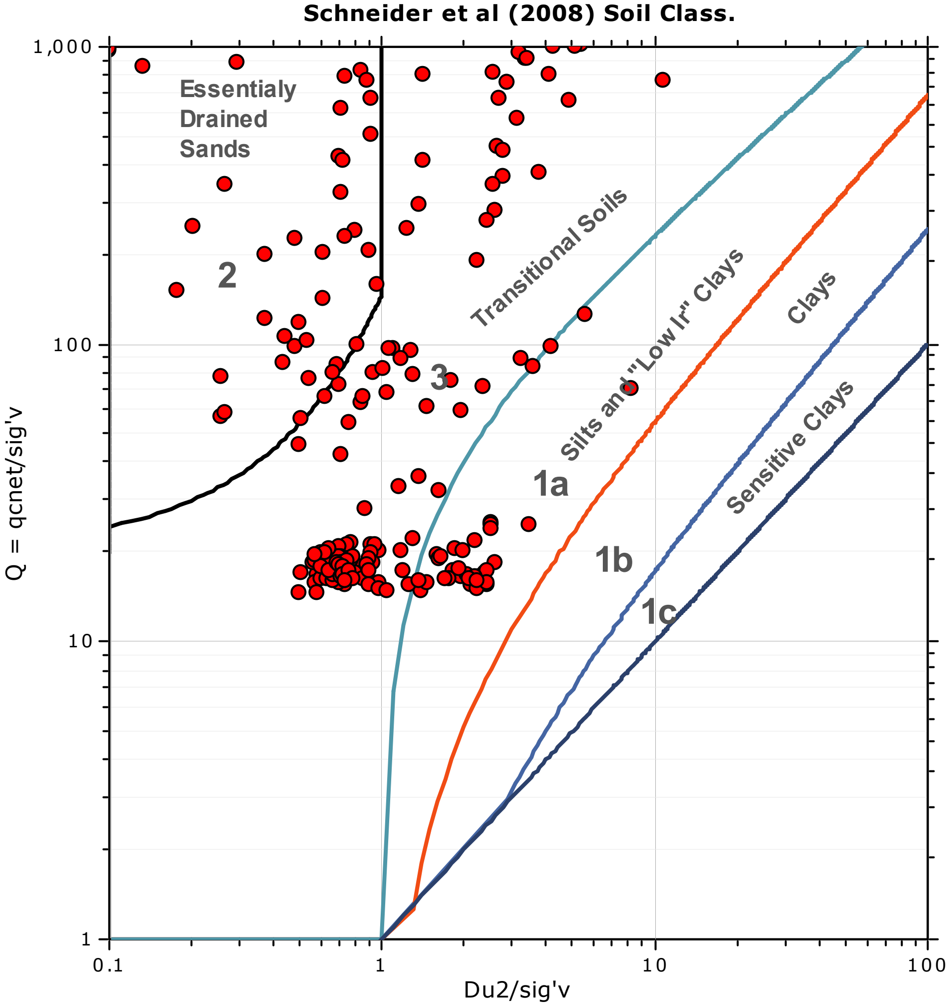
🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟫 6. Clean sand to silty sand
🟧 7. Gravelly sand to sand
⬛ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

SBT - Bq plots (normalized)



Bq plots (Schneider)





CPT: CPT-04

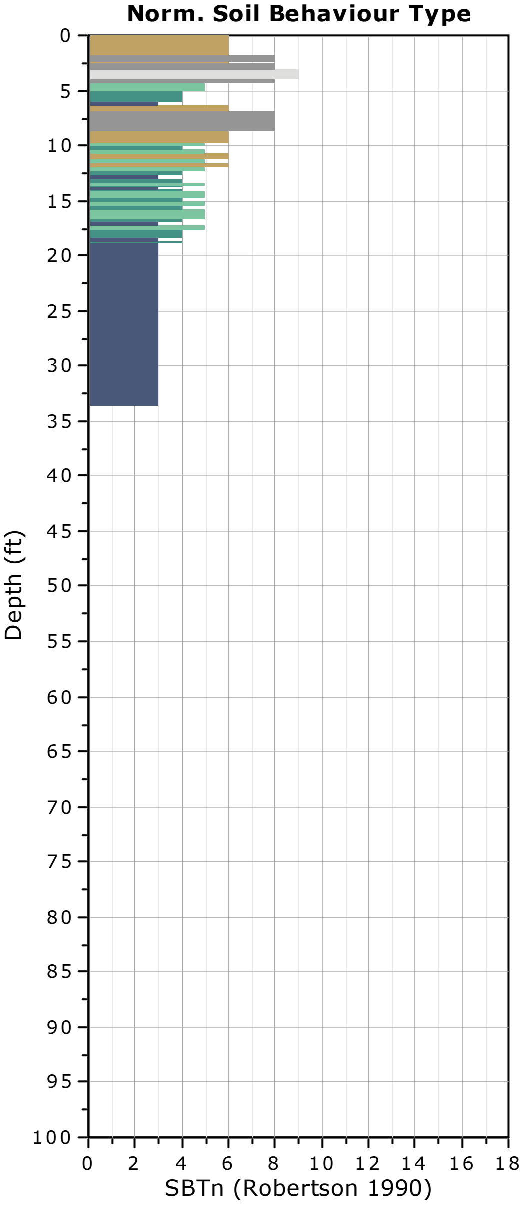
Total depth: 34.28 ft, Date: 11/1/2024

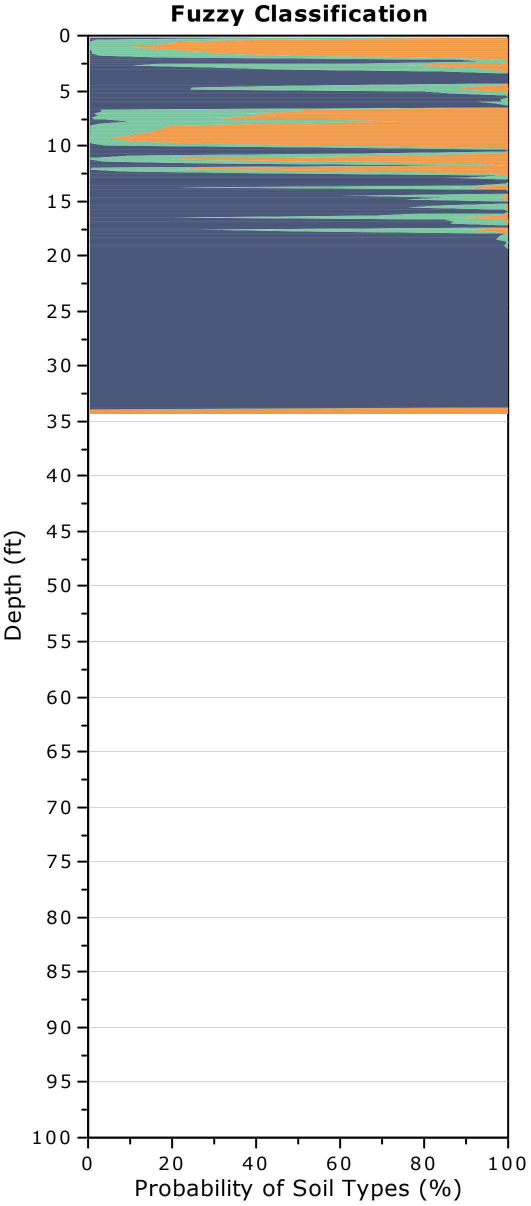
Cone Type: 15cm²

Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**

Location: **90 Minto Road, Watsonville, CA 95076**





Fuzzy classifiaction legend

■	Highly probable clayey soil
■	Highly probable mixture soil
■	Highly probable sandy soil

Haley & Aldrich Inc.

https://www.haleyaldrich.com/

Project: New Leaf Minto Road Watsonville
Location: 90 Minto Road, Watsonville, CA 95076

CPT: CPT-04
Total depth: 34.28 ft, Date: 11/1/2024
Cone Type: 15cm²
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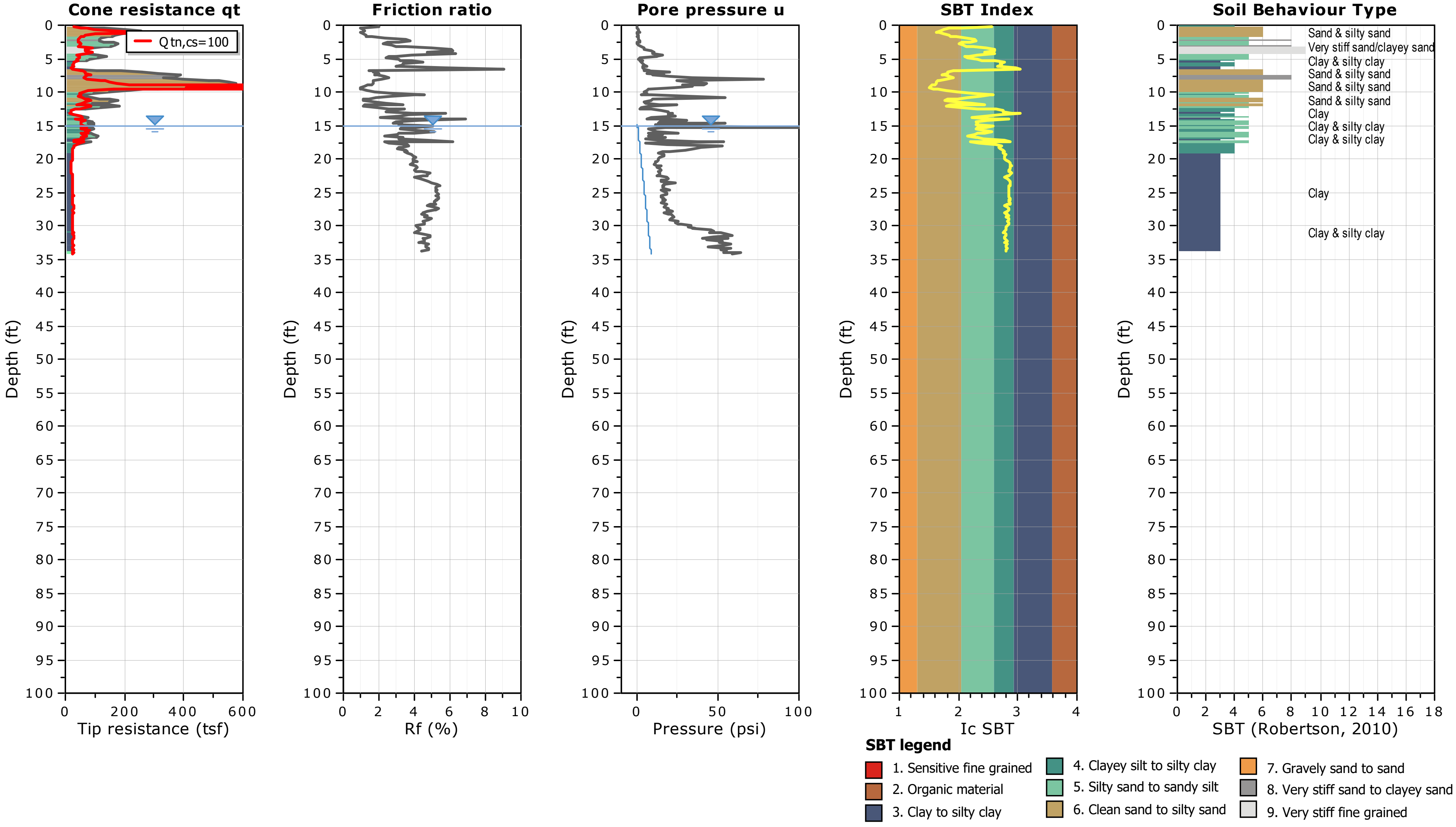

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CPT: CPT-04

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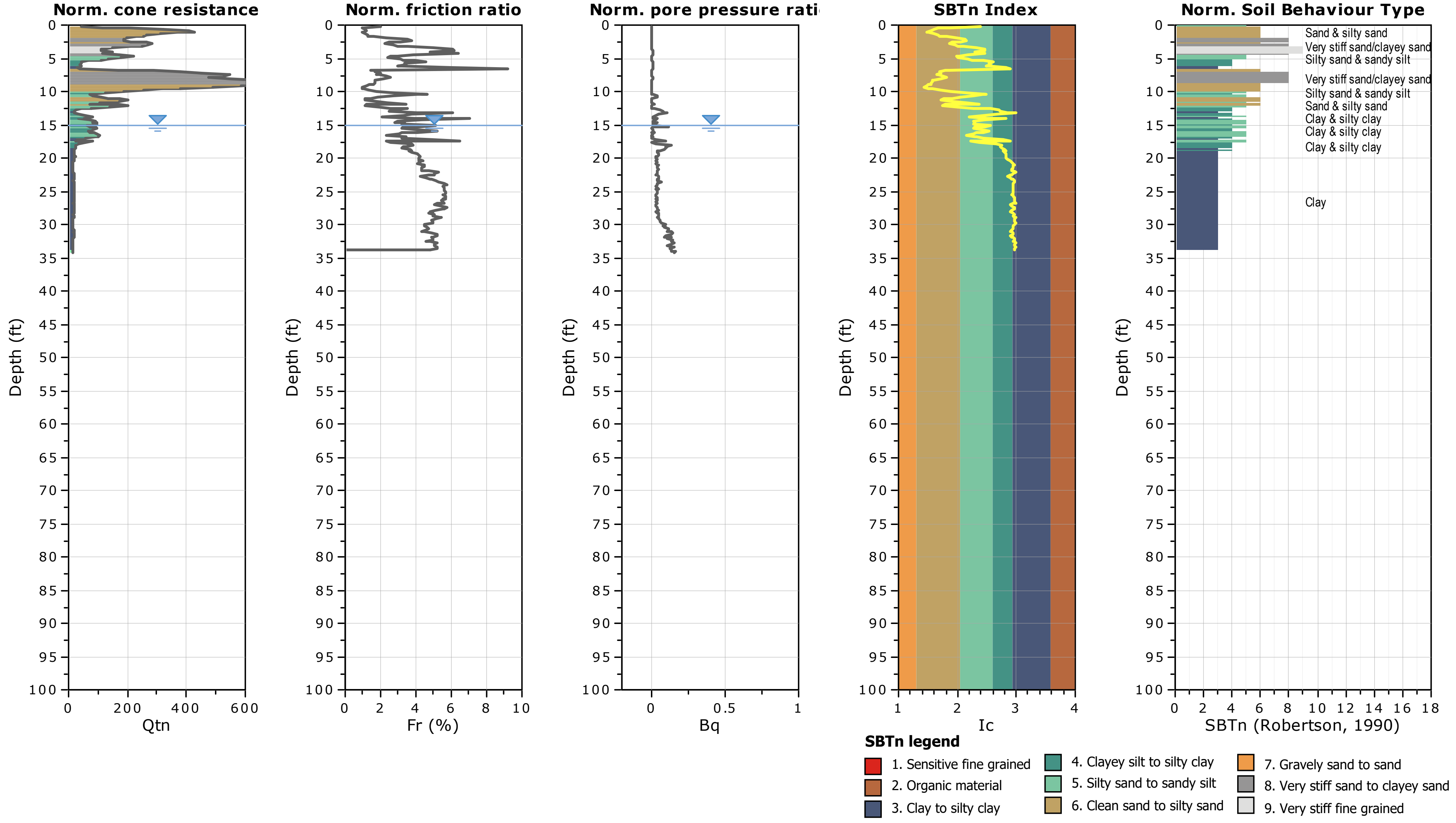

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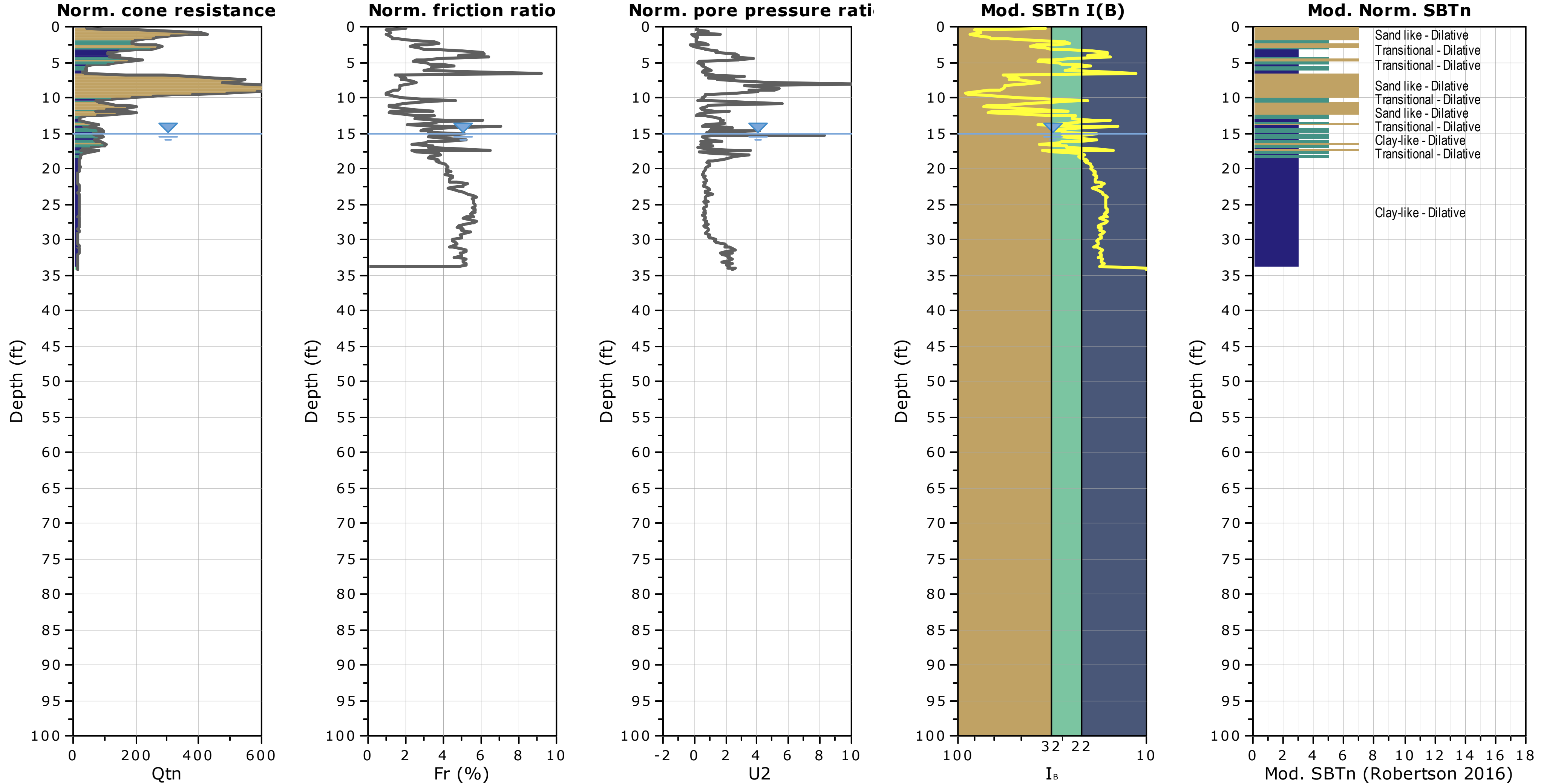
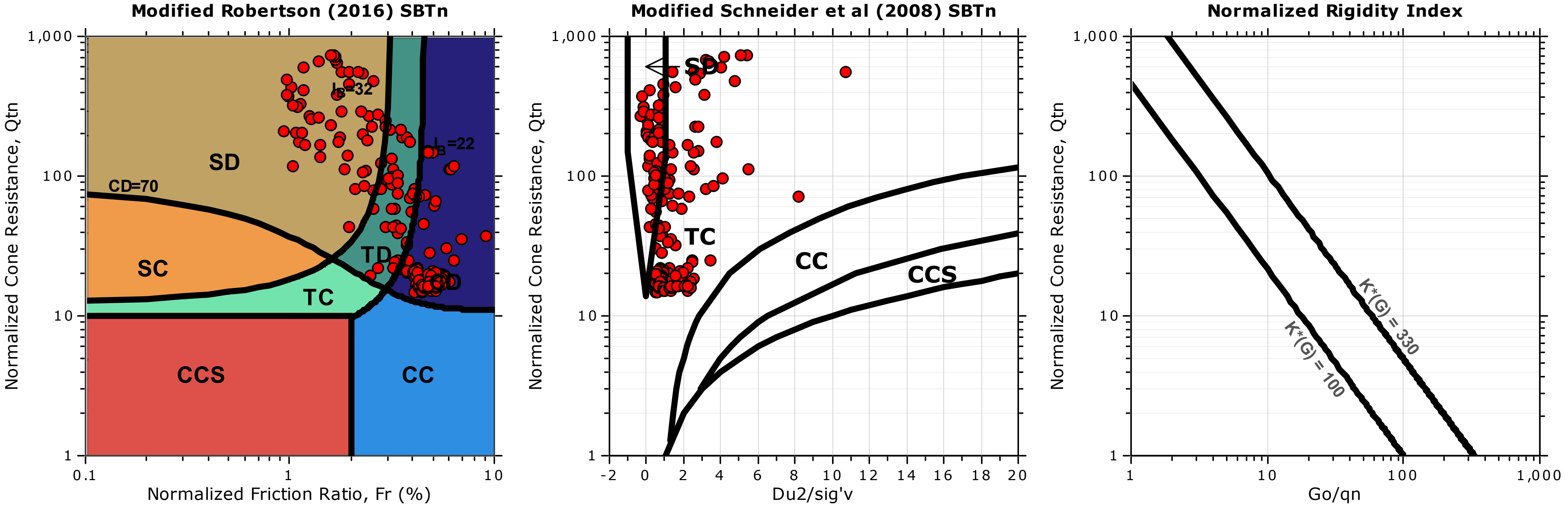


Mod. SBTn legend

- 🟥 1. CCS: ClayLike - Contractive, Sensitive
- 🟦 2. CC: Clay-like - Contractive
- 🟦 3. CD: Clay-Like: Dilative
- 🟩 4. TC: Transitional - Contractive
- 🟩 5. TD: Transitional - Dilative
- 🟧 6. SC: Sand-like - Contractive
- 🟫 7. SD: Sand-like - Dilative

Updated SBTn plots

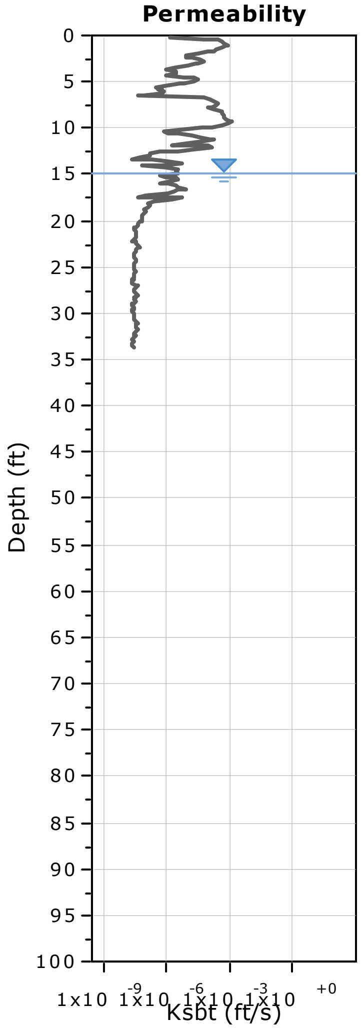


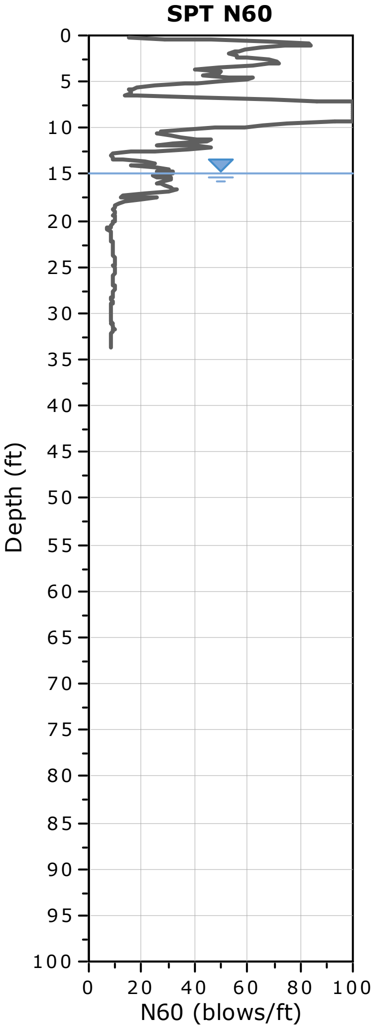
CCS:	Clay-like - Contractive - Sensitive
CC:	Clay-like - Contractive
CD:	Clay-like - Dilative
TC:	Transitional - Contractive
TD:	Transitional - Dilative
SC:	Sand-like - Contractive
SD:	Sand-like - Dilative

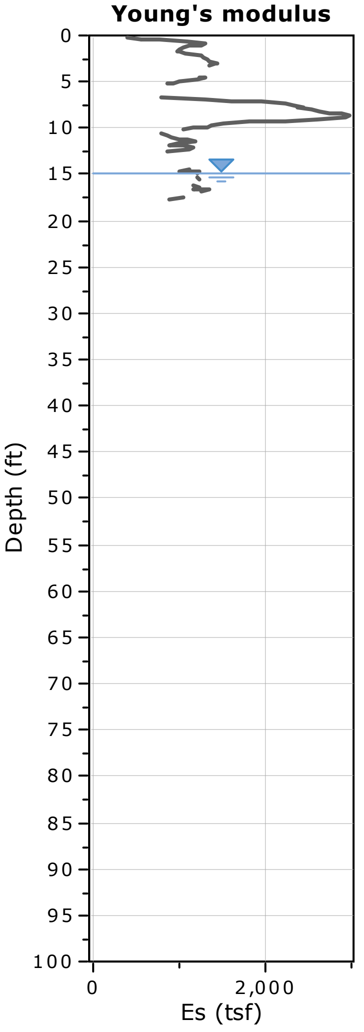
K(G) > 330: Soils with significant microstructure (e.g. age/cementation)

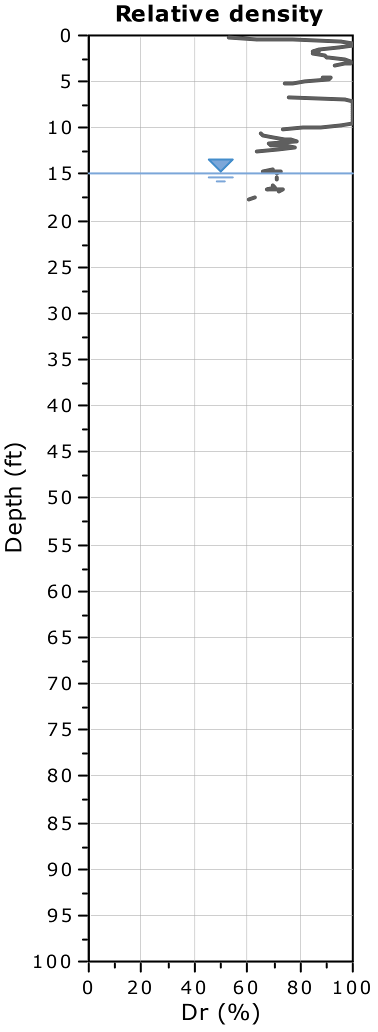
CPT: CPT-04
Total depth: 34.28 ft, Date: 11/1/2024
Cone Type: 15cm²
Cone Operator: JM-IY

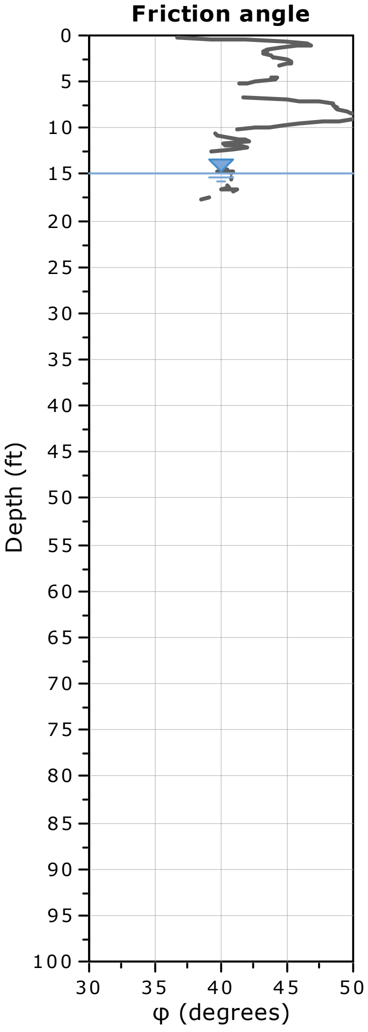
Project: **New Leaf Minto Road Watsonville**
Location: **90 Minto Road, Watsonville, CA 95076**







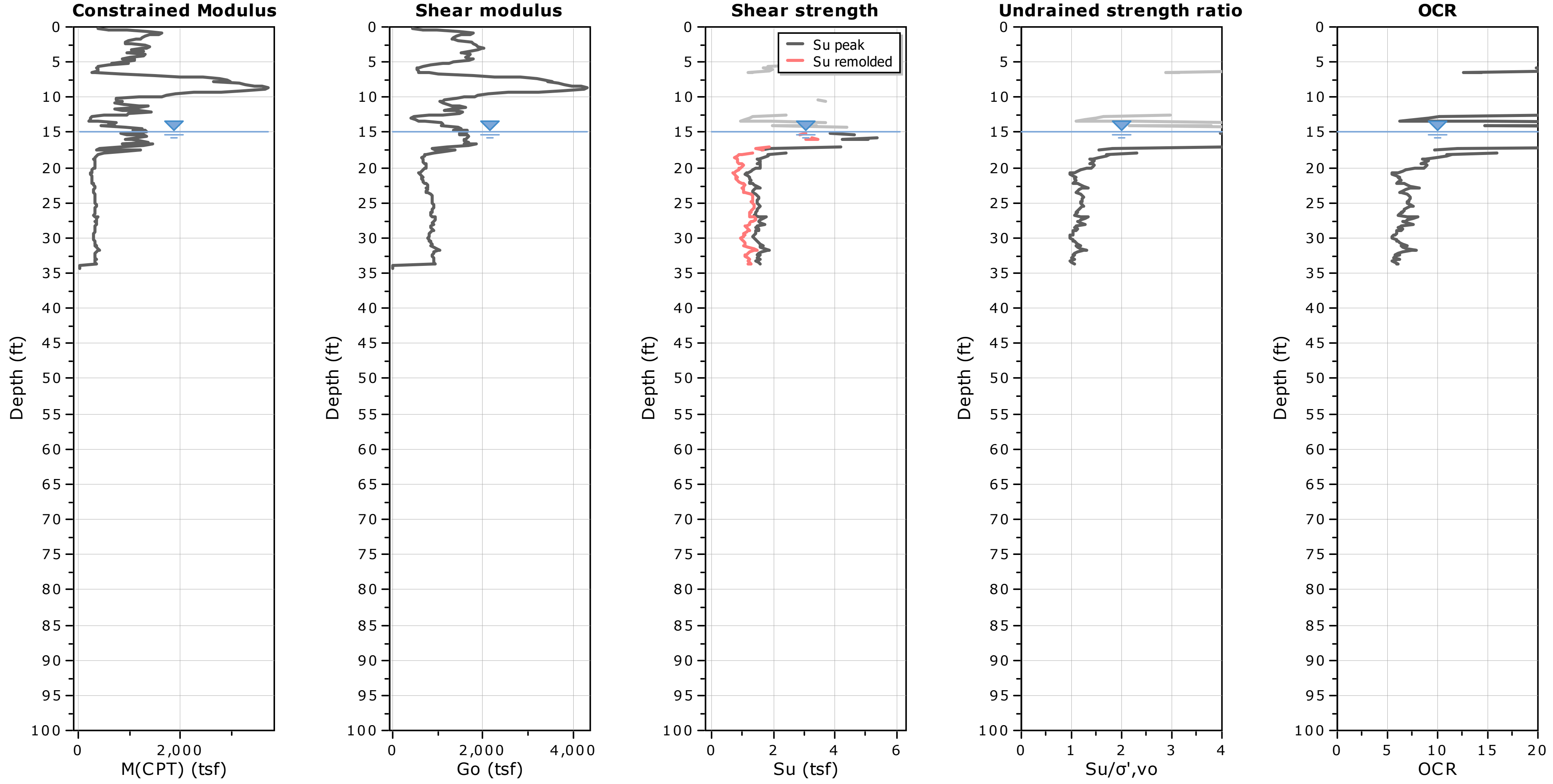




Calculation parameters

Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young’s modulus: Based on variable alpha using I_c (Robertson, 2009)

Relative density constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)



Calculation parameters

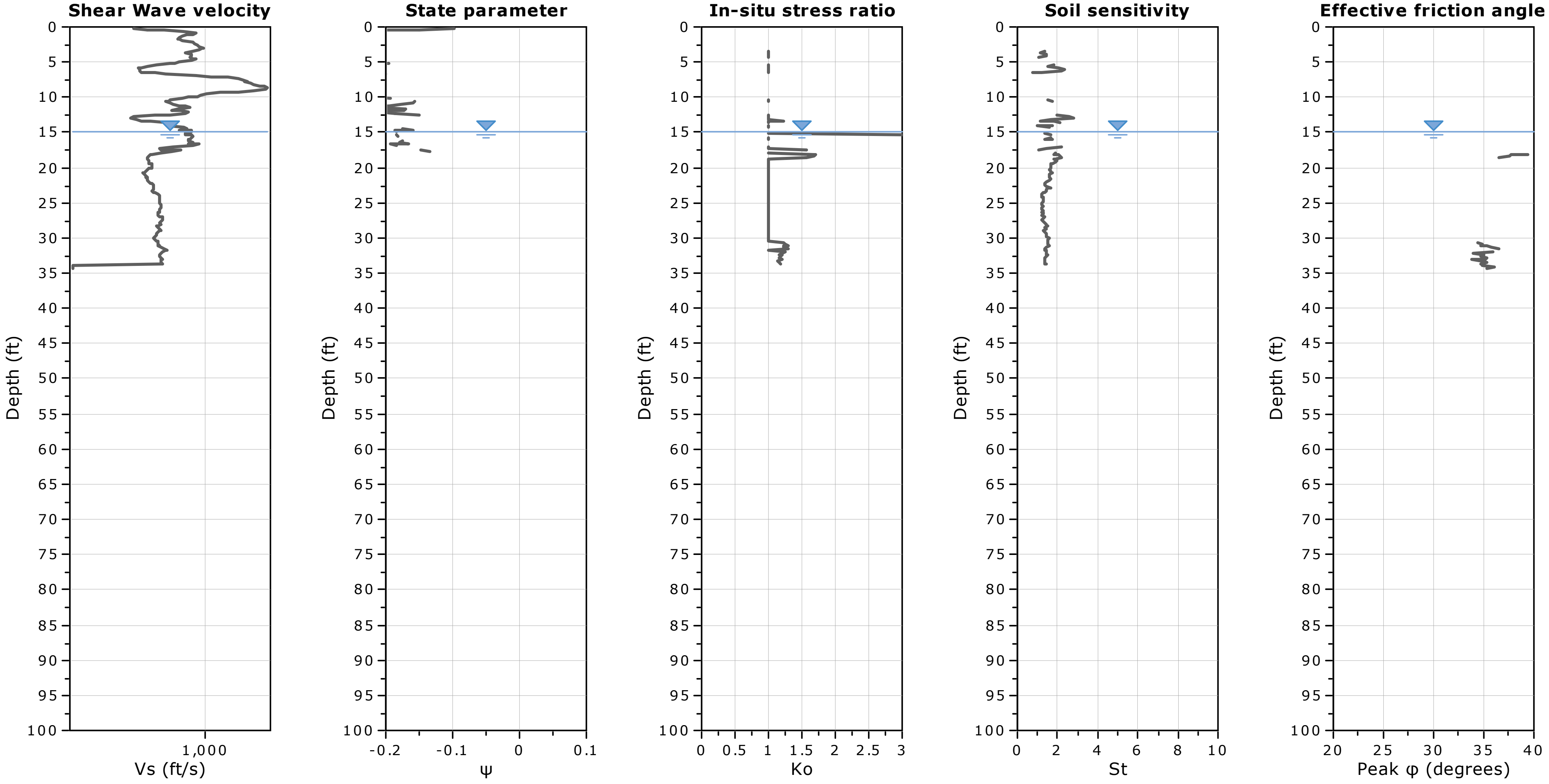
Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)

Go: Based on variable *alpha* using I_c (Robertson, 2009)

Undrained shear strength cone factor for clays, N_{kt}: Auto

OCR factor for clays, N_{kt}: Auto

● Flat Dilatometer Test data



Calculation parameters

Soil Sensitivity factor, N_S: 7.00

CPT: CPT-04

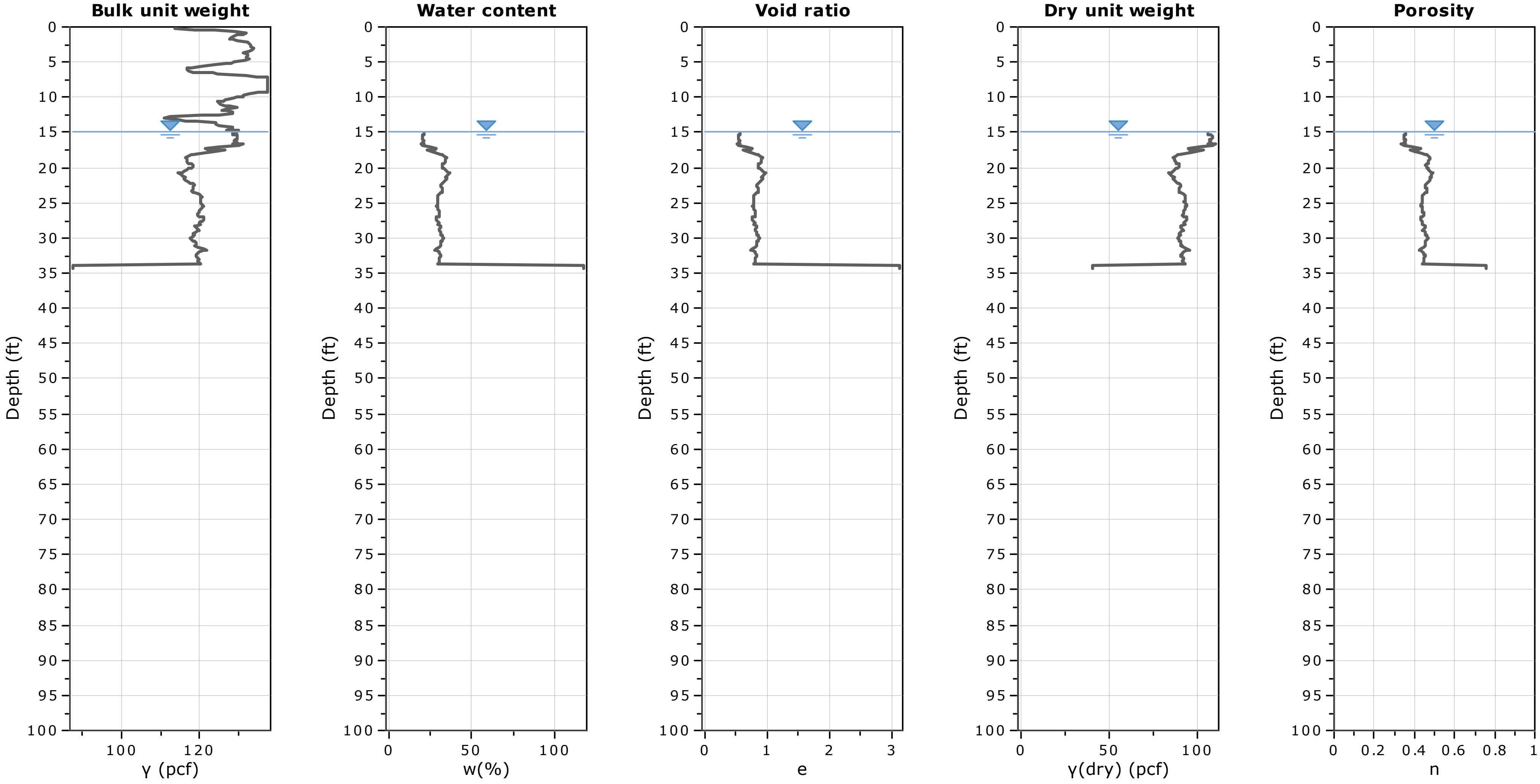
Total depth: 34.28 ft, Date: 11/1/2024

Cone Type: 15cm²

Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**

Location: **90 Minto Road, Watsonville, CA 95076**



Project: **New Leaf Minto Road Watsonville**

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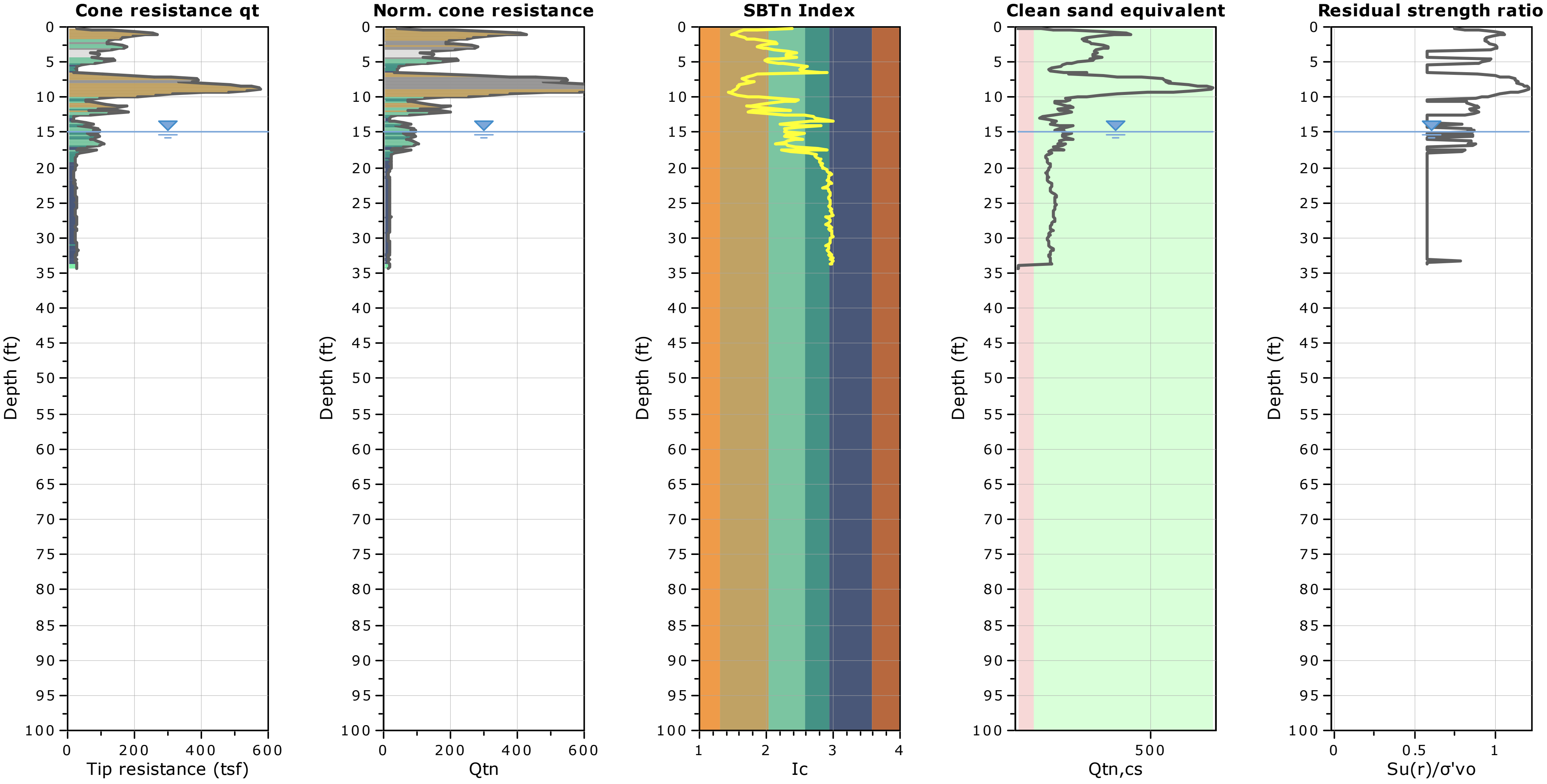
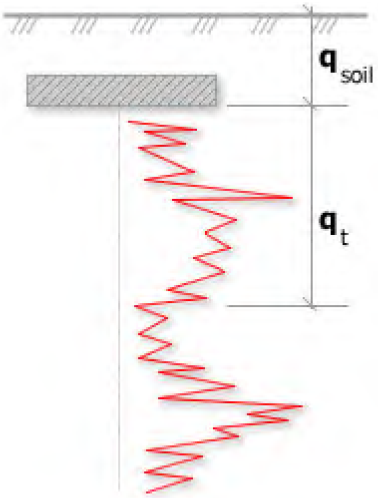
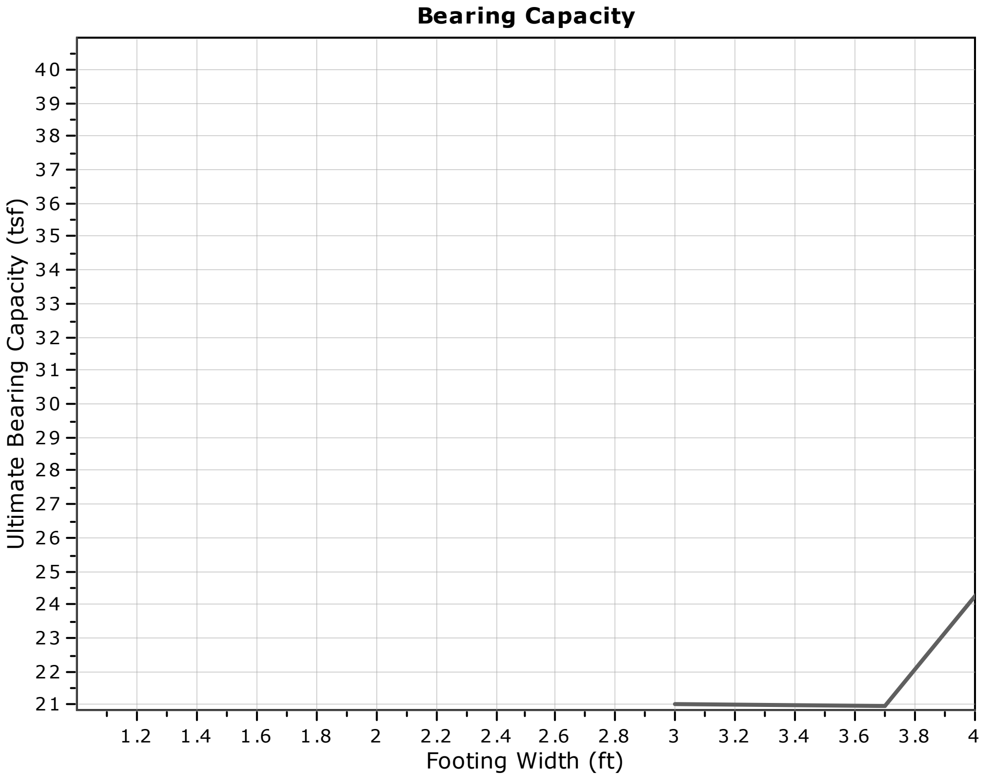




Bearing Capacity calculation is perfromed based on the formula:

$$Q_{ult} = R_k \times q_t + q_{soil}$$

where:
R_k: Bearing capacity factor
q_t: Average corrected cone resistance over calculation depth
q_{soil}: Pressure applied by soil above footing



:: Tabular results ::

No	B (ft)	Start Depth (ft)	End Depth (ft)	Ave. q_t (tsf)	R_k	Soil Press. (tsf)	Ult. bearing cap. (tsf)
1	3.00	1.60	6.10	104.54	0.20	0.10	21.00
2	3.70	1.60	7.15	104.34	0.20	0.10	20.96
3	4.40	1.60	8.20	142.50	0.20	0.10	28.60
4	5.10	1.60	9.25	199.23	0.20	0.10	39.94
5	5.80	1.60	10.30	203.68	0.20	0.10	40.83
6	6.50	1.60	11.35	191.78	0.20	0.10	38.45
7	7.20	1.60	12.40	185.83	0.20	0.10	37.26
8	7.90	1.60	13.45	172.93	0.20	0.10	34.68
9	8.60	1.60	14.50	162.77	0.20	0.10	32.65
10	9.30	1.60	15.55	157.14	0.20	0.10	31.52
11	10.00	1.60	16.60	151.94	0.20	0.10	30.48
12	10.70	1.60	17.65	146.82	0.20	0.10	29.46
13	11.40	1.60	18.70	140.38	0.20	0.10	28.17
14	12.10	1.60	19.75	133.10	0.20	0.10	26.72
15	12.80	1.60	20.80	127.45	0.20	0.10	25.59

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m^3) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236\right)$$

where g_w = water unit weight

:: Permeability, k (m/s) ::

$I_c < 3.27$ and $I_c > 1.00$ then $k = 10^{0.952\text{-}3.04 \cdot I_c}$

$I_c \leq 4.00$ and $I_c > 3.27$ then $k = 10^{-4.52\text{-}1.37 \cdot I_c}$

:: N$_{SPT}$ (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT$_n$: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Drained Friction Angle, φ (°) ::

$$\varphi = \varphi'_{cv} + 15.94 \cdot \log(Q_{tn,cs}) - 26.88$$

(applicable only to SBT$_n$: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$
$\alpha = 14$ for $Q_{tn} > 14$
$\alpha = Q_{tn}$ for $Q_{tn} \leq 14$
$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$

If $I_c \geq 2.20$

$$M_{CPT} = 0.03 \cdot (q_t - \sigma_v) \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Small strain shear Modulus, Go (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Shear Wave Velocity, Vs (m/s) ::

$$V_s = \left(\frac{G_0}{\rho}\right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot + 7 \cdot \log(F_r))}\right]^{1.25} \text{ or user defined}$$

$$OCR = k_{OCR} \cdot Q_{tn}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_O = (1 - \sin\varphi') \cdot OCR^{\sin\varphi'}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Soil Sensitivity, S$_t$ ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Peak Friction Angle, φ' (°) ::

$$\varphi' = 29.5^\circ \cdot B_q^{0.121} \cdot \left(0.256 + 0.336 \cdot B_q + \log Q_t\right)$$

(applicable for $0.10 < B_q < 1.00$)

References

- Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 5th Edition, November 2012
- Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)
- N Barounis, J Philpot, Estimation of in-situ water content, void ratio, dry unit weight and porosity using CPT for saturated sands, Proc. 20th NZGS Geotechnical Symposium

Middle Earth GEO TESTING INC.

Haley & Aldrich Inc.

https://www.haleyaldrich.com/

CPT: CPT-04

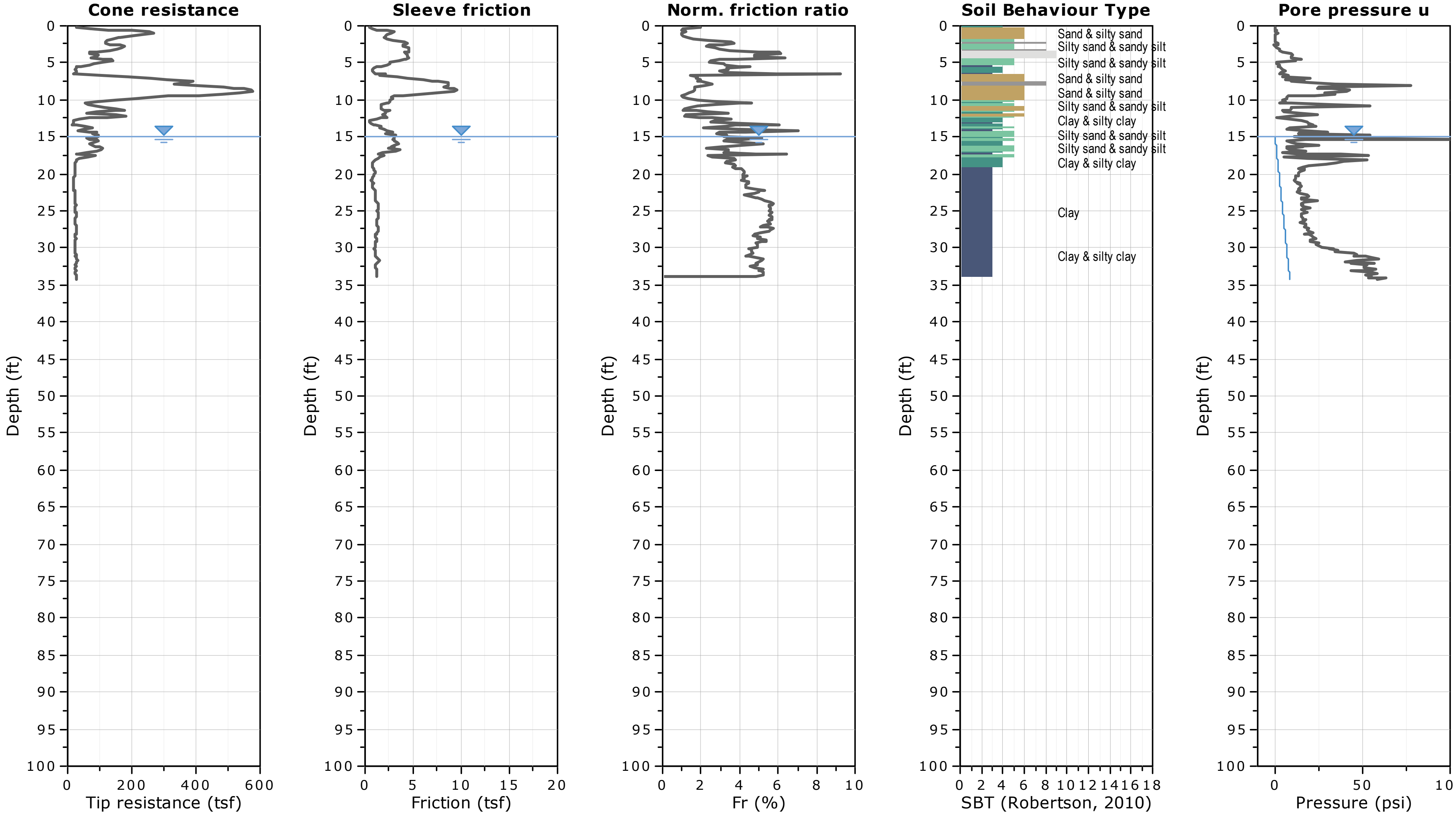
Total depth: 34.28 ft, Date: 11/1/2024

Cone Type: 15cm²

Cone Operator: JM-IY

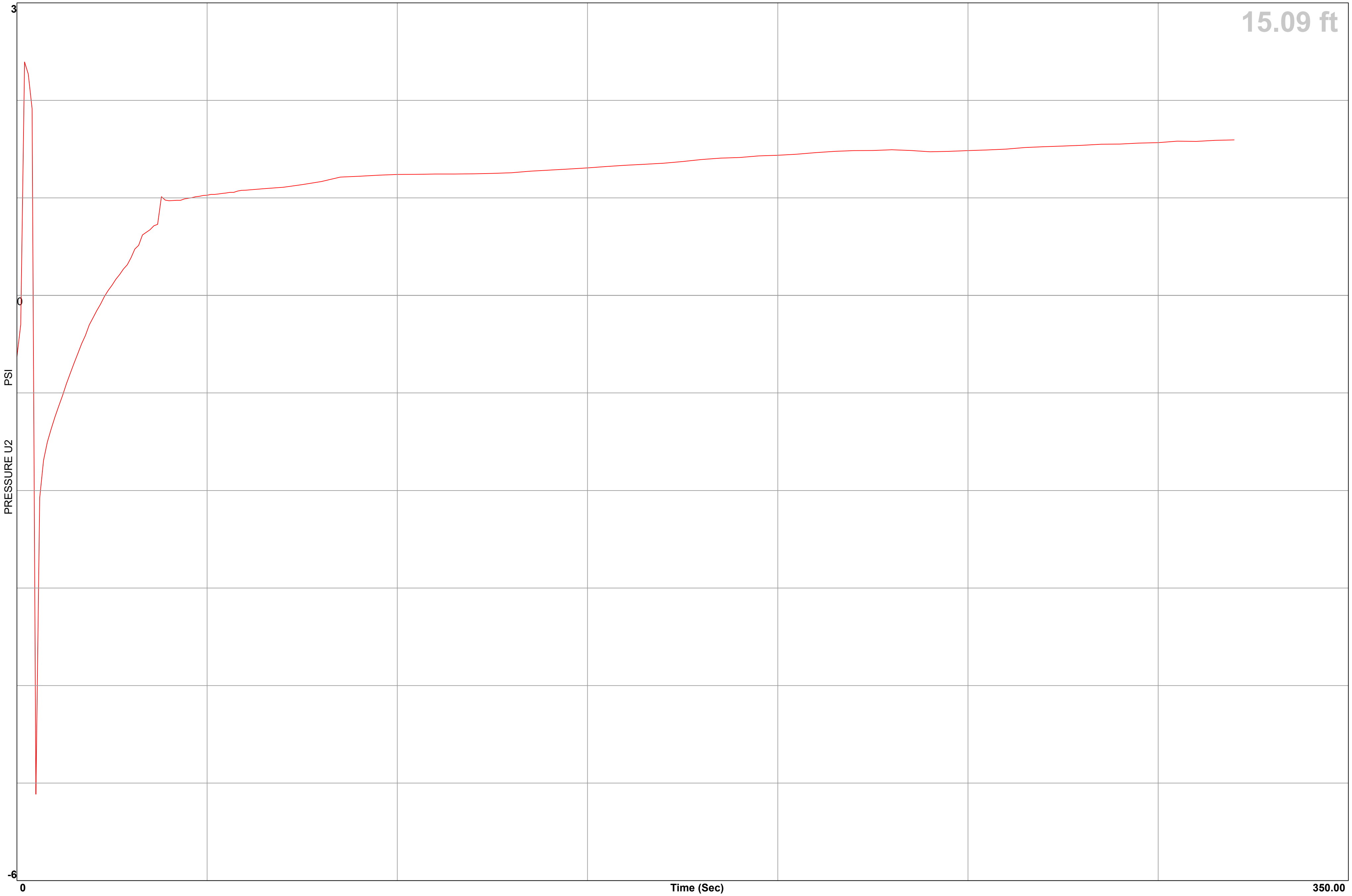
Project: New Leaf Minto Road Watsonville

Location: 90 Minto Road, Watsonville, CA 95076



Haley & Aldrich Inc.

Location	New Leaf Minto Road Watsonville	**Operator**	JM-IY		
Job Number	021059-003	**Cone Number**	DDG1589	**GPS**	
Hole Number	CPT-05	**Date and Time**	11/1/2024 1:43:15 PM		
Equilized Pressure	1.5	**EST GW Depth During Test**	11.4		



CPT: CPT-05

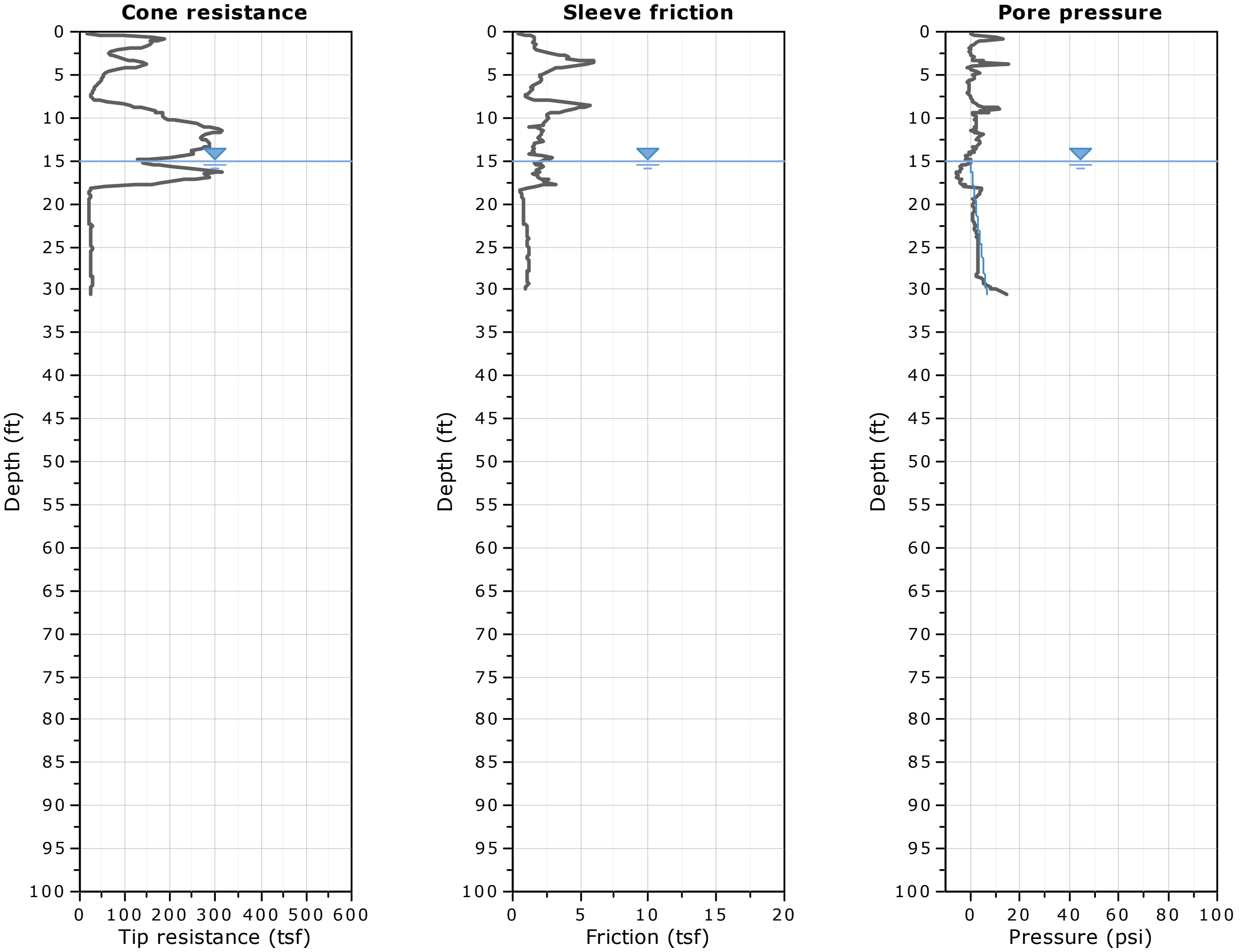
Total depth: 30.51 ft, Date: 11/1/2024

Cone Type: 15cm²

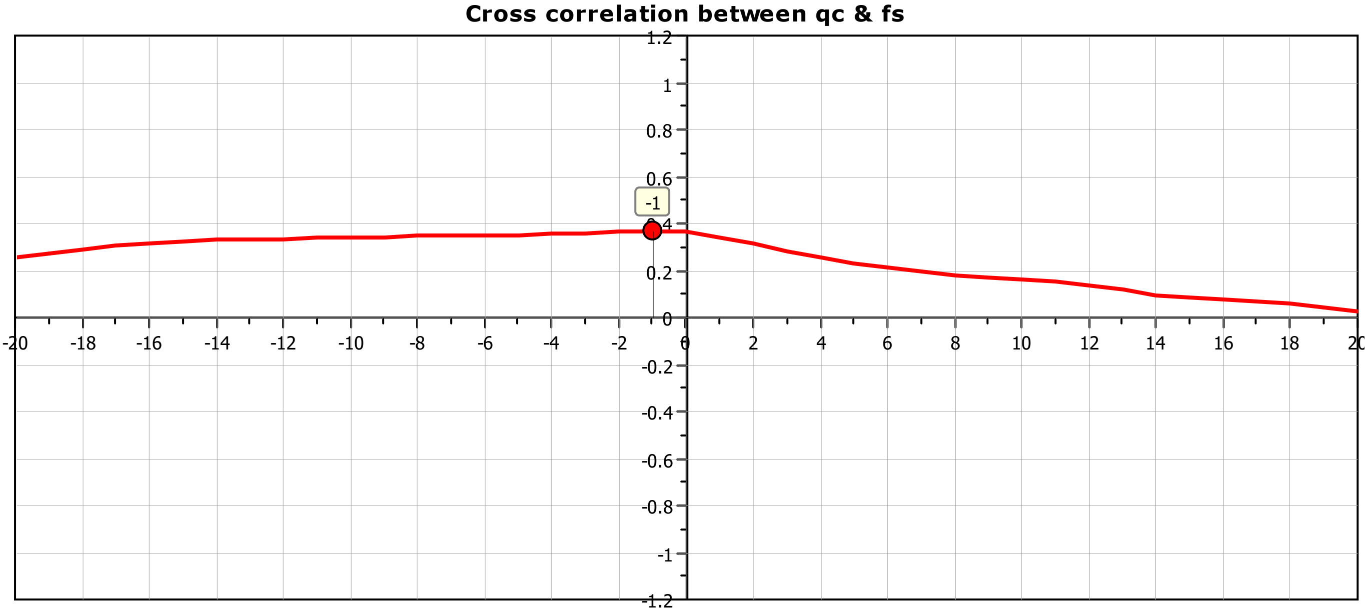
Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**

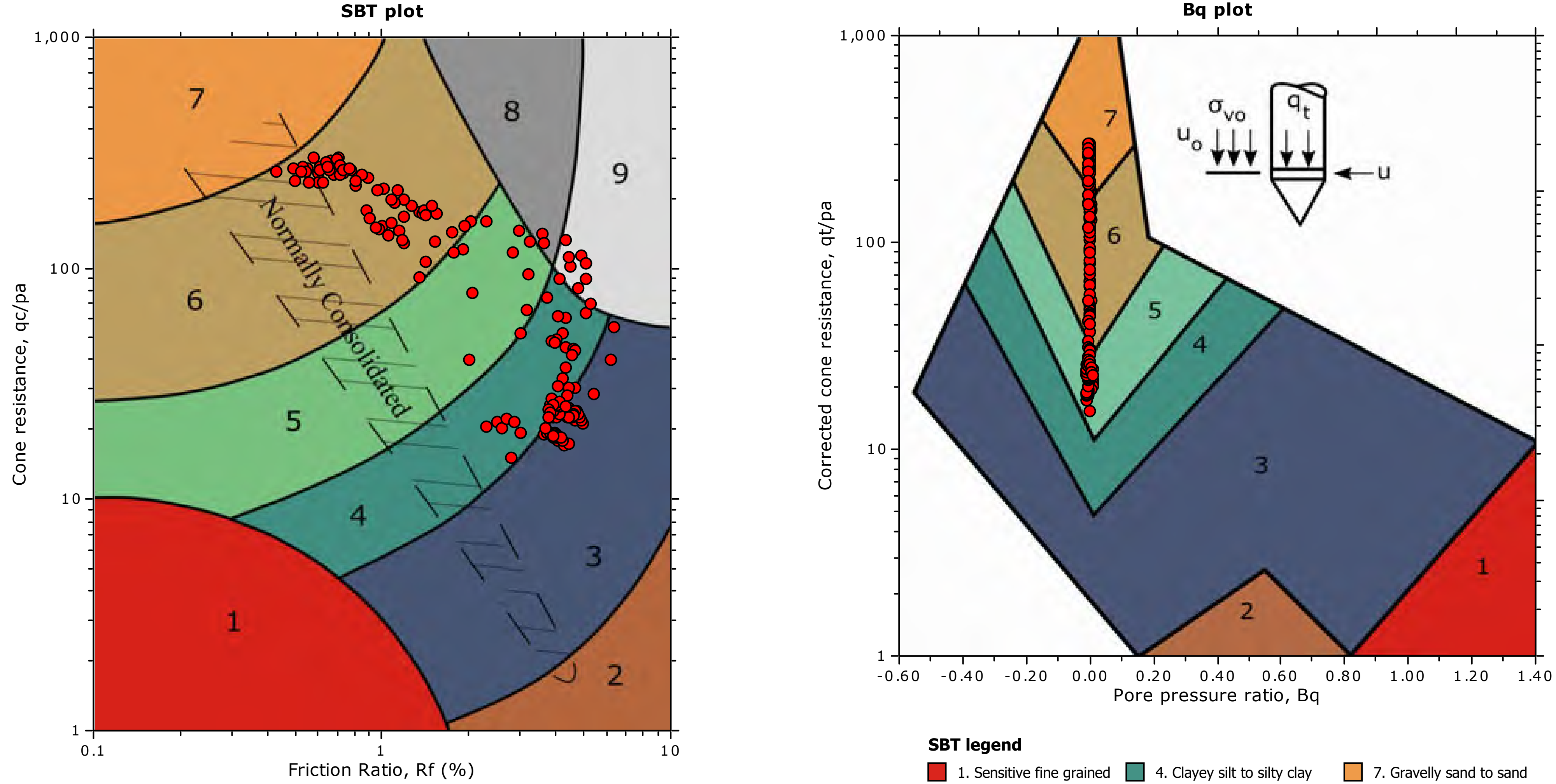
Location: **90 Minto Road, Watsonville, CA 95076**



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



SBT - Bq plots



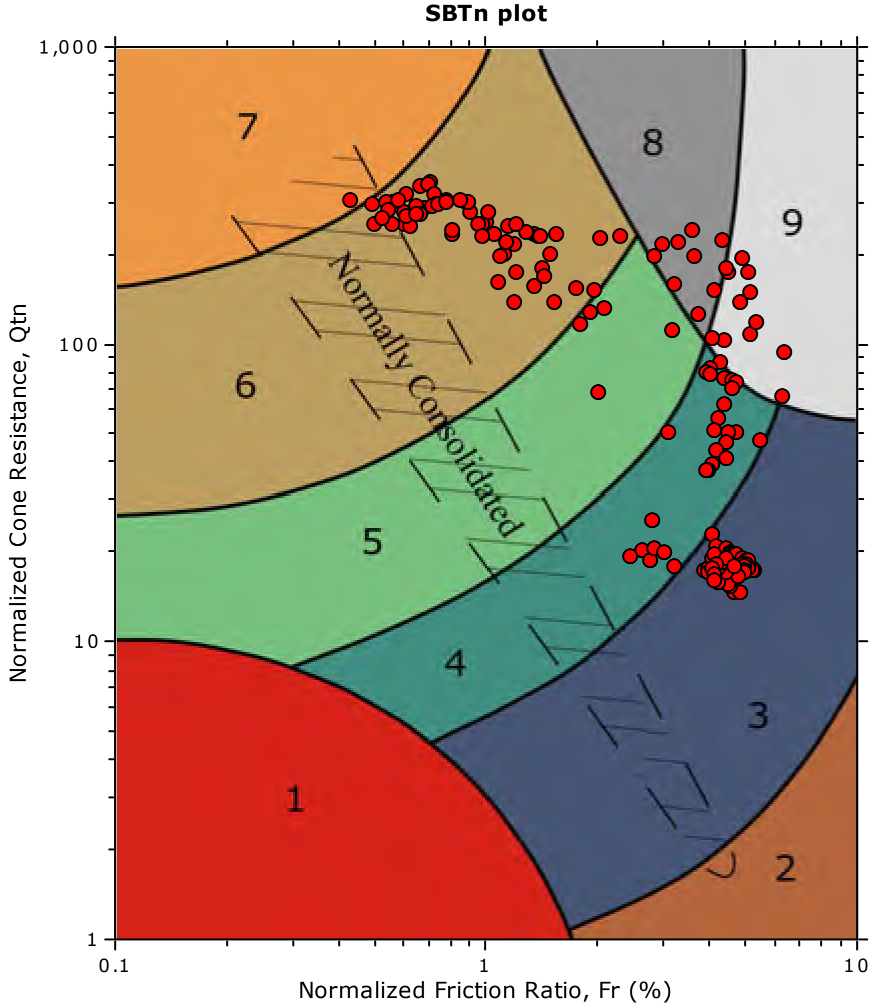
SBT legend

🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟫 6. Clean sand to silty sand
🟧 7. Gravelly sand to sand
⬛ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

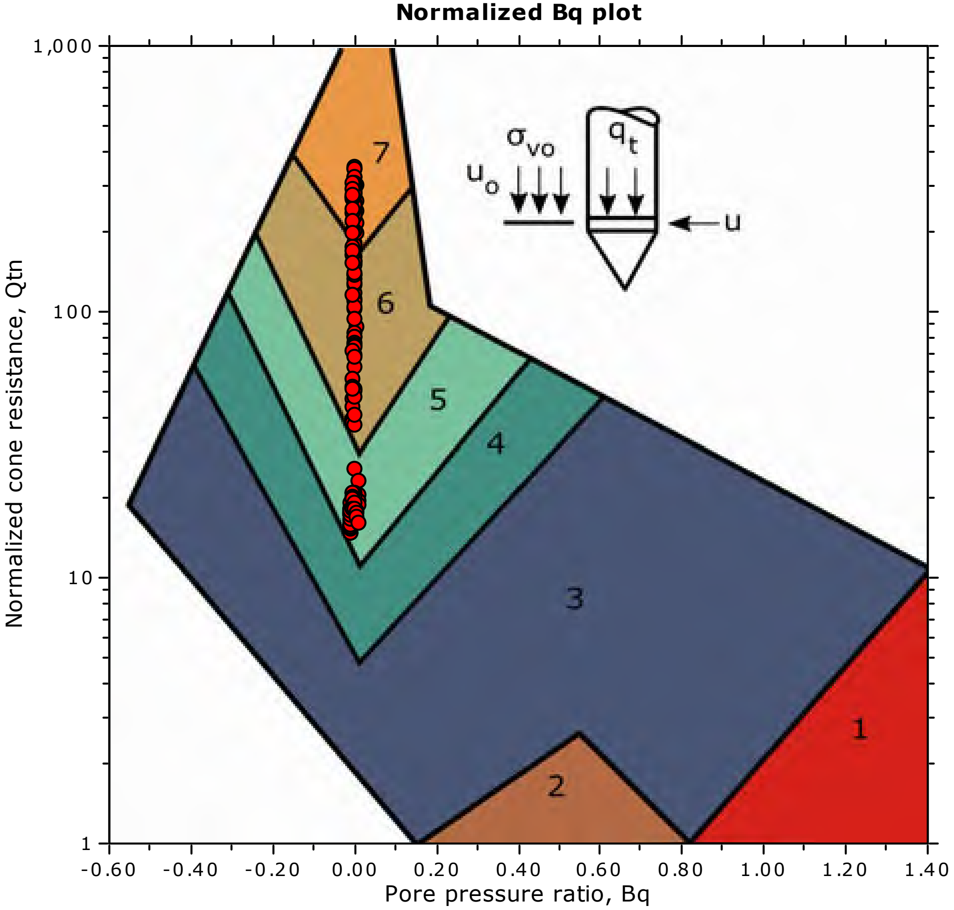
CPT: CPT-05
Total depth: 30.51 ft, Date: 11/1/2024
Cone Type: 15cm²
Cone Operator: JM-IY

SBT - Bq plots (normalized)

SBTn plot



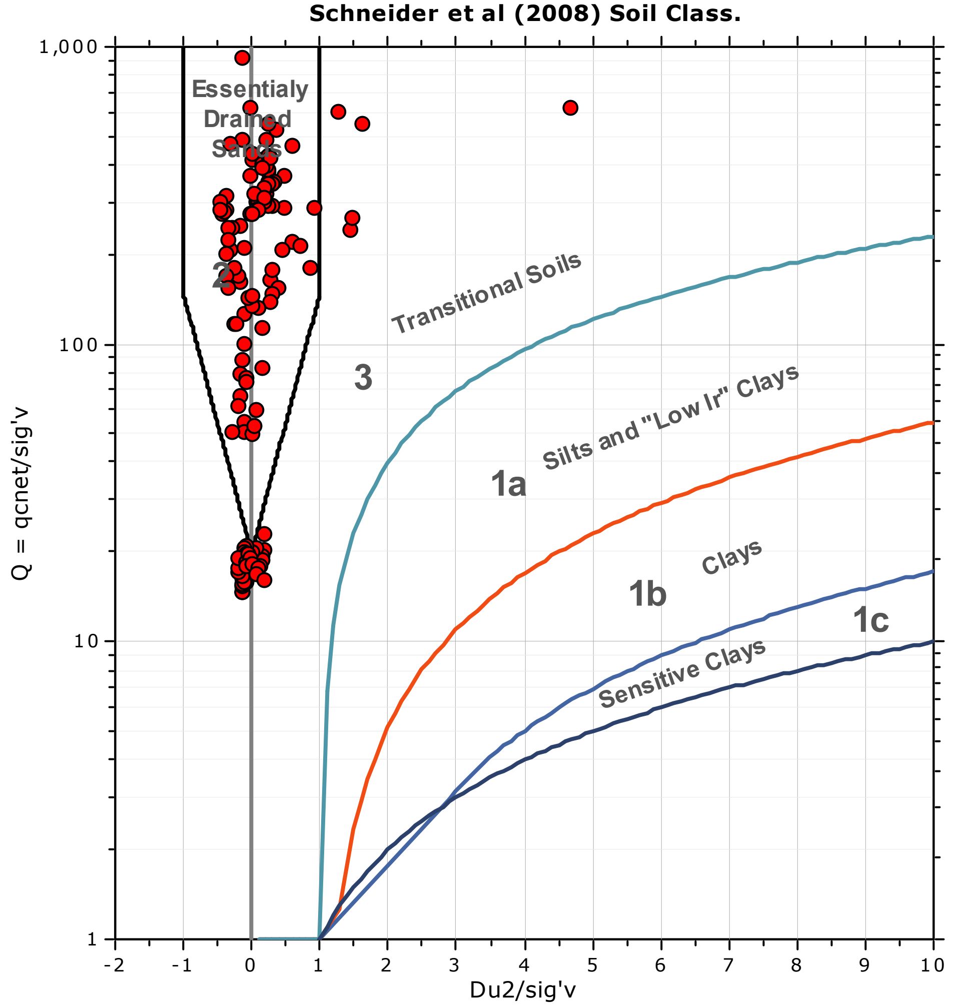
Normalized Bq plot

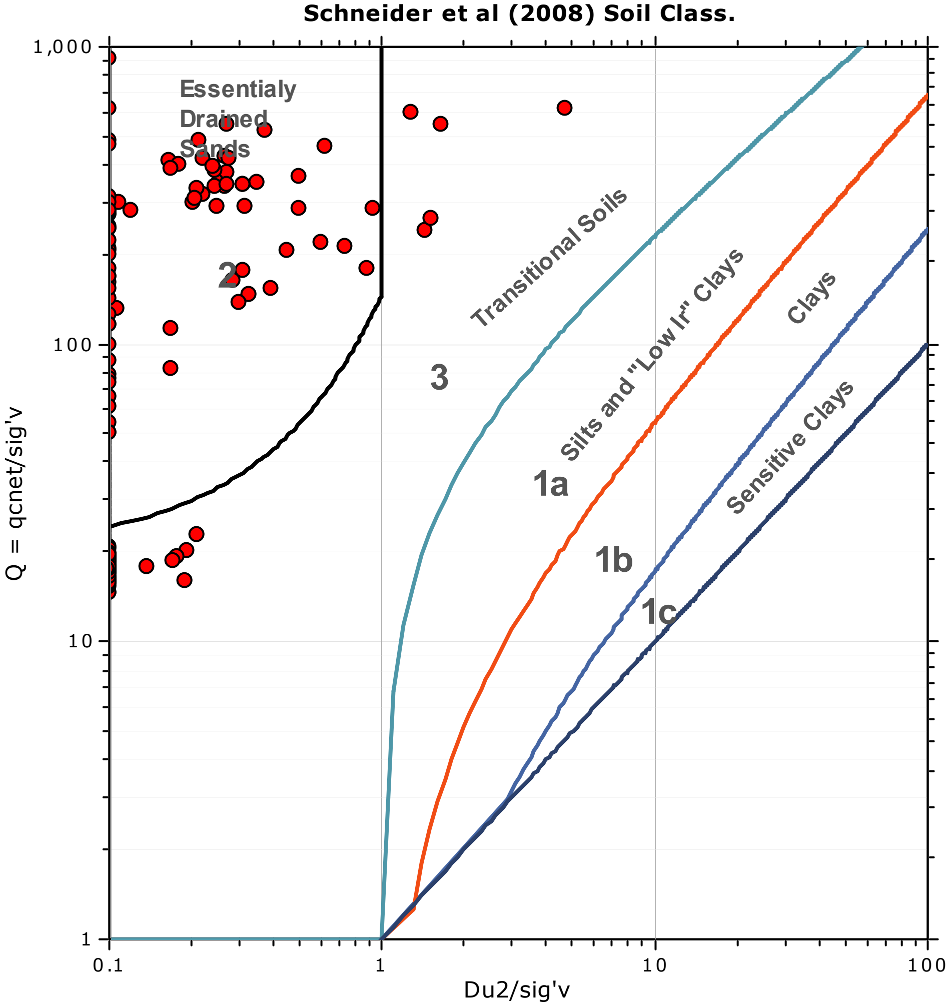


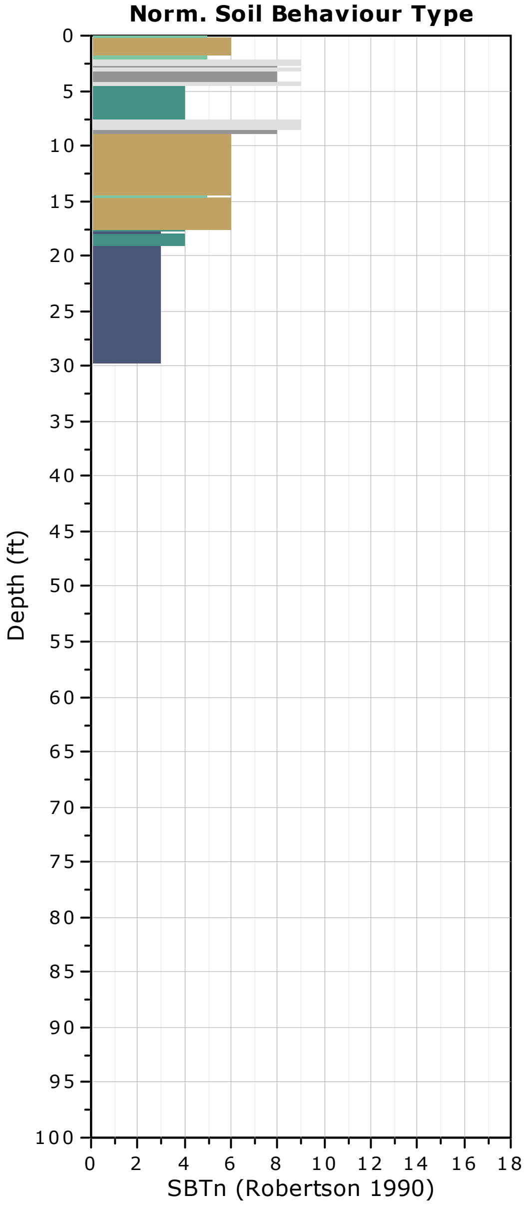
SBTn legend

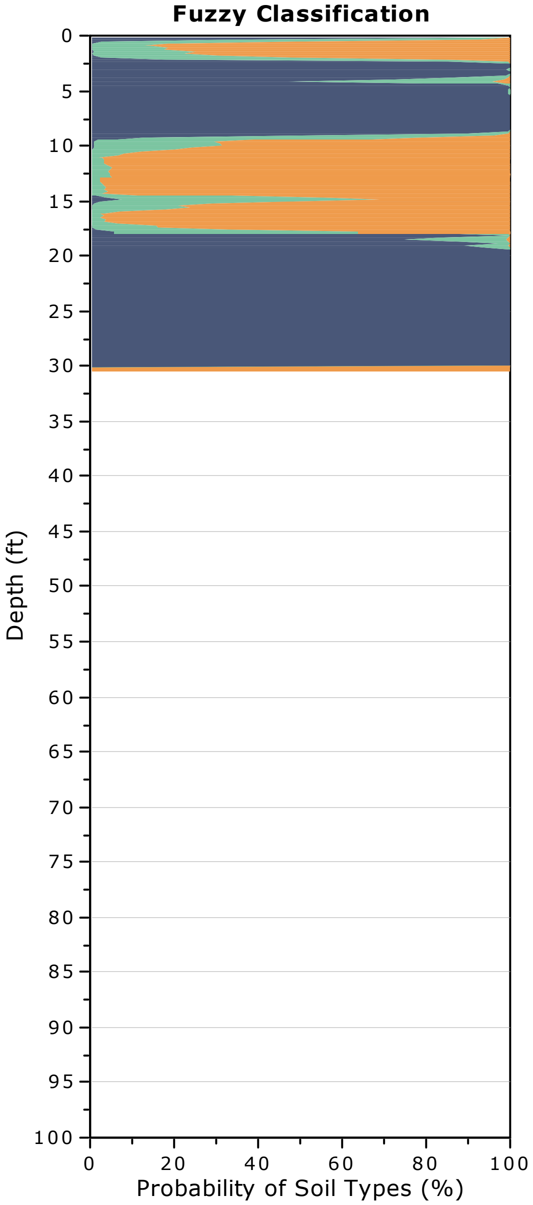
- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravelly sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

Bq plots (Schneider)









Fuzzy classifiaction legend

- ■ Highly probable clayey soil
- 🟩 Highly probable mixture soil
- 🟧 Highly probable sandy soil

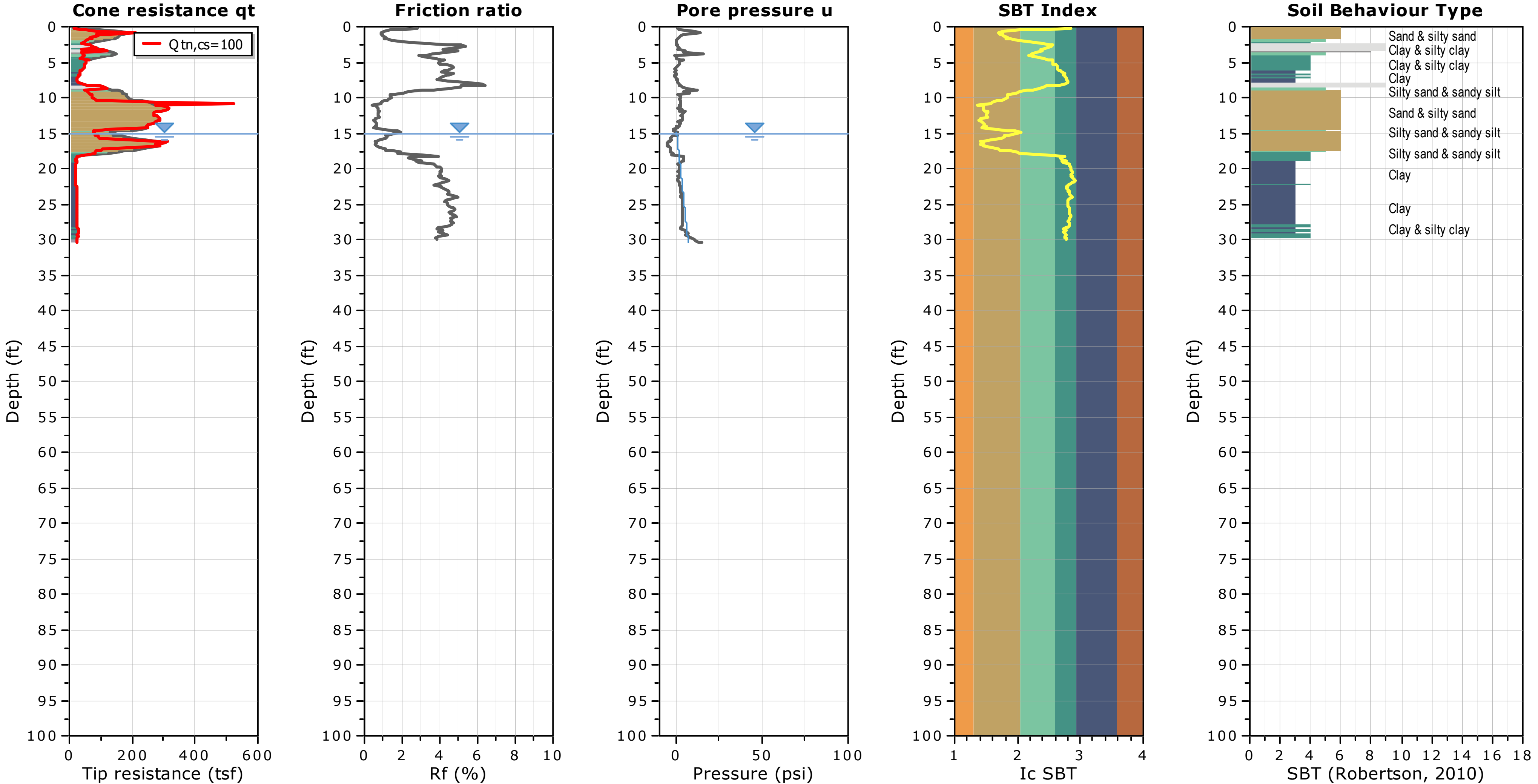
Haley & Aldrich Inc.

https://www.haleyaldrich.com/

CPT: CPT-05

Total depth: 30.51 ft, Date: 11/1/2024
Cone Type: 15cm²
Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**
Location: **90 Minto Road, Watsonville, CA 95076**



SBT legend

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

CPT: CPT-05

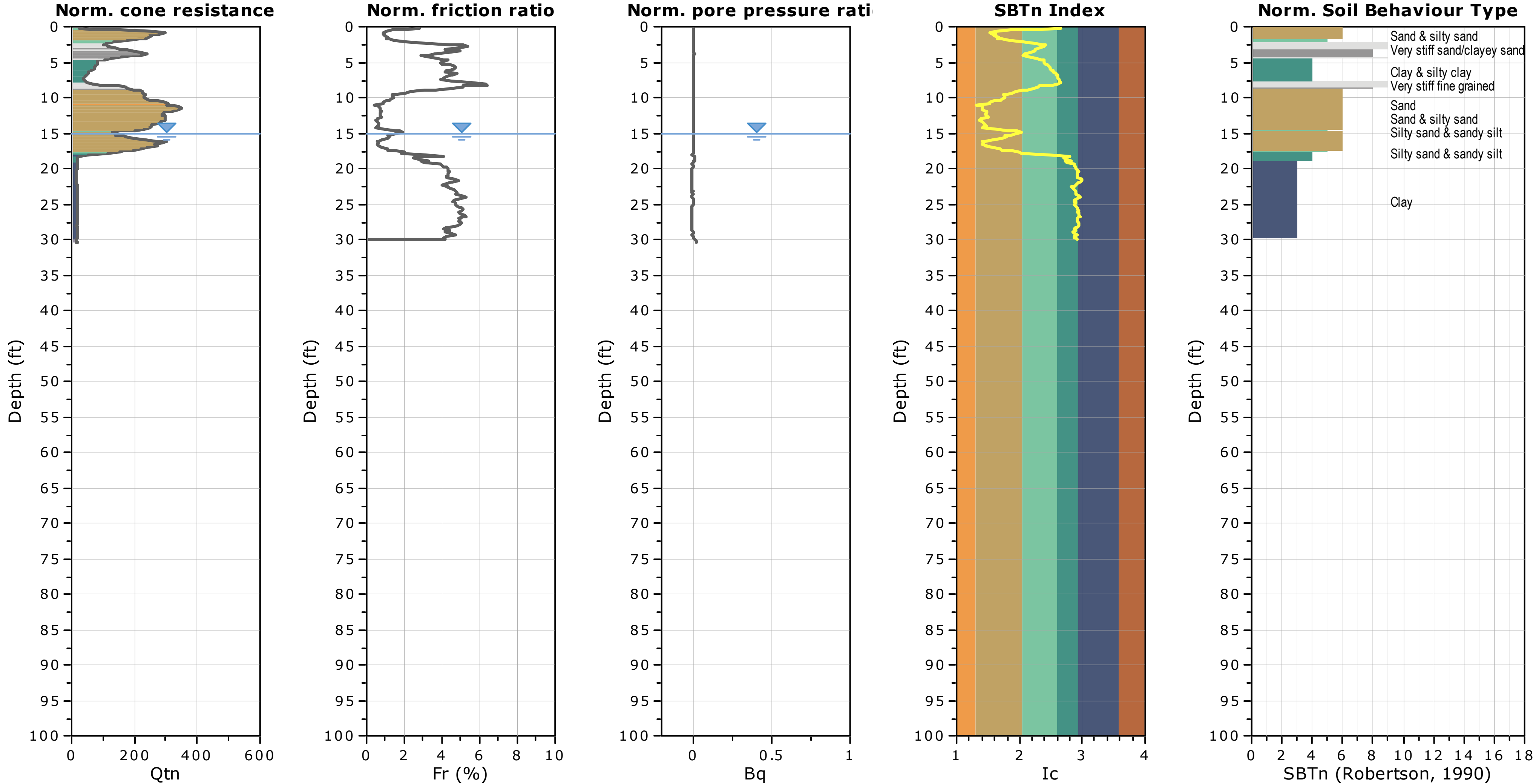
Total depth: 30.51 ft, Date: 11/1/2024

Cone Type: 15cm²

Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**

Location: **90 Minto Road, Watsonville, CA 95076**



SBTn legend

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CPT: CPT-05

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Cone Type: 15cm²

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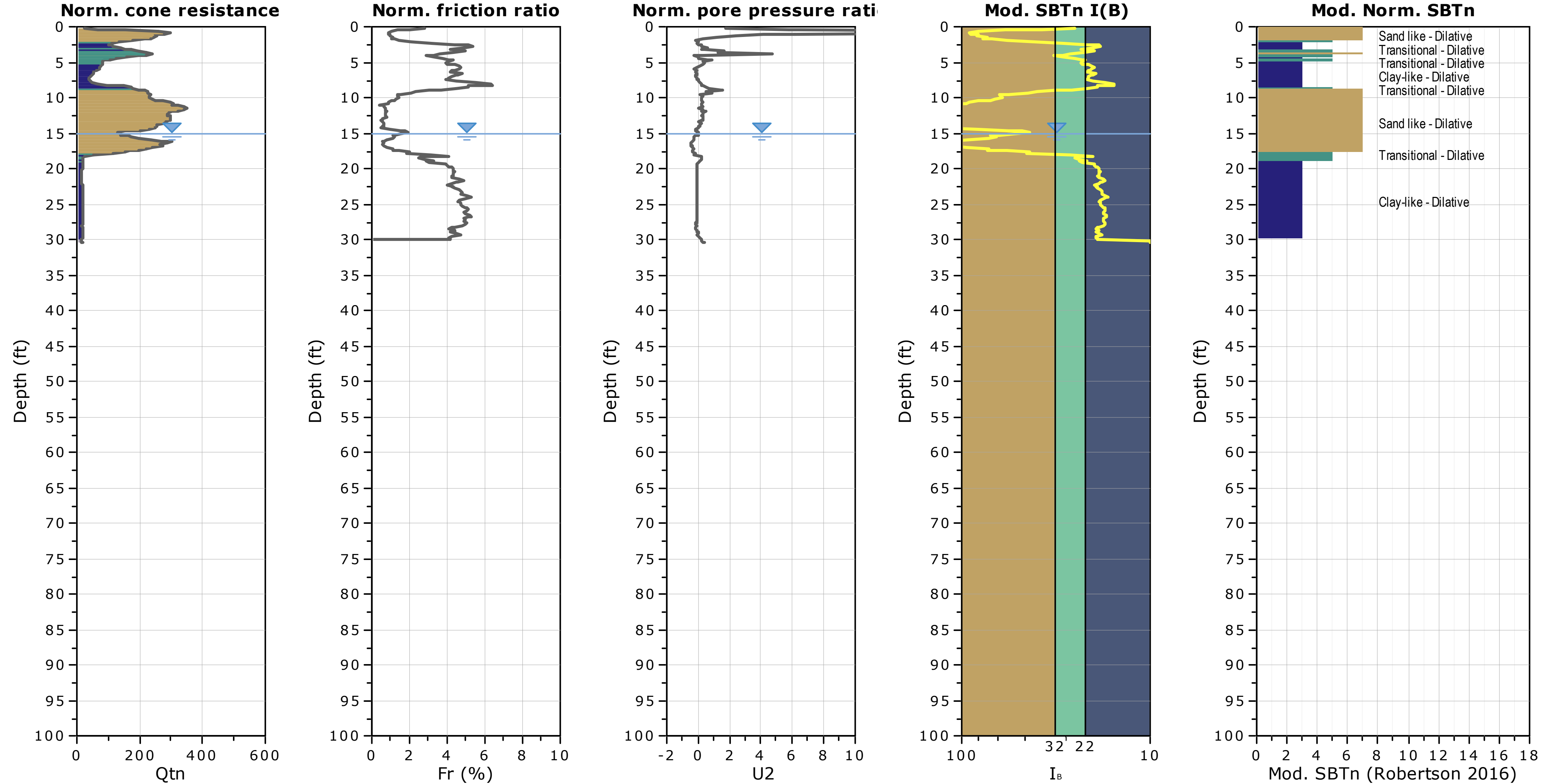
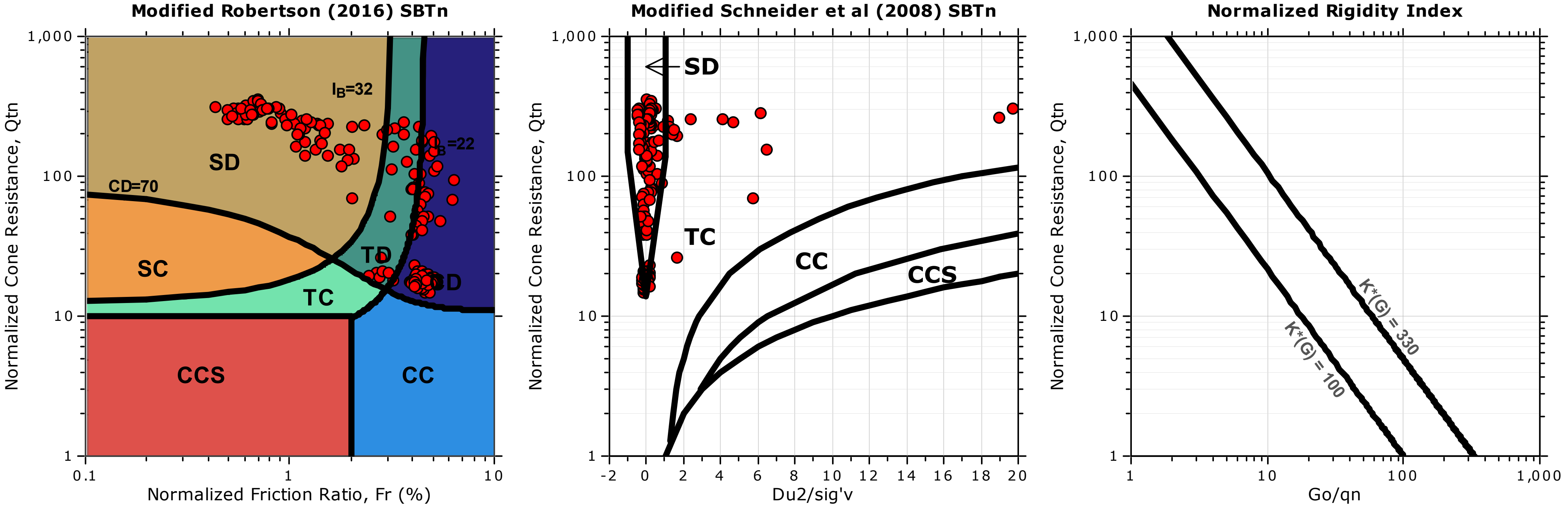


Mod. SBTn legend

- 🟥 1. CCS: ClayLike - Contractive, Sensitive
- 🟦 2. CC: Clay-like - Contractive
- 🟦 3. CD: Clay-Like: Dilative
- 🟩 4. TC: Transitional - Contractive
- 🟩 5. TD: Transitional - Dilative
- 🟧 6. SC: Sand-like - Contractive
- 🟫 7. SD: Sand-like - Dilative

Updated SBTn plots



CCS:	Clay-like - Contractive - Sensitive
CC:	Clay-like - Contractive
CD:	Clay-like - Dilative
TC:	Transitional - Contractive
TD:	Transitional - Dilative
SC:	Sand-like - Contractive
SD:	Sand-like - Dilative

K(G) > 330: Soils with significant microstructure (e.g. age/cementation)

Middle Earth
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CPT: CPT-05

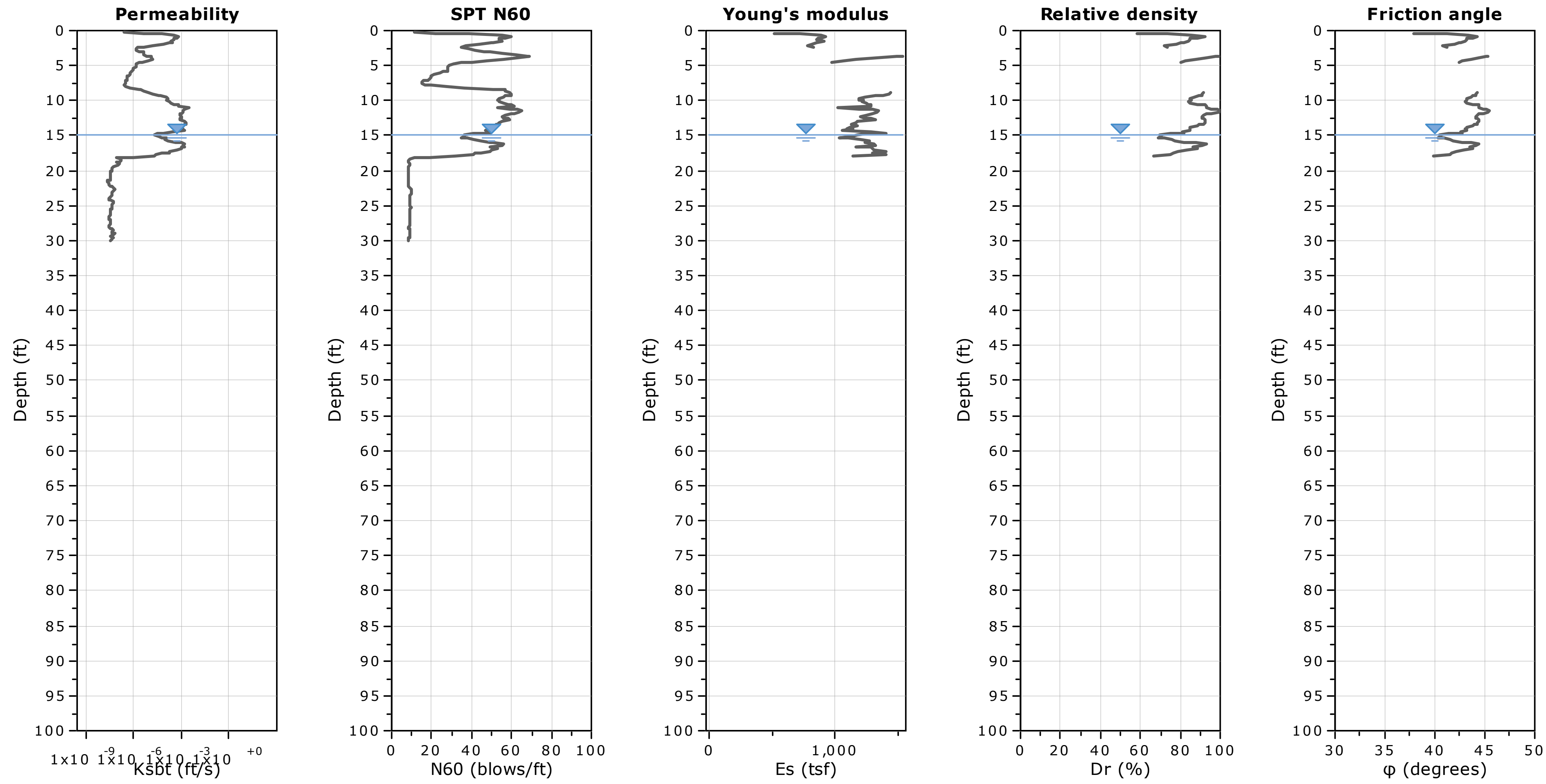
Total depth: 30.51 ft, Date: 11/1/2024

Cone Type: 15cm²

Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**

Location: **90 Minto Road, Watsonville, CA 95076**



Calculation parameters

Permeability: Based on SBT_n

SPT N_{60}: Based on I_c and q_t

Young's modulus: Based on variable alpha using I_c (Robertson, 2009)

Relative density constant, C_{Dr}: 350.0

Phi: Based on Kulhawy & Mayne (1990)

CPT: CPT-05

Total depth: 30.51 ft, Date: 11/1/2024

Cone Type: 15cm²

Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**

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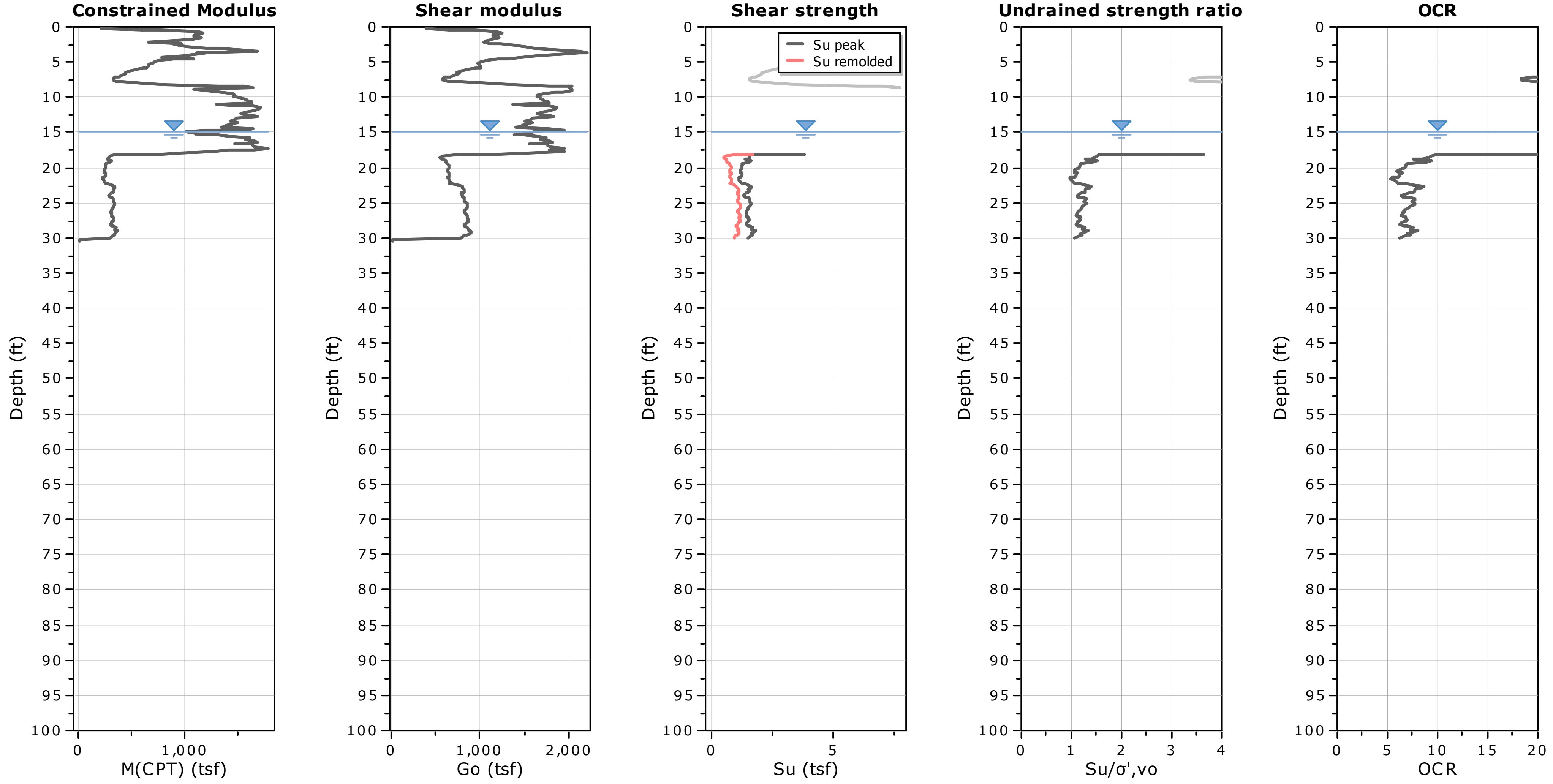

Calculation parameters

Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)

Go: Based on variable *alpha* using I_c (Robertson, 2009)

Undrained shear strength cone factor for clays, N_{kt}: Auto

OCR factor for clays, N_{kt}: Auto

● Flat Dilatometer Test data

CPT: CPT-05

Total depth: 30.51 ft, Date: 11/1/2024

Cone Type: 15cm²

Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**

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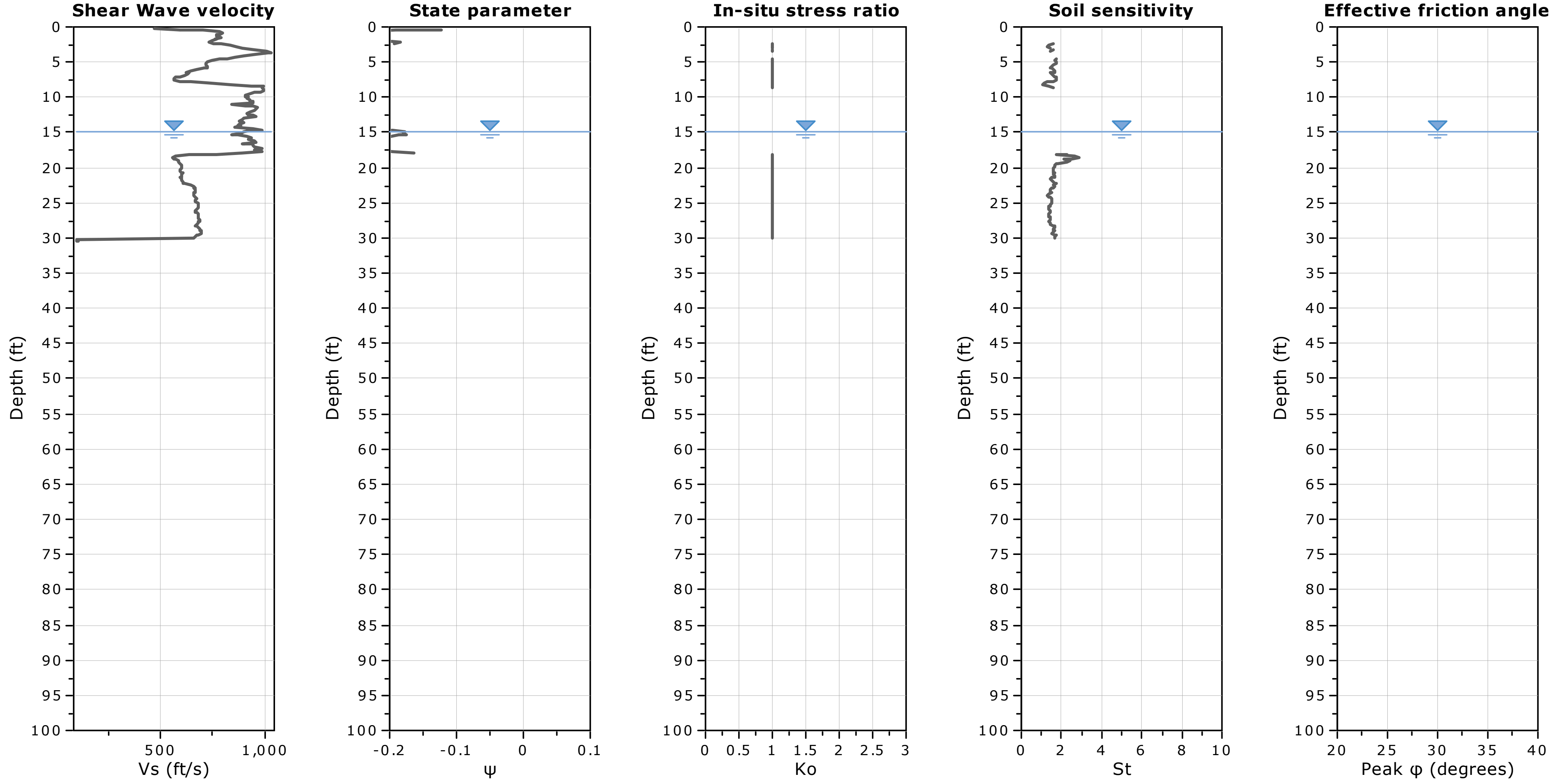

Calculation parameters

Soil Sensitivity factor, N_S: 7.00

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CPT: CPT-05

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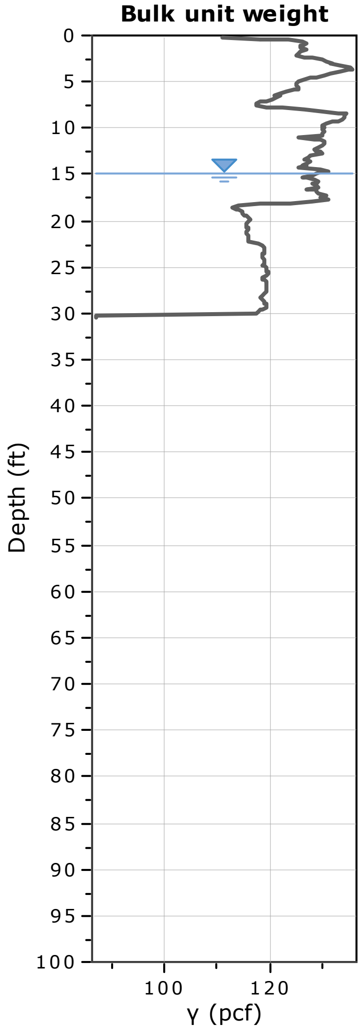

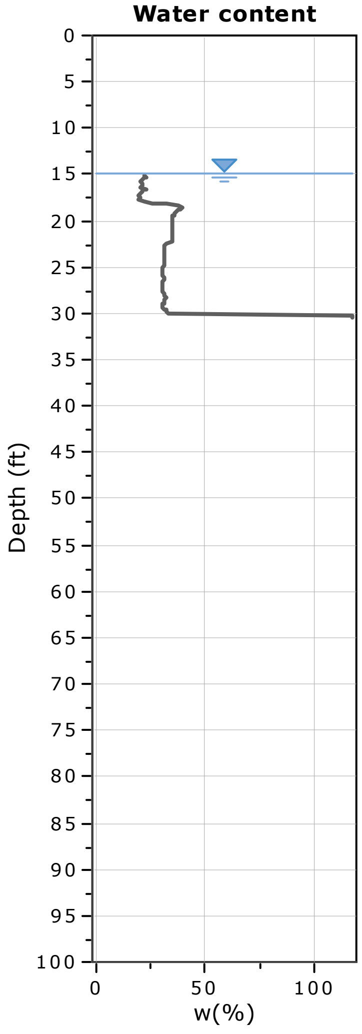

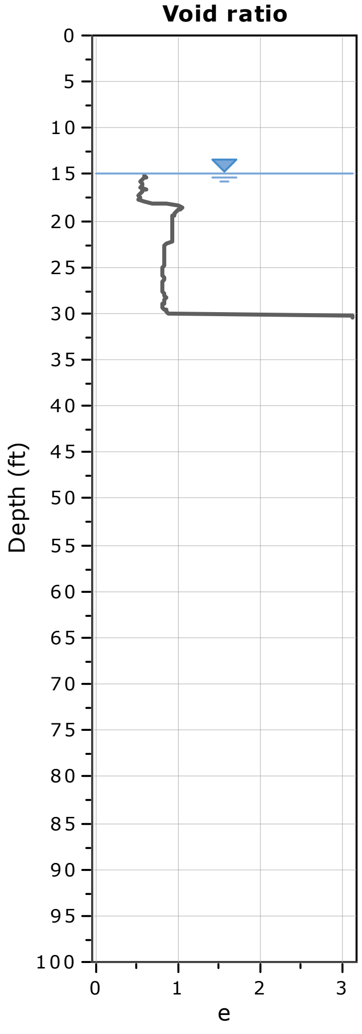

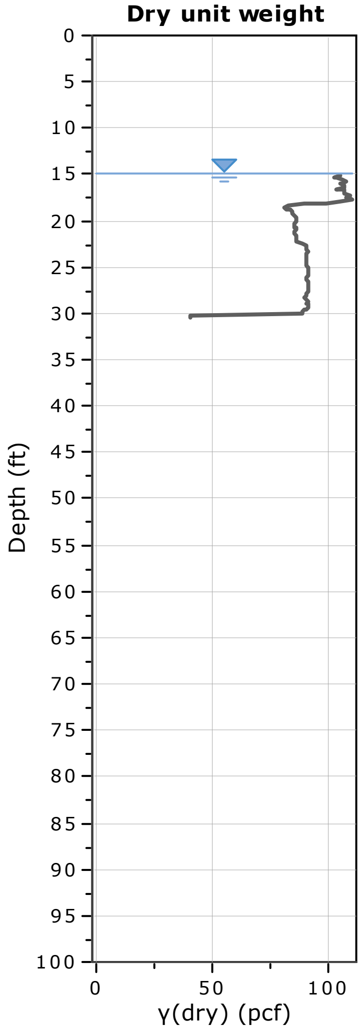

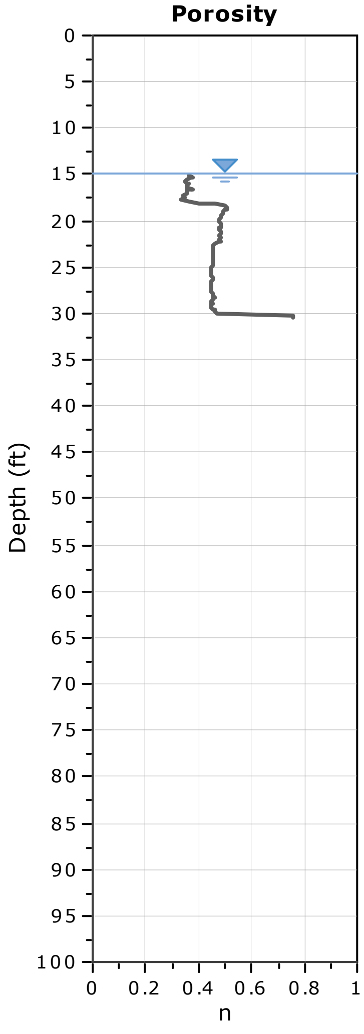

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CPT: CPT-05

Total depth: 30.51 ft, Date: 11/1/2024

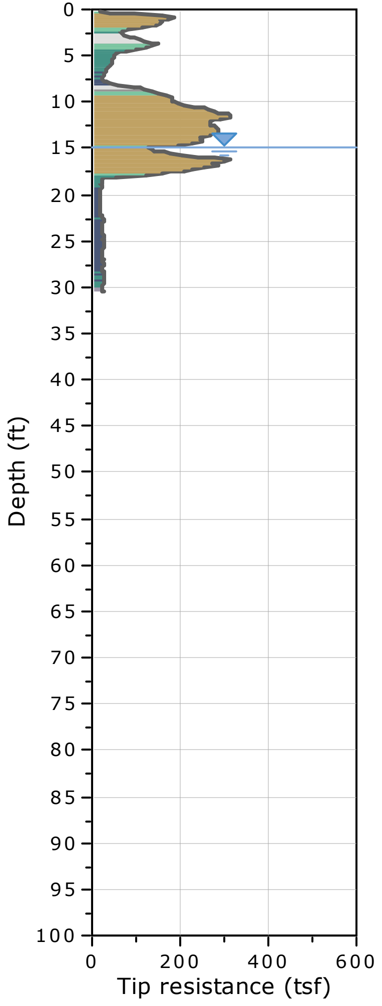
Cone Type: 15cm²

Cone Operator: JM-IY

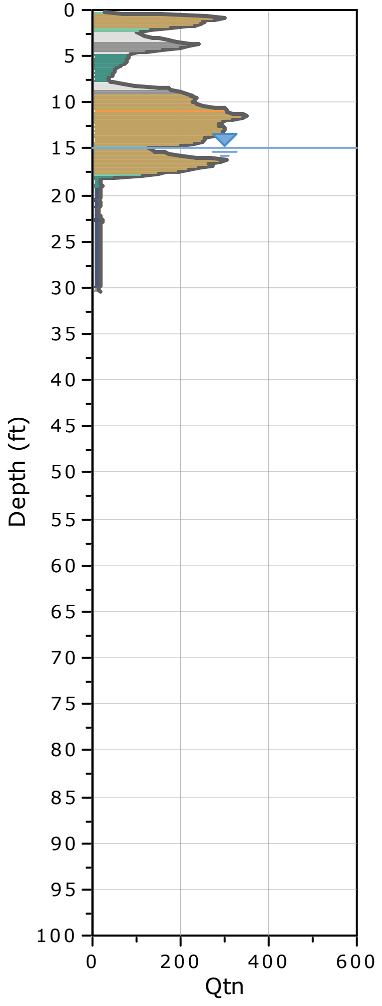
Project: **New Leaf Minto Road Watsonville**

Location: **90 Minto Road, Watsonville, CA 95076**

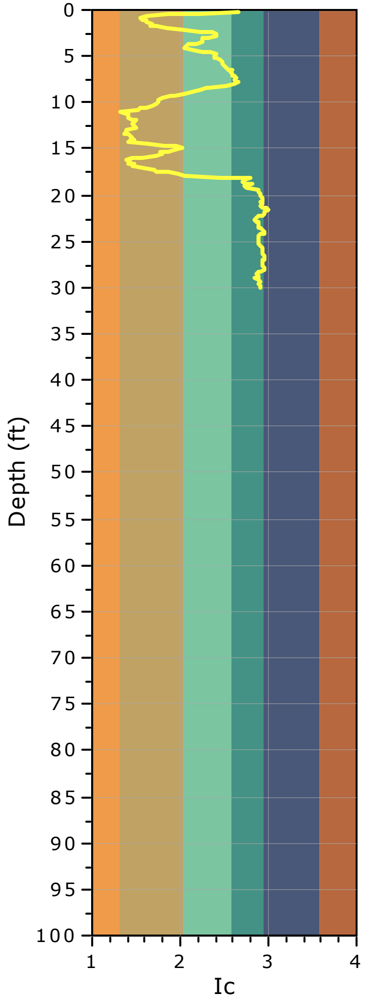
Cone resistance qt



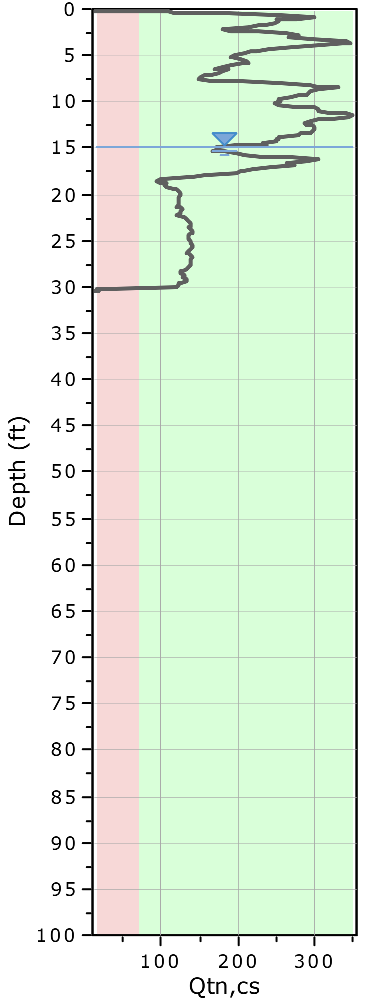
Norm. cone resistance



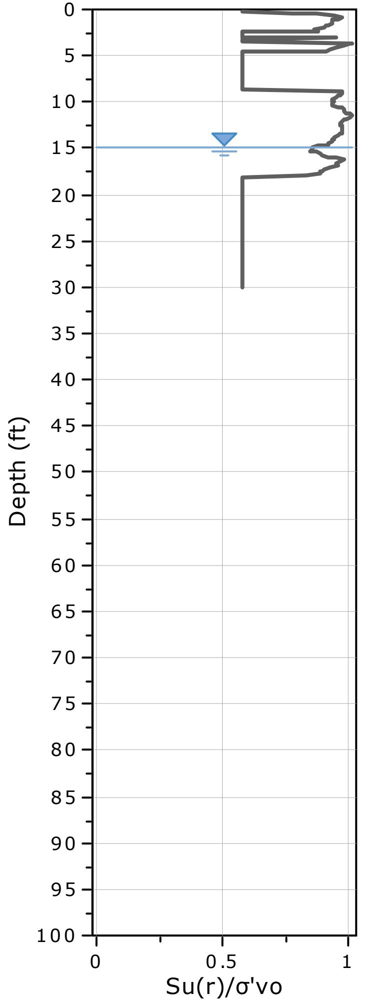
SBTn Index

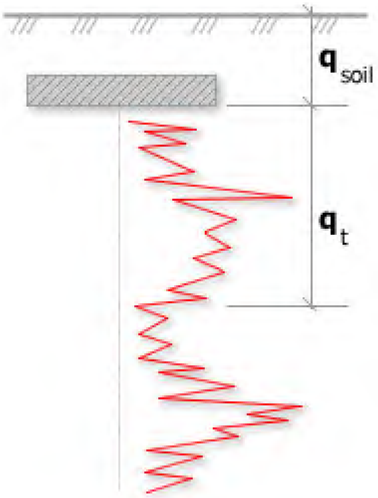


Clean sand equivalent



Residual strength ratio

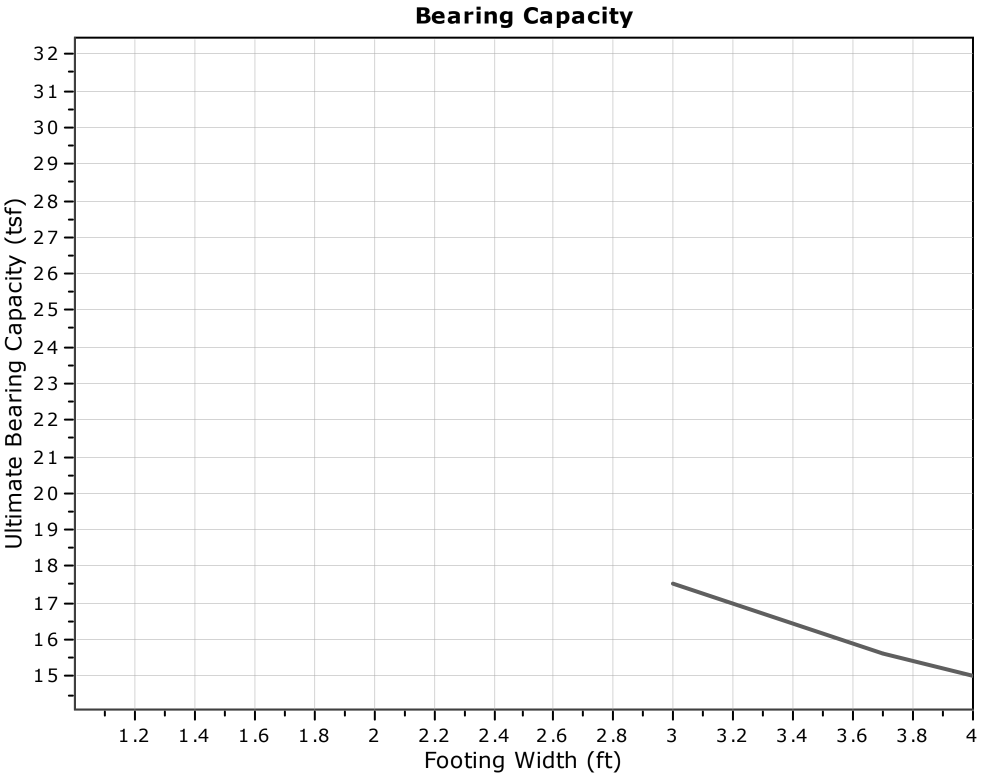




Bearing Capacity calculation is perfromed based on the formula:

$$Q_{ult} = R_k \times q_t + q_{soil}$$

where:
R_k: Bearing capacity factor
q_t: Average corrected cone resistance over calculation depth
q_{soil}: Pressure applied by soil above footing



:: Tabular results ::

No	B (ft)	Start Depth (ft)	End Depth (ft)	Ave. q_t (tsf)	R_k	Soil Press. (tsf)	Ult. bearing cap. (tsf)
1	3.00	1.60	6.10	87.23	0.20	0.10	17.54
2	3.70	1.60	7.15	77.66	0.20	0.10	15.63
3	4.40	1.60	8.20	70.46	0.20	0.10	14.19
4	5.10	1.60	9.25	77.59	0.20	0.10	15.61
5	5.80	1.60	10.30	89.25	0.20	0.10	17.95
6	6.50	1.60	11.35	109.03	0.20	0.10	21.90
7	7.20	1.60	12.40	125.16	0.20	0.10	25.13
8	7.90	1.60	13.45	137.95	0.20	0.10	27.69
9	8.60	1.60	14.50	147.92	0.20	0.10	29.68
10	9.30	1.60	15.55	148.25	0.20	0.10	29.75
11	10.00	1.60	16.60	156.29	0.20	0.10	31.35
12	10.70	1.60	17.65	161.29	0.20	0.10	32.35
13	11.40	1.60	18.70	155.96	0.20	0.10	31.29
14	12.10	1.60	19.75	147.54	0.20	0.10	29.60
15	12.80	1.60	20.80	141.04	0.20	0.10	28.30

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236\right)$$

where $g_w =$ water unit weight

:: Permeability, k (m/s) ::

$I_c < 3.27$ and $I_c > 1.00$ then $k = 10^{0.952\text{-}3.04 \cdot I_c}$

$I_c \leq 4.00$ and $I_c > 3.27$ then $k = 10^{-4.52\text{-}1.37 \cdot I_c}$

:: N_{SPT} (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Drained Friction Angle, φ (°) ::

$$\varphi = \varphi'_{cv} + 15.94 \cdot \log(Q_{tn,cs}) - 26.88$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$
$\alpha = 14$ for $Q_{tn} > 14$
$\alpha = Q_{tn}$ for $Q_{tn} \leq 14$
$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$

If $I_c \geq 2.20$

$$M_{CPT} = 0.03 \cdot (q_t - \sigma_v) \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Small strain shear Modulus, Go (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Shear Wave Velocity, Vs (m/s) ::

$$V_s = \left(\frac{G_0}{\rho}\right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot + 7 \cdot \log(F_r))}\right]^{1.25}$$ or user defined

$$OCR = k_{OCR} \cdot Q_{tn}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_O = (1 - \sin\varphi') \cdot OCR^{\sin\varphi'}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Soil Sensitivity, S_t ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Peak Friction Angle, φ' (°) ::

$$\varphi' = 29.5° \cdot B_q^{0.121} \cdot \left(0.256 + 0.336 \cdot B_q + \log Q_t\right)$$

(applicable for $0.10 < B_q < 1.00$)

References

- Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 5th Edition, November 2012
- Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)
- N Barounis, J Philpot, Estimation of in-situ water content, void ratio, dry unit weight and porosity using CPT for saturated sands, Proc. 20th NZGS Geotechnical Symposium

Haley & Aldrich Inc.

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CPT: CPT-05

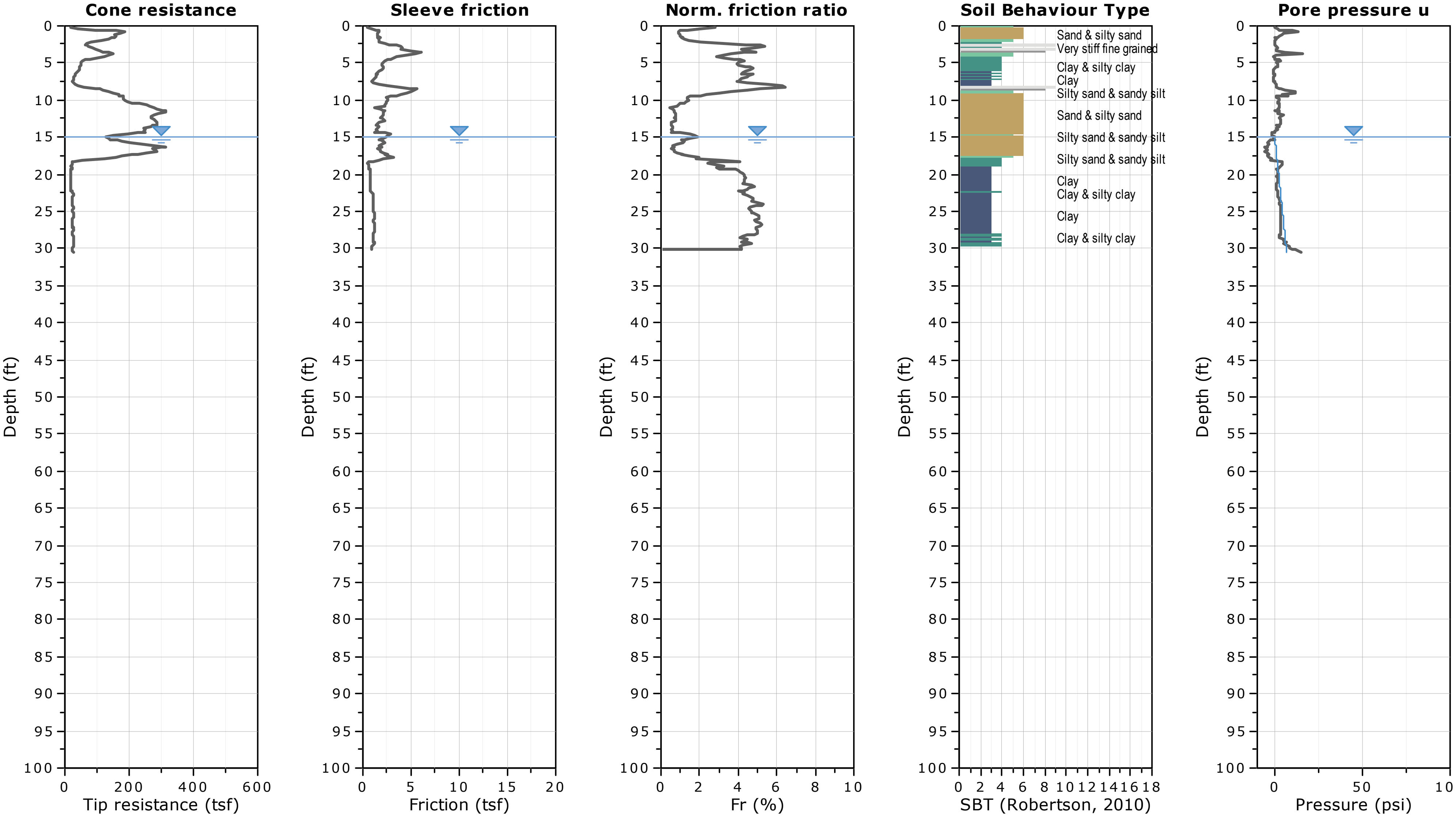
Total depth: 30.51 ft, Date: 11/1/2024

Cone Type: 15cm²

Cone Operator: JM-IY

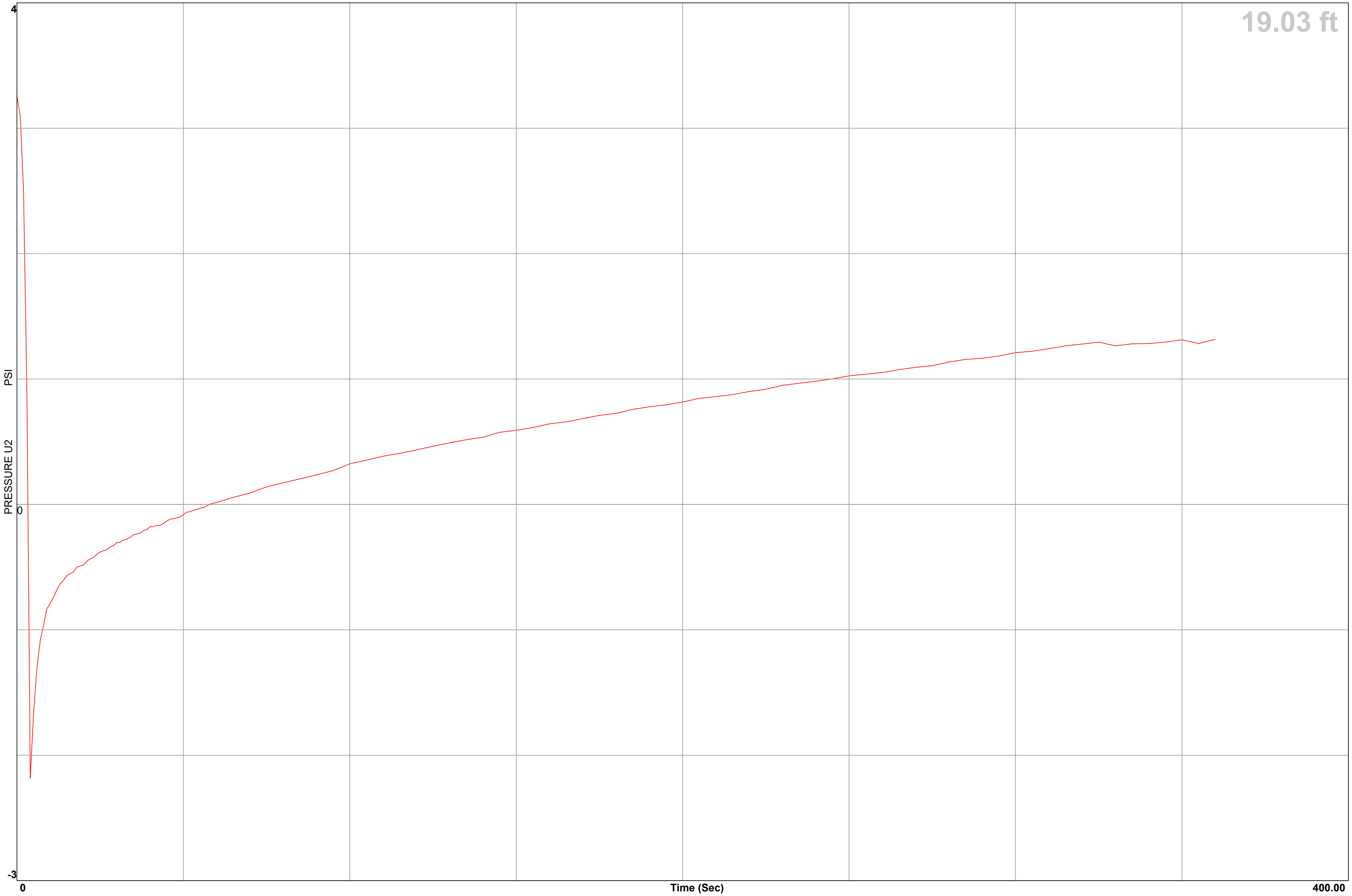
Project: New Leaf Minto Road Watsonville

Location: 90 Minto Road, Watsonville, CA 95076



Haley & Aldrich Inc.

Location	New Leaf Minto Road Watsonville	Operator	JM-IY		
Job Number	021059-003	Cone Number	DDG1589	GPS	
Hole Number	CPT-06	Date and Time	11/1/2024 2:28:29 PM		
Equilized Pressure	1.3	EST GW Depth During Test	15.9		



CPT: CPT-06

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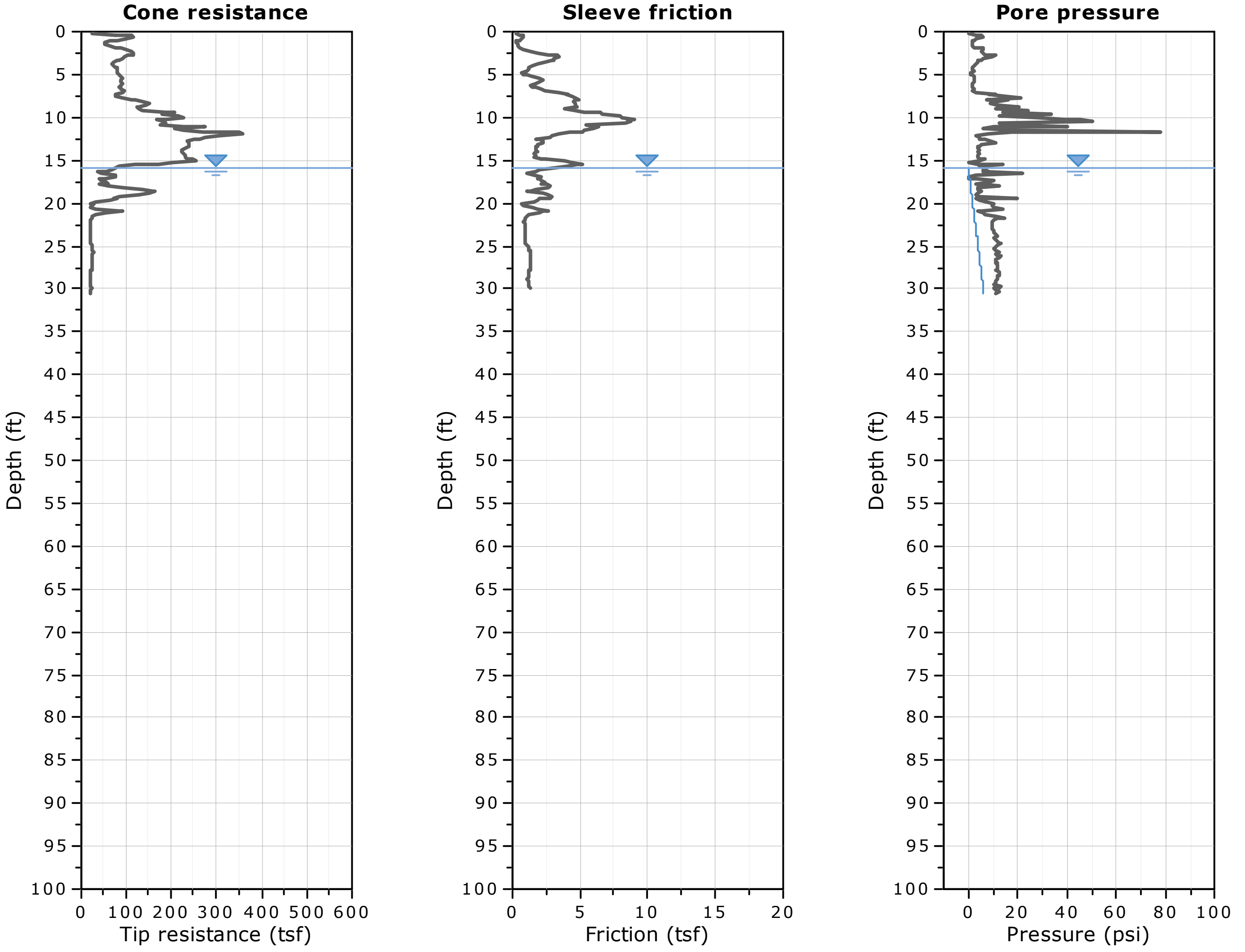
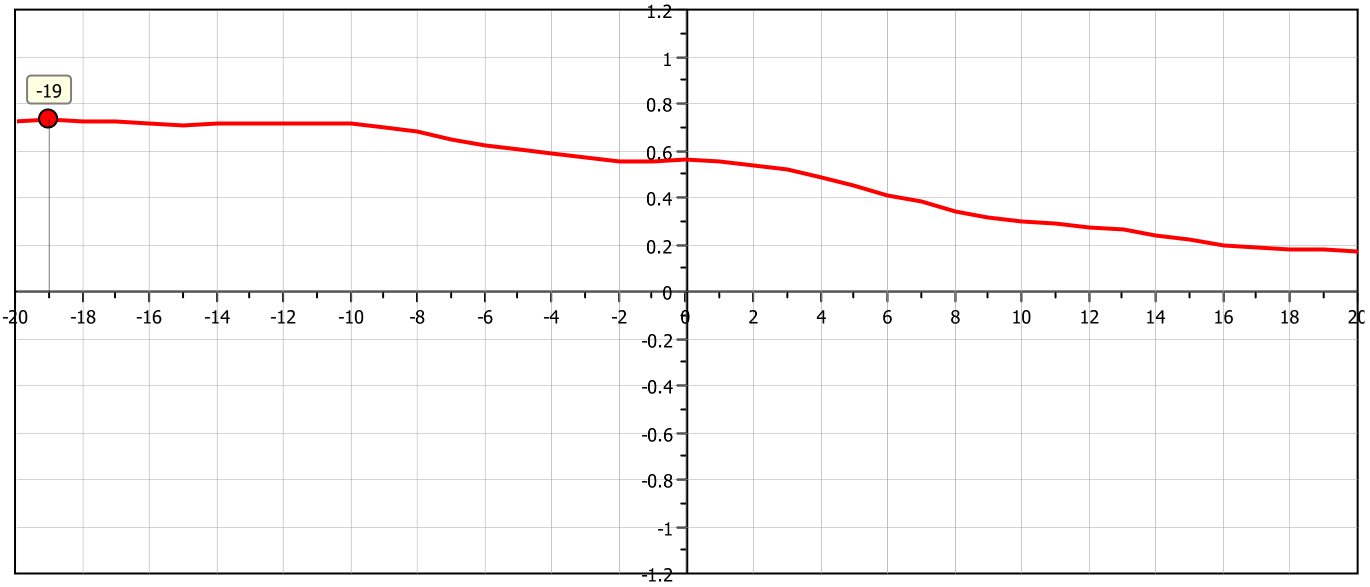
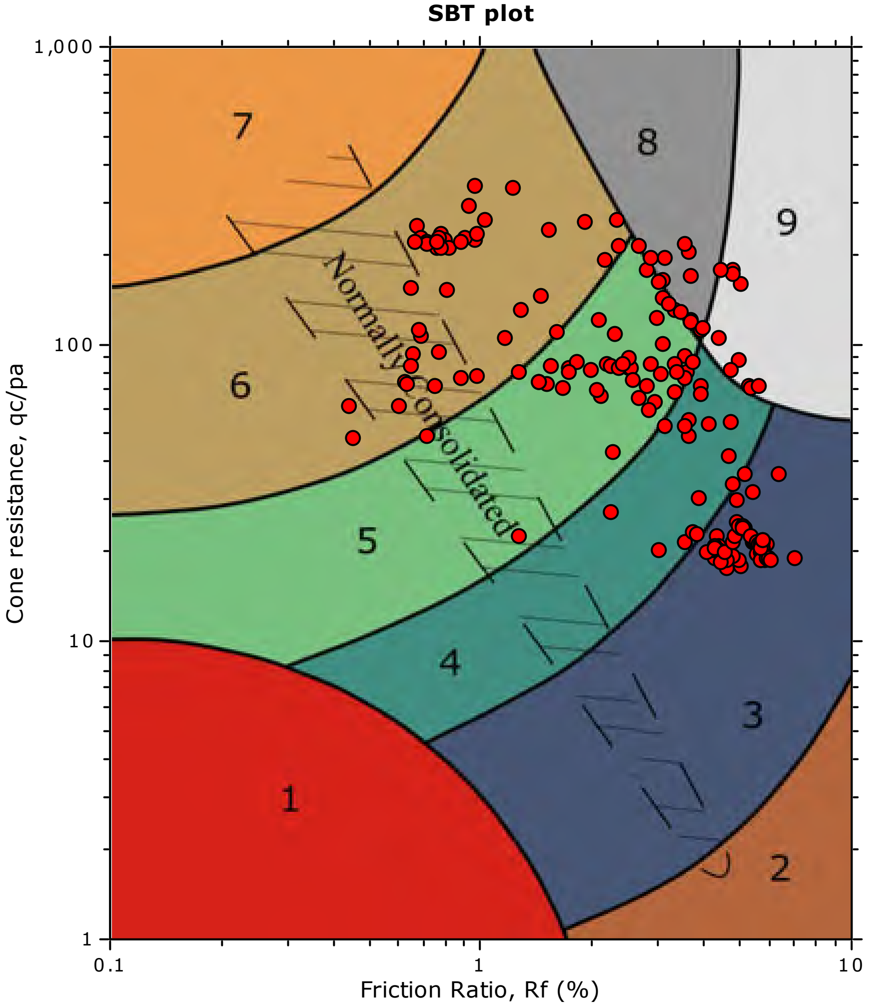


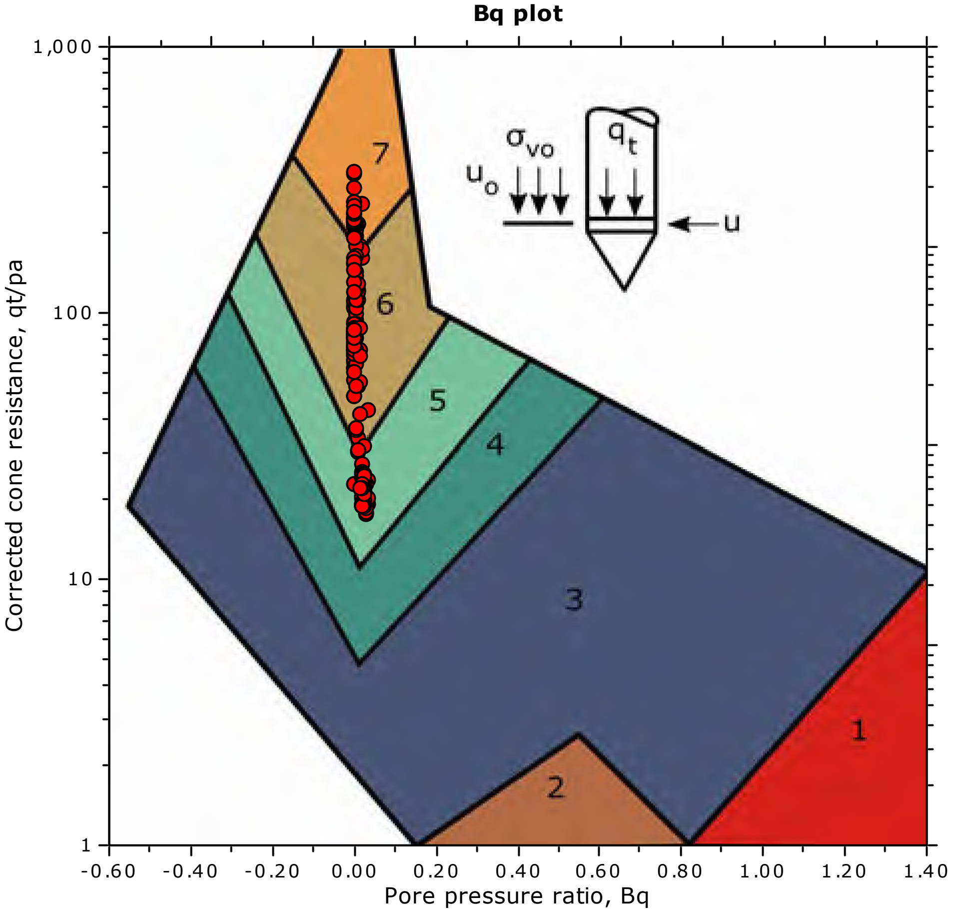
The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).

Cross correlation between qc & fs



SBT - Bq plots





SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
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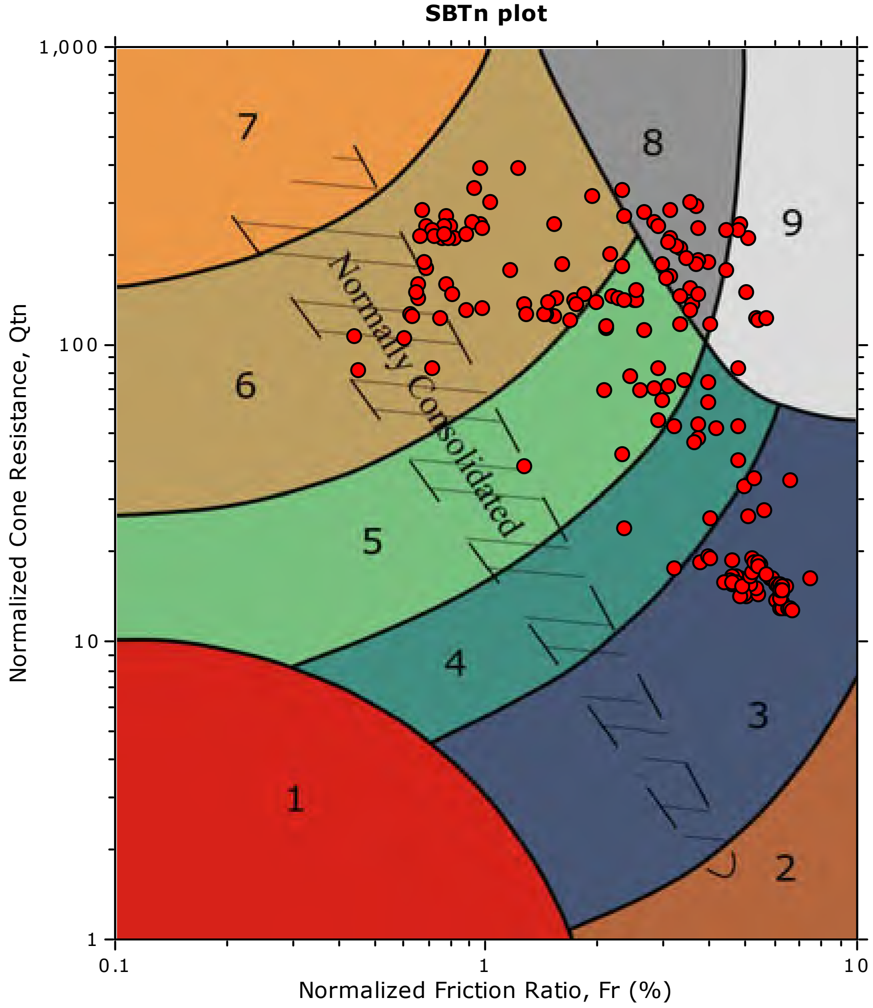
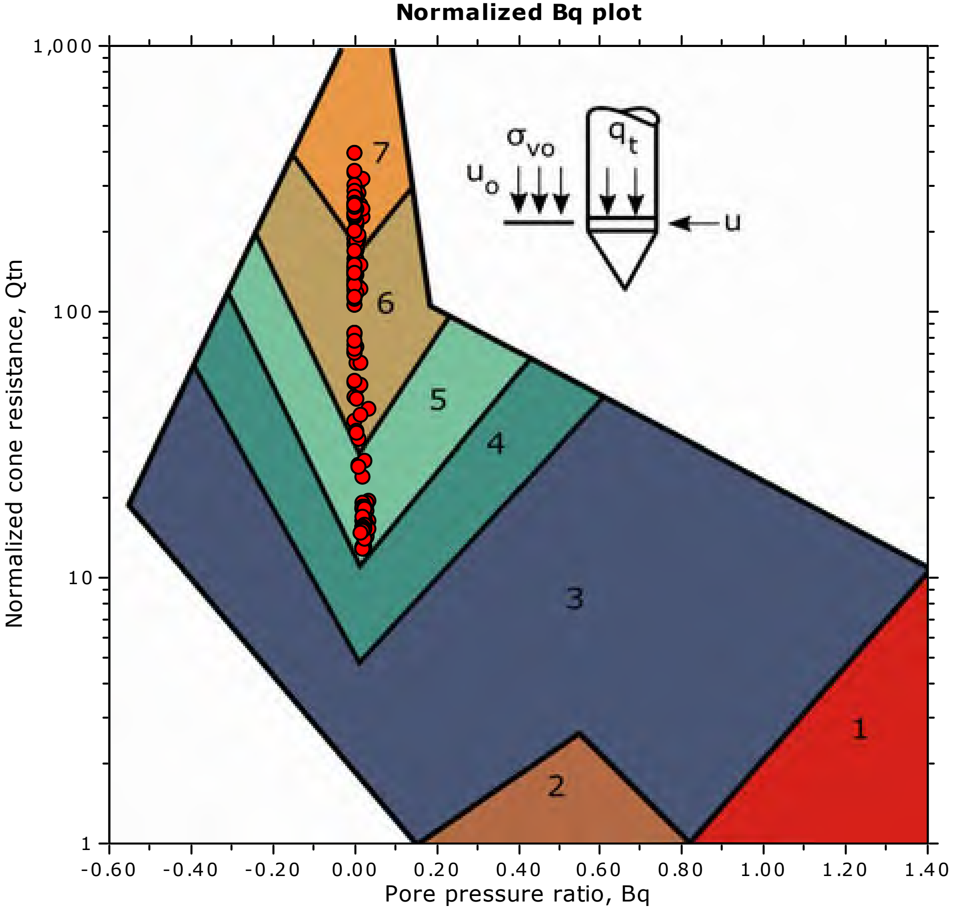
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravelly sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

SBT - Bq plots (normalized)





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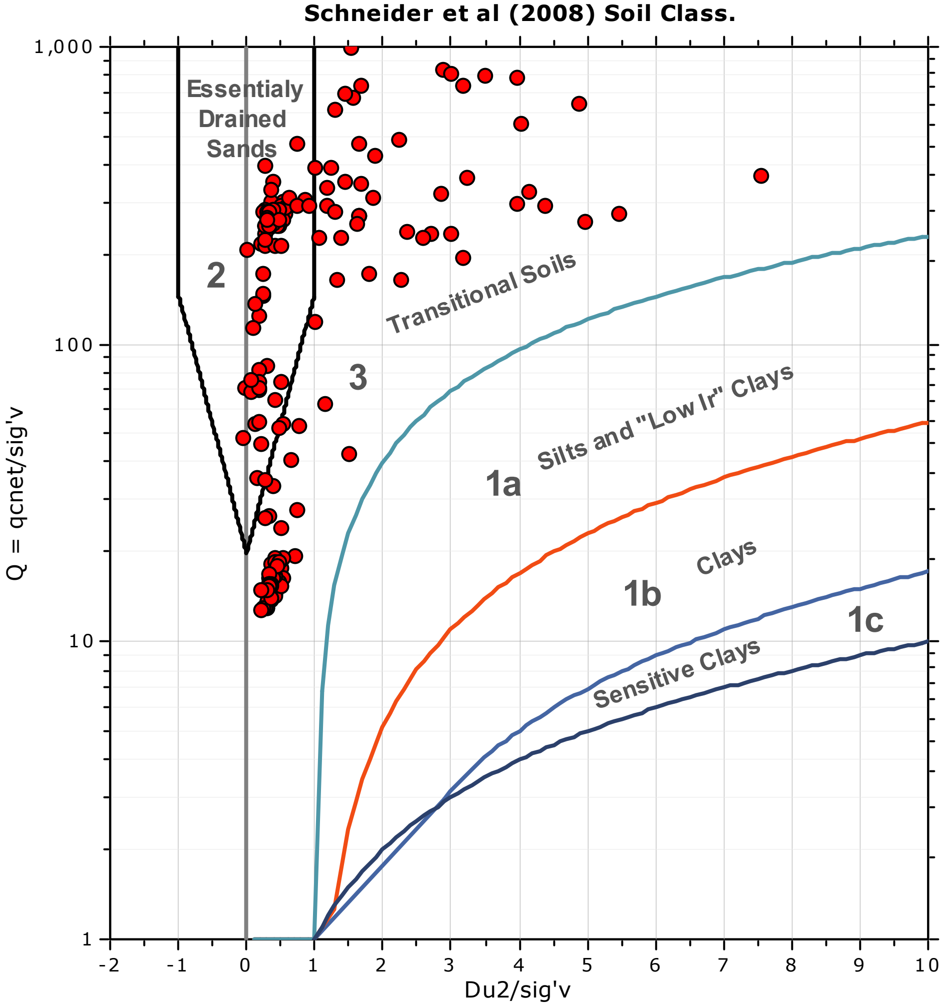
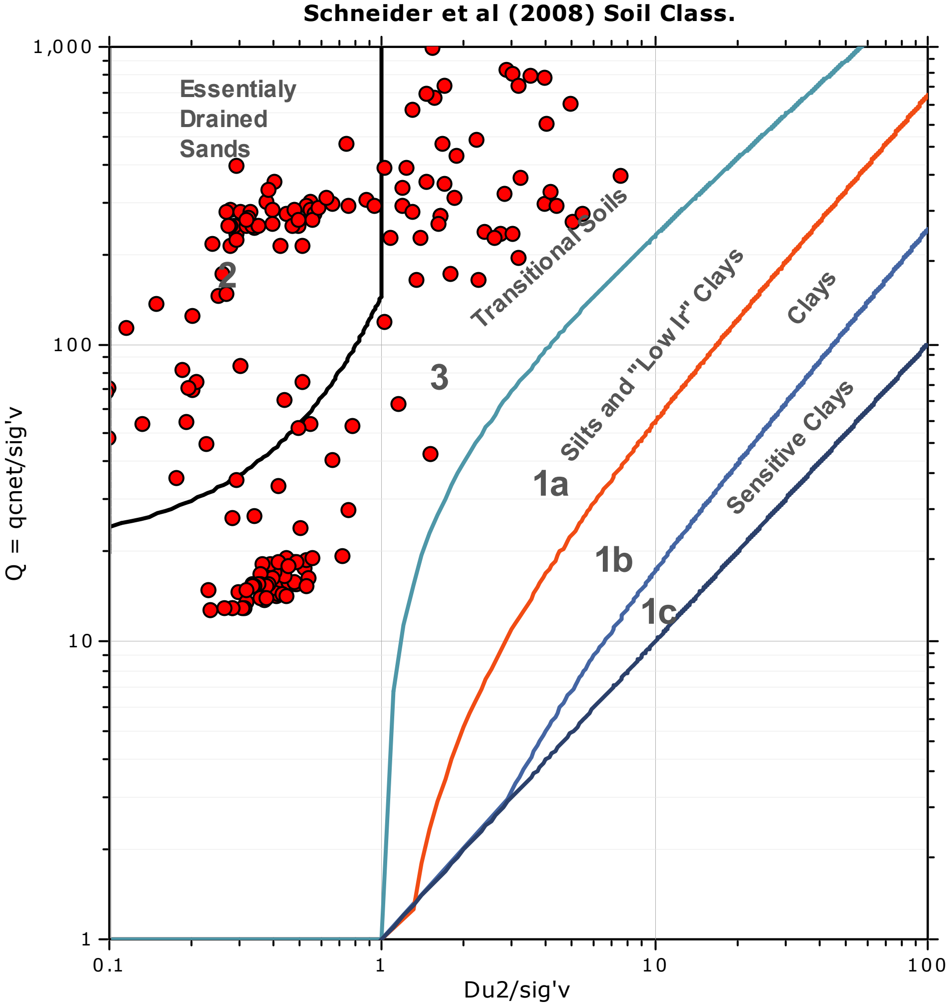
⬜ 9. Very stiff fine grained

Bq plots (Schneider)





Haley & Aldrich Inc.

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Total depth: 30.51 ft, Date: 11/1/2024

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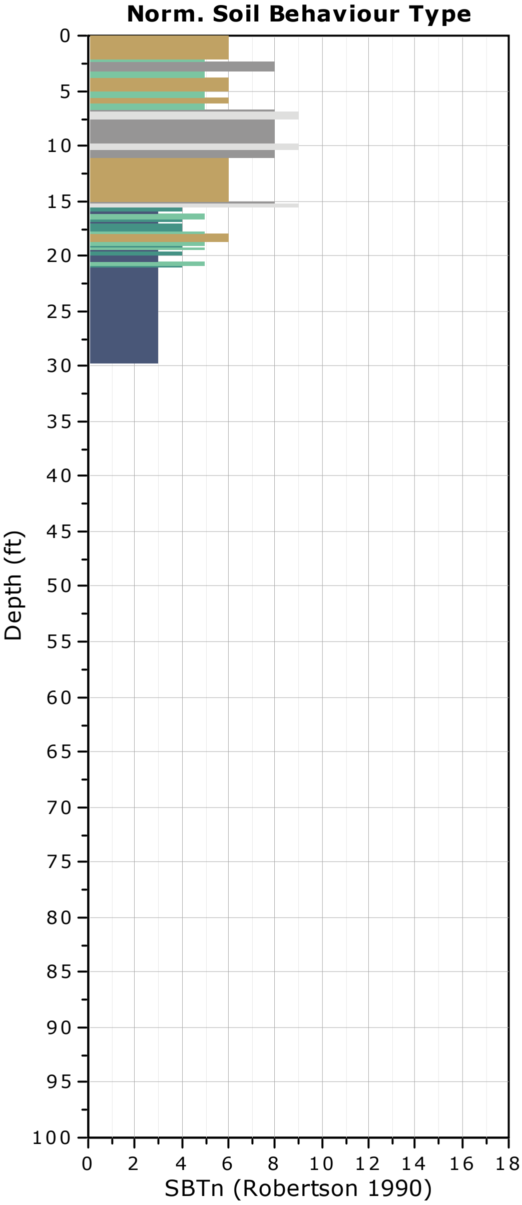

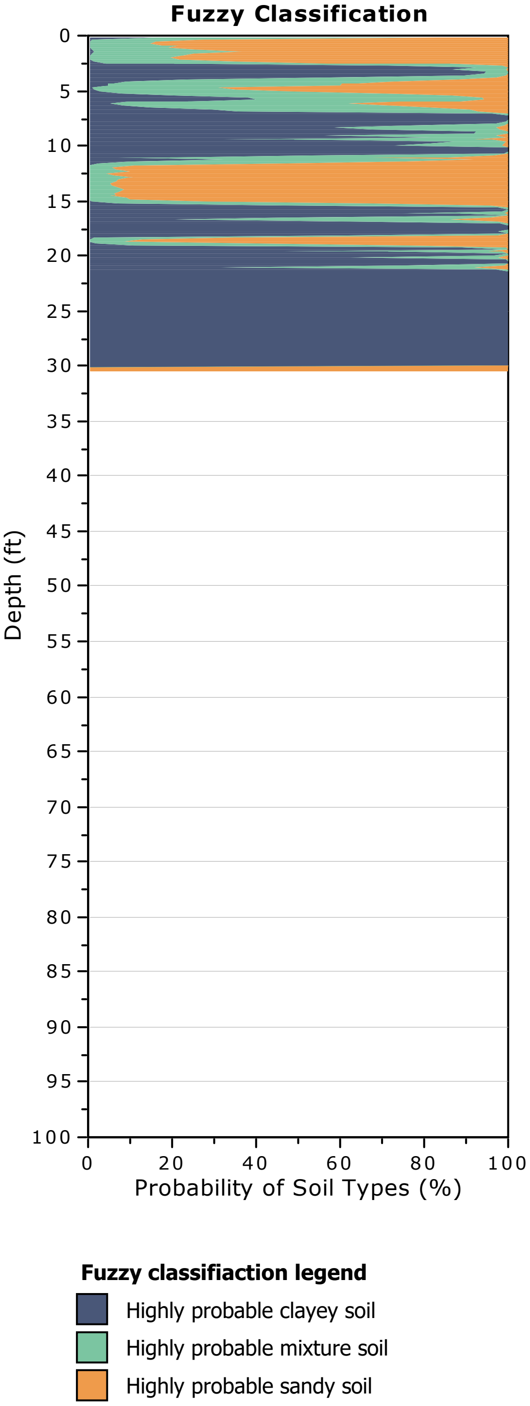

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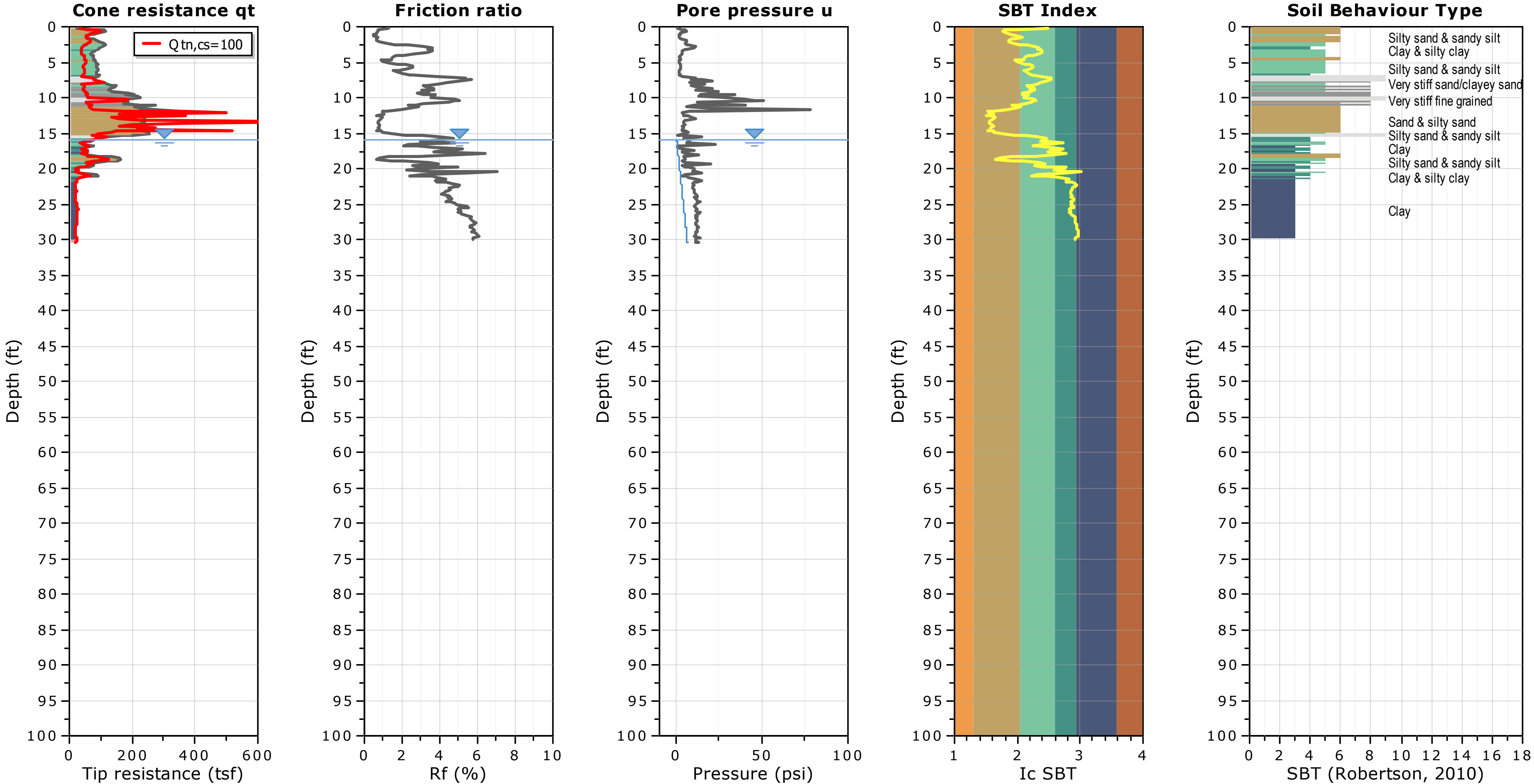

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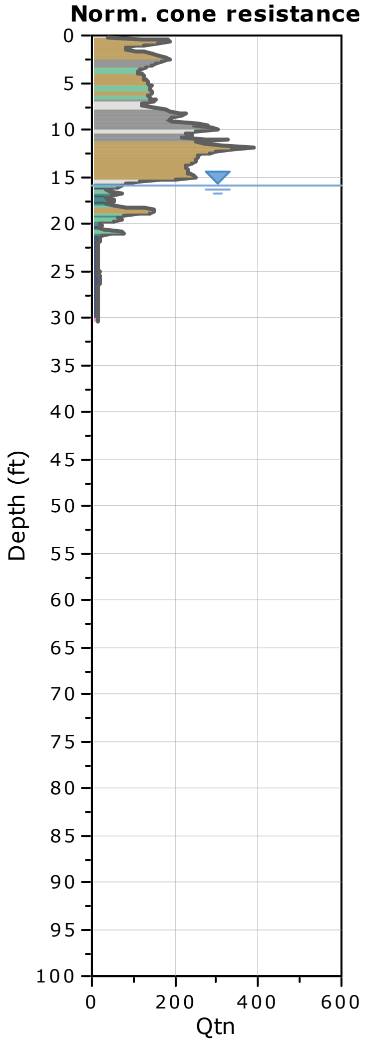
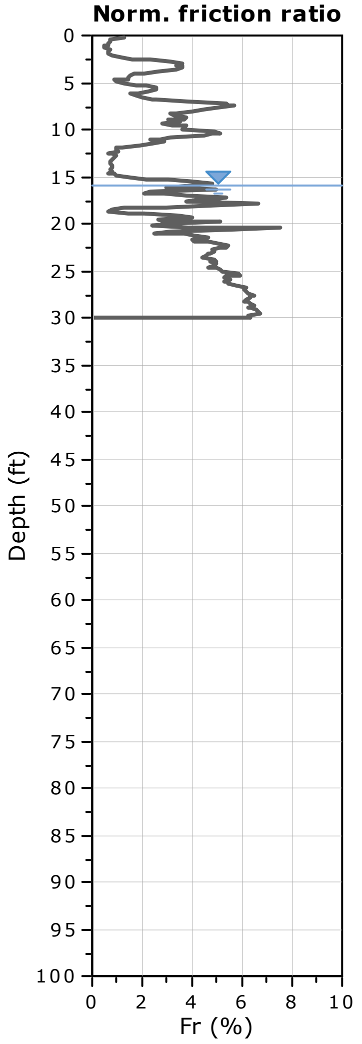
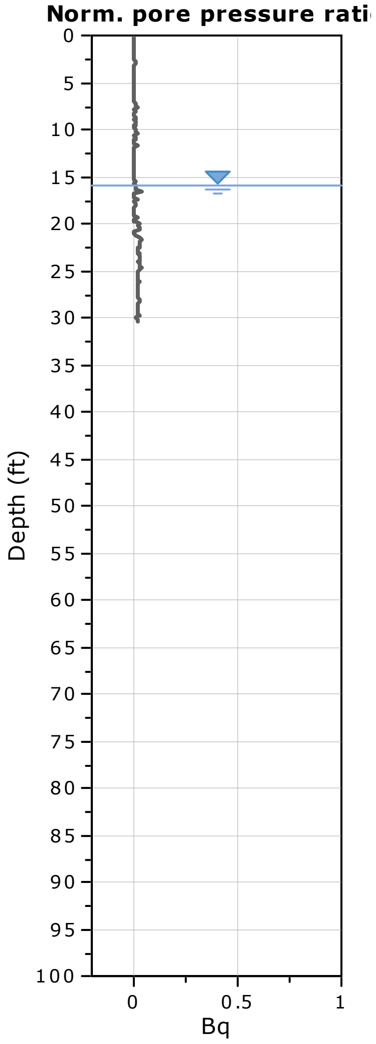
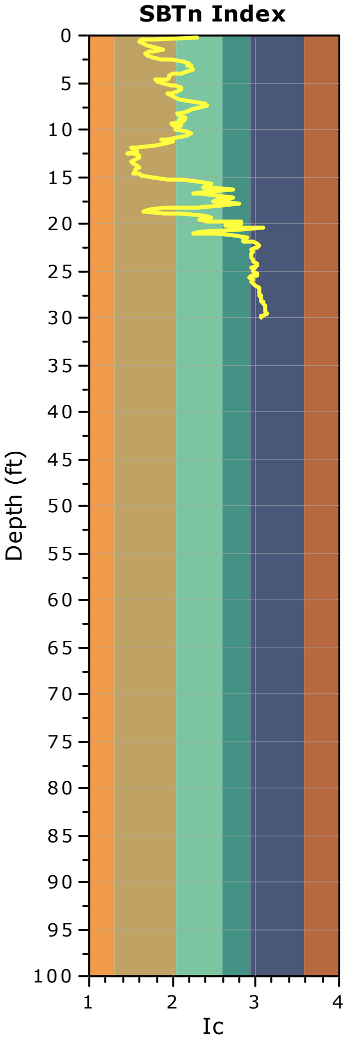
Location: **90 Minto Road, Watsonville, CA 95076**

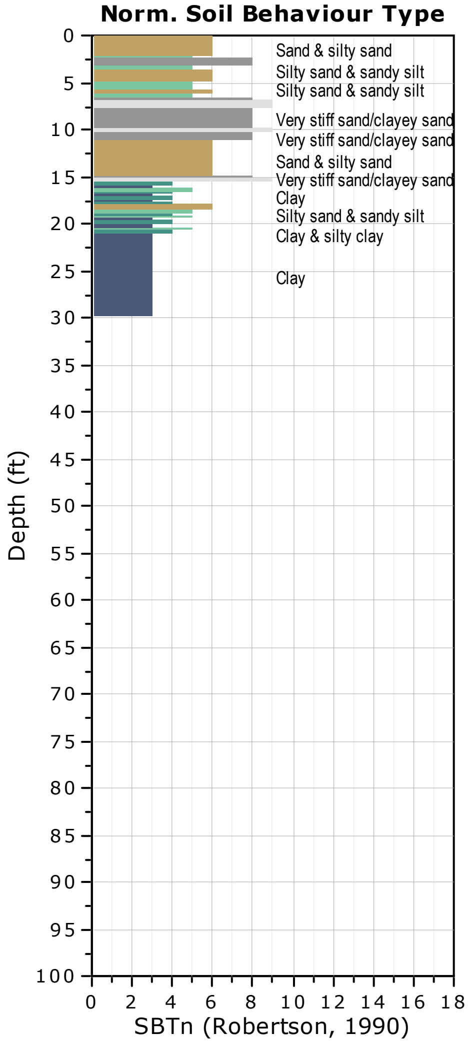
CPT: CPT-06
Total depth: 30.51 ft, Date: 11/1/2024
Cone Type: 15cm²
Cone Operator: JM-IY









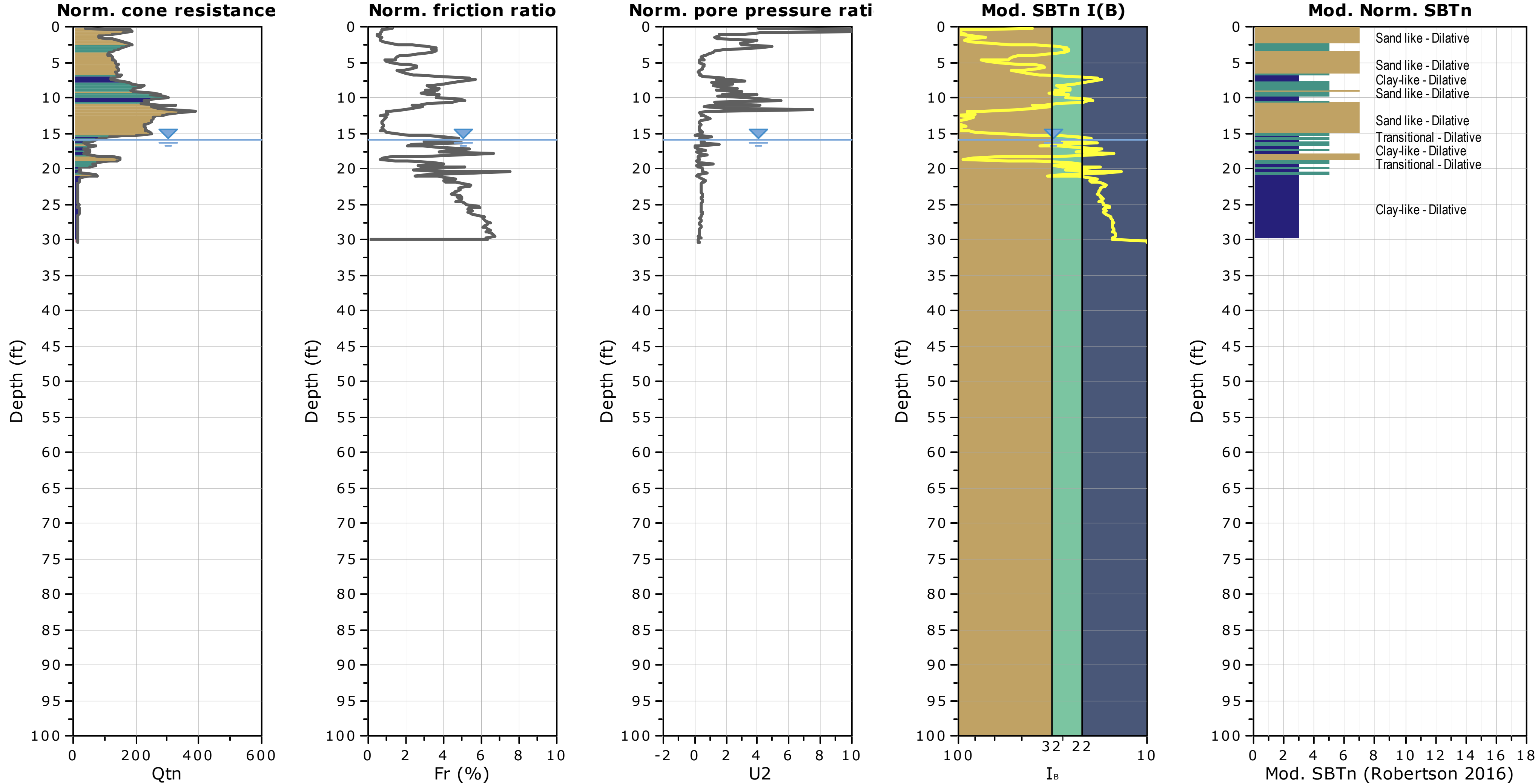


SBTn legend

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

CPT: CPT-06
Total depth: 30.51 ft, Date: 11/1/2024
Cone Type: 15cm²
Cone Operator: JM-IY

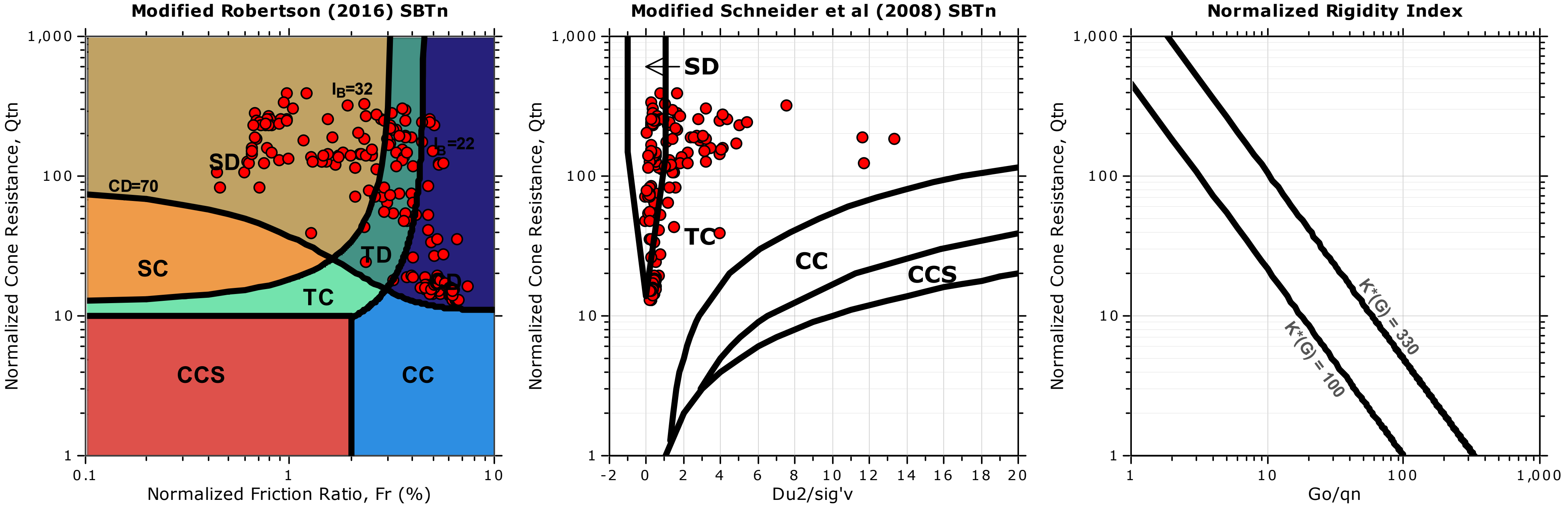
Project: **New Leaf Minto Road Watsonville**
Location: **90 Minto Road, Watsonville, CA 95076**



Mod. SBTn legend

🟥 1. CCS: ClayLike - Contractive, Sensitive
🟦 2. CC: Clay-like - Contractive
🟦 3. CD: Clay-Like: Dilative
🟩 4. TC: Transitional - Contractive
🟩 5. TD: Transitional - Dilative
🟧 6. SC: Sand-like - Contractive
🟫 7. SD: Sand-like - Dilative

Updated SBTn plots



CCS: Clay-like - Contractive - Sensitive
CC: Clay-like - Contractive
CD: Clay-like - Dilative
TC: Transitional - Contractive
TD: Transitional - Dilative
SC: Sand-like - Contractive
SD: Sand-like - Dilative

K(G) > 330: Soils with significant microstructure (e.g. age/cementation)

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CPT: CPT-06

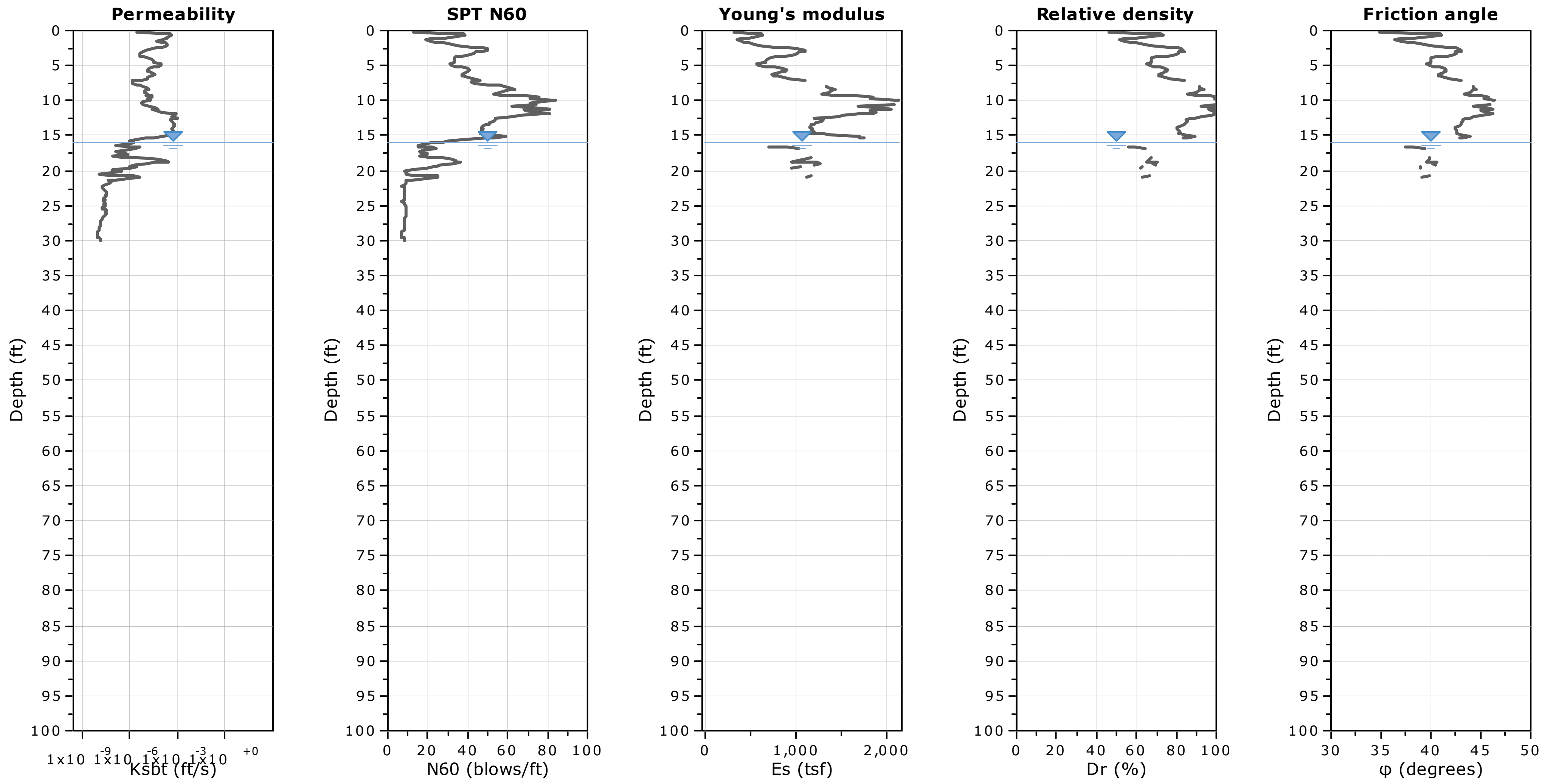
Total depth: 30.51 ft, Date: 11/1/2024

Cone Type: 15cm²

Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**

Location: **90 Minto Road, Watsonville, CA 95076**



Calculation parameters

Permeability: Based on SBT_n

SPT N_{60}: Based on I_c and q_t

Young's modulus: Based on variable alpha using I_c (Robertson, 2009)

Relative density constant, C_{Dr}: 350.0

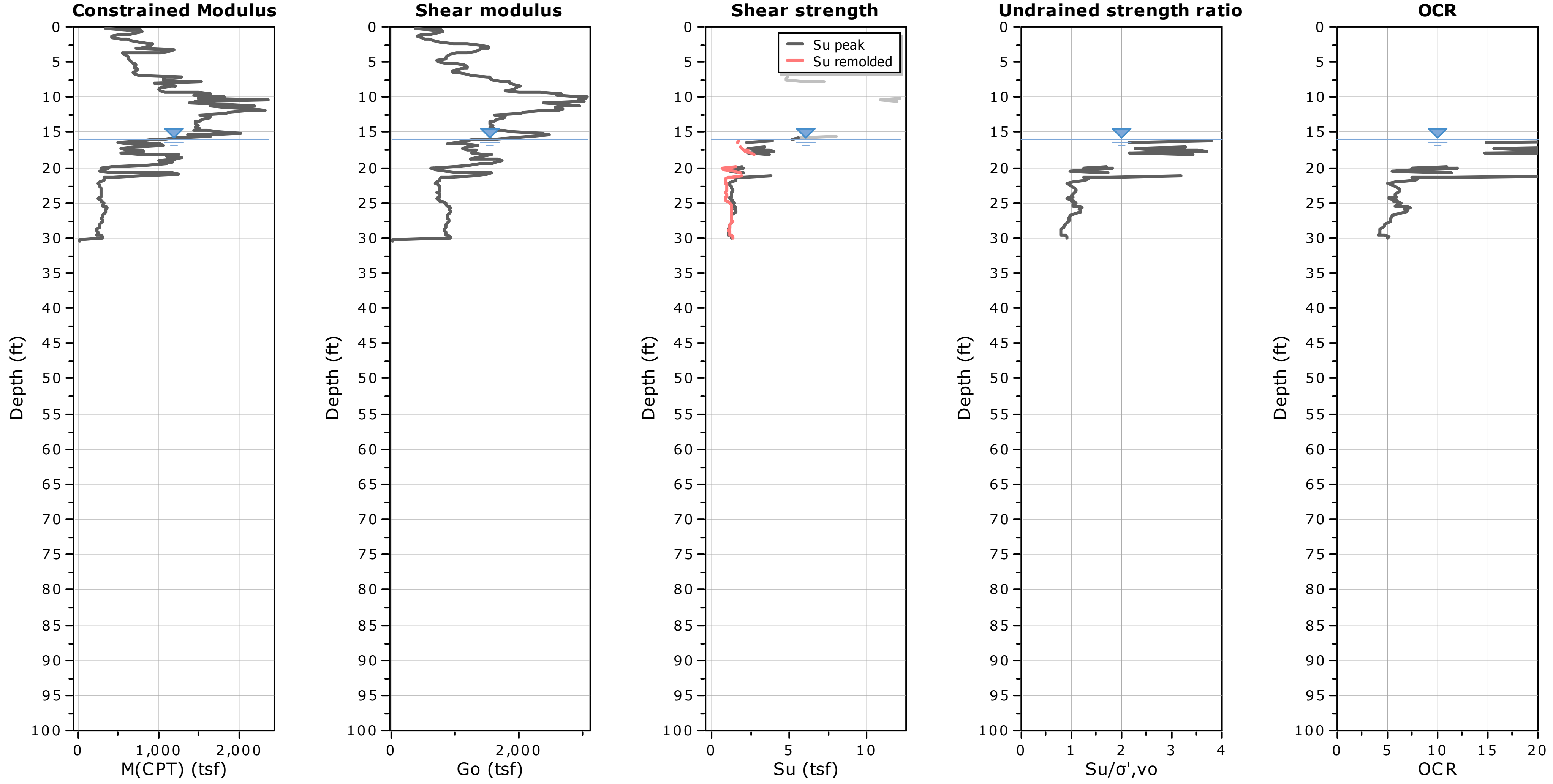
Phi: Based on Kulhawy & Mayne (1990)

Haley & Aldrich Inc.

https://www.haleyaldrich.com/

CPT: CPT-06
Total depth: 30.51 ft, Date: 11/1/2024
Cone Type: 15cm²
Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**
Location: **90 Minto Road, Watsonville, CA 95076**



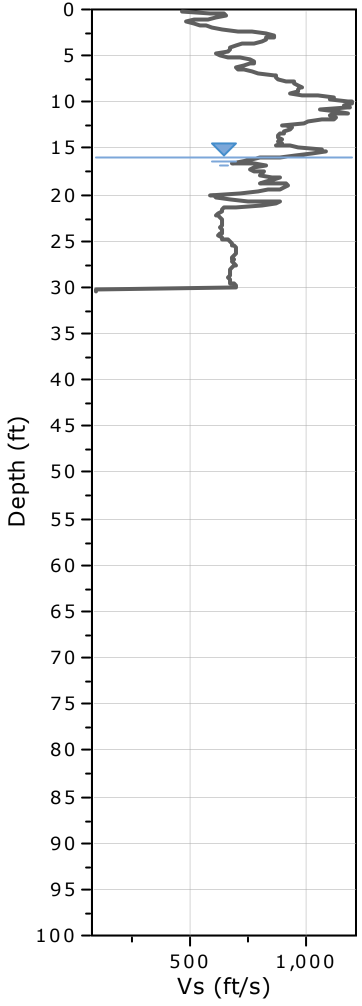
Calculation parameters

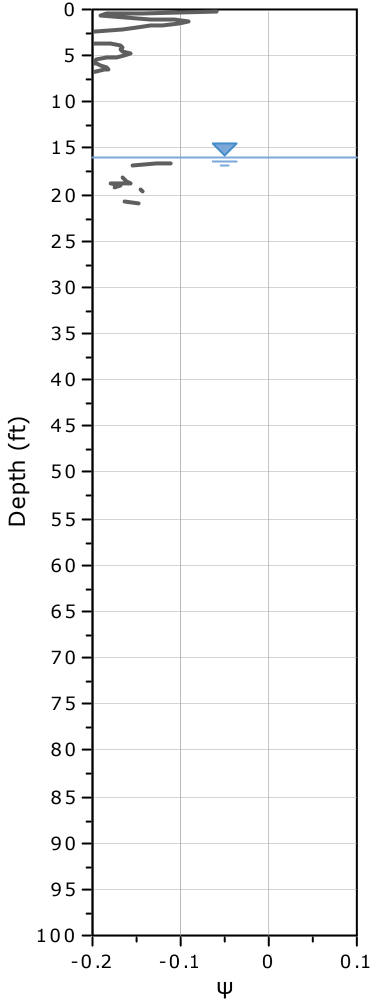
Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable *alpha* using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: Auto

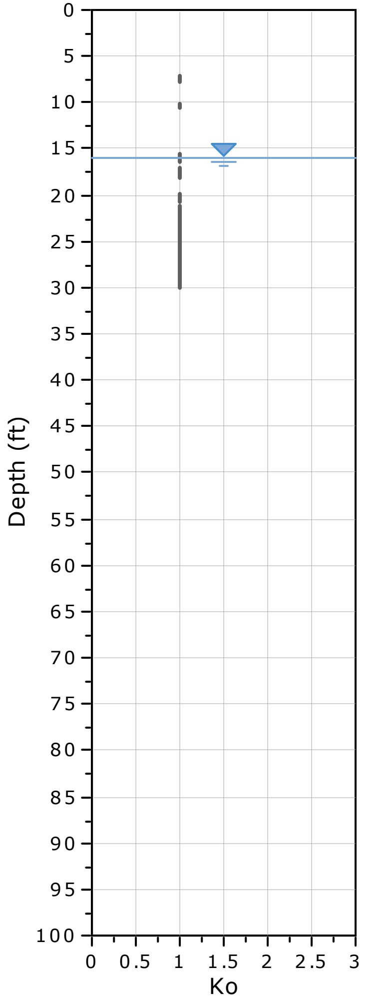
OCR factor for clays, N_{kt}: Auto
● Flat Dilatometer Test data

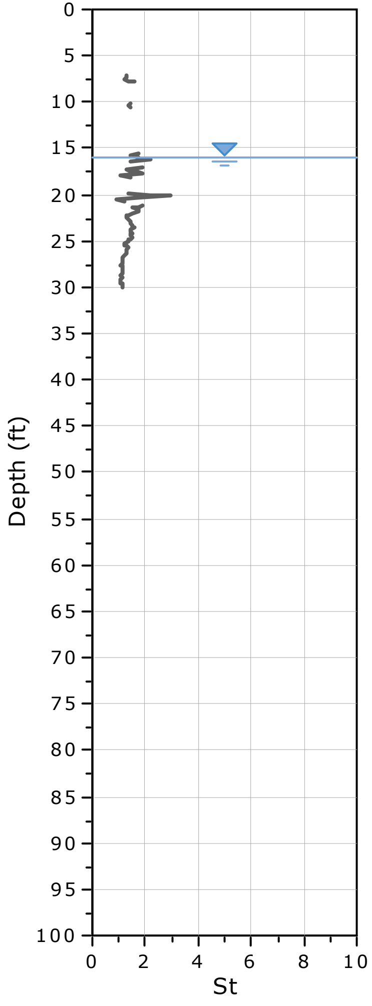
CPT: CPT-06
Total depth: 30.51 ft, Date: 11/1/2024
Cone Type: 15cm²
Cone Operator: JM-IY

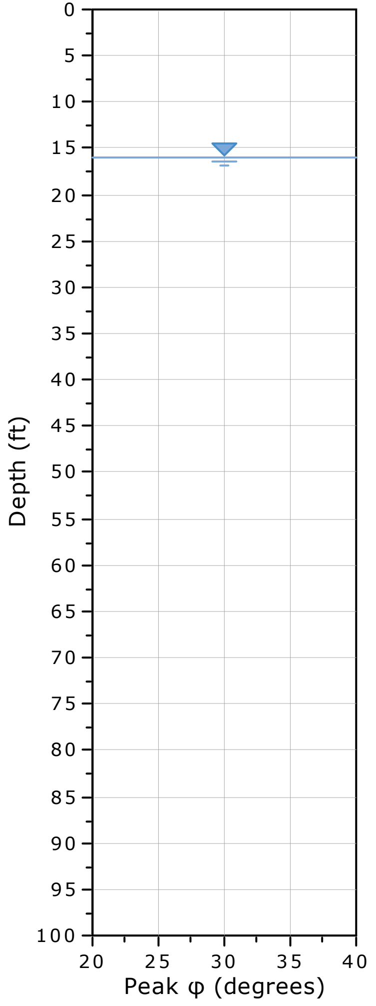
Project: **New Leaf Minto Road Watsonville**
Location: **90 Minto Road, Watsonville, CA 95076**


Shear Wave velocity


State parameter


In-situ stress ratio


Soil sensitivity


Effective friction angle

Calculation parameters

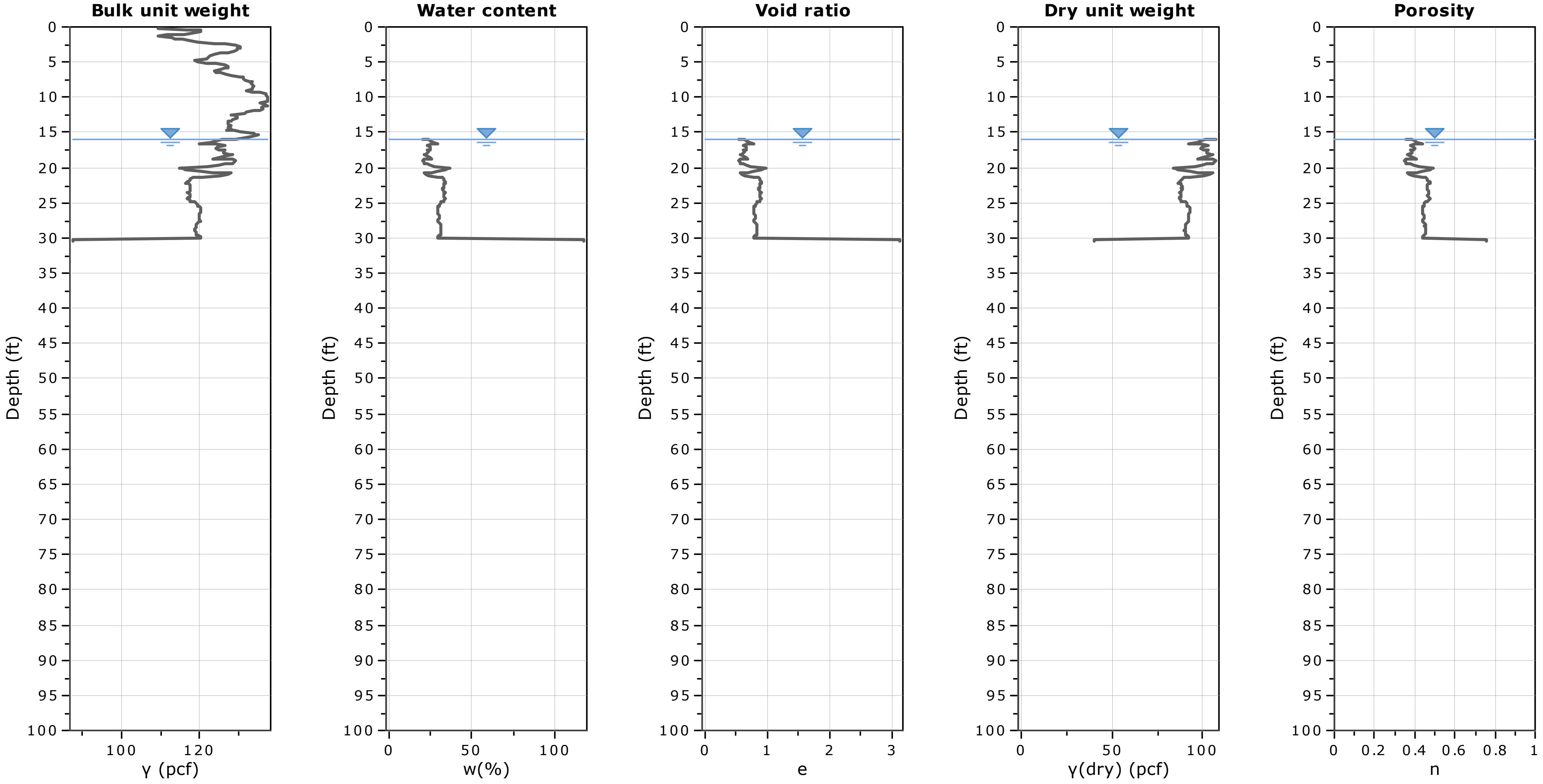
Soil Sensitivity factor, N_S: 7.00

CPT: CPT-06

Total depth: 30.51 ft, Date: 11/1/2024

Cone Type: 15cm²

Cone Operator: JM-IY



CPT: CPT-06

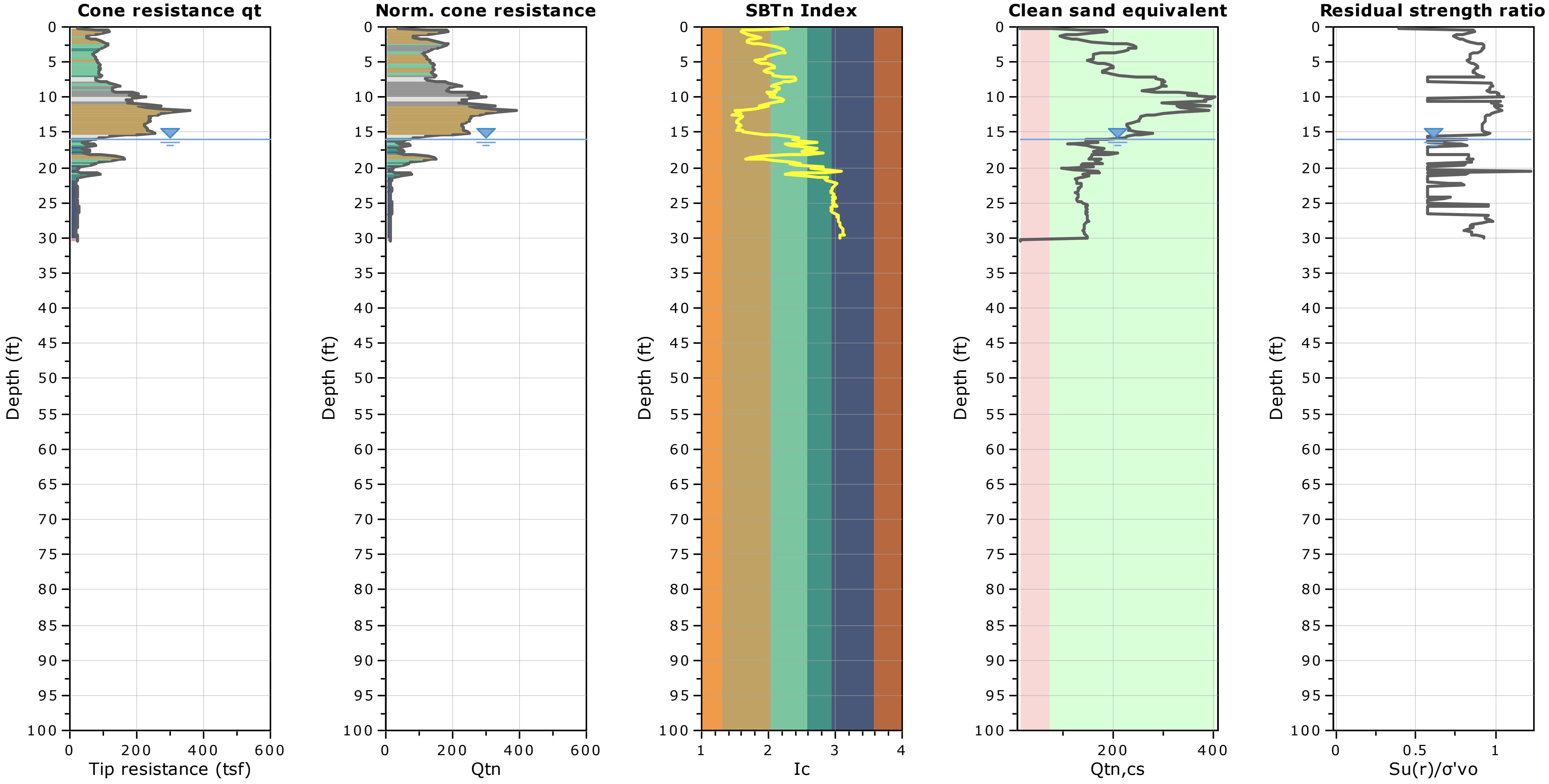
Total depth: 30.51 ft, Date: 11/1/2024

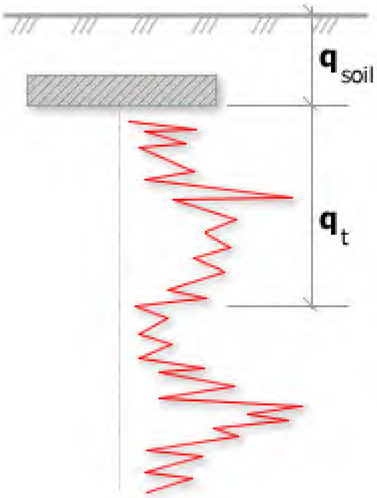
Cone Type: 15cm²

Cone Operator: JM-IY

Project: **New Leaf Minto Road Watsonville**

Location: **90 Minto Road, Watsonville, CA 95076**

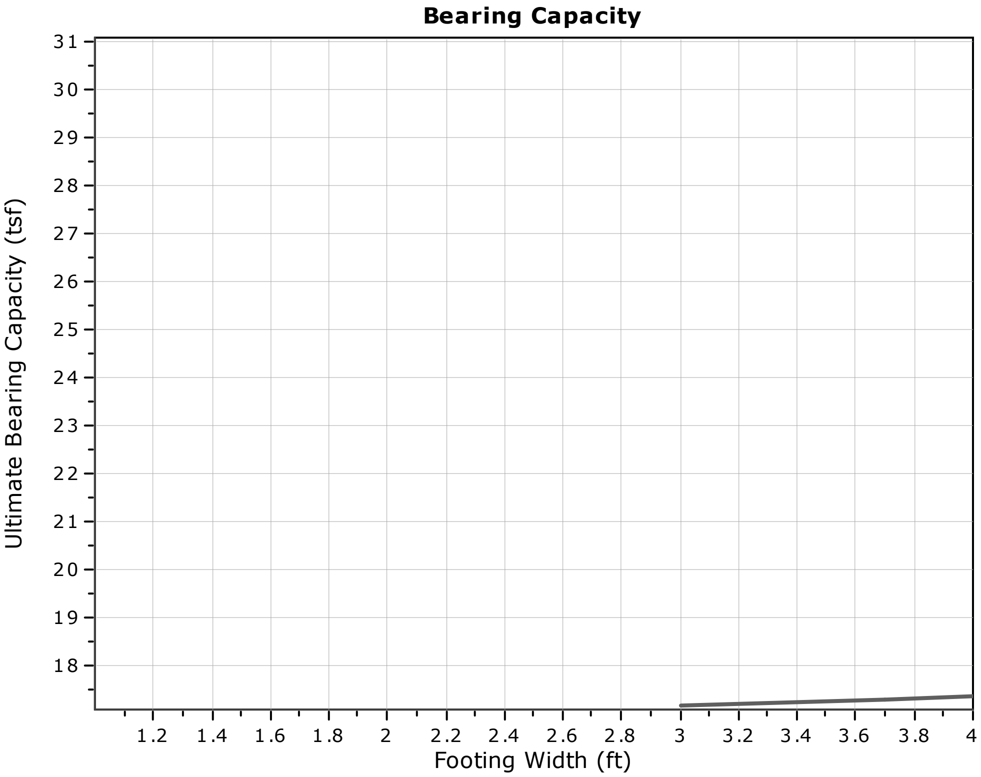




Bearing Capacity calculation is perfromed based on the formula:

$$Q_{ult} = R_k \times q_t + q_{soil}$$

where:
R_k: Bearing capacity factor
q_t: Average corrected cone resistance over calculation depth
q_{soil}: Pressure applied by soil above footing



:: Tabular results ::

No	B (ft)	Start Depth (ft)	End Depth (ft)	Ave. q_t (tsf)	R_k	Soil Press. (tsf)	Ult. bearing cap. (tsf)
1	3.00	1.60	6.10	85.35	0.20	0.10	17.17
2	3.70	1.60	7.15	85.97	0.20	0.10	17.29
3	4.40	1.60	8.20	86.75	0.20	0.10	17.45
4	5.10	1.60	9.25	93.80	0.20	0.10	18.86
5	5.80	1.60	10.30	105.58	0.20	0.10	21.21
6	6.50	1.60	11.35	116.69	0.20	0.10	23.43
7	7.20	1.60	12.40	132.89	0.20	0.10	26.67
8	7.90	1.60	13.45	142.01	0.20	0.10	28.50
9	8.60	1.60	14.50	149.52	0.20	0.10	30.00
10	9.30	1.60	15.55	154.62	0.20	0.10	31.02
11	10.00	1.60	16.60	148.19	0.20	0.10	29.73
12	10.70	1.60	17.65	142.65	0.20	0.10	28.63
13	11.40	1.60	18.70	139.66	0.20	0.10	28.03
14	12.10	1.60	19.75	137.54	0.20	0.10	27.60
15	12.80	1.60	20.80	131.88	0.20	0.10	26.47

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236\right)$$

where $g_w =$ water unit weight

:: Permeability, k (m/s) ::

$I_c < 3.27$ and $I_c > 1.00$ then $k = 10^{0.952-3.04 \cdot I_c}$

$I_c \leq 4.00$ and $I_c > 3.27$ then $k = 10^{-4.52-1.37 \cdot I_c}$

:: N_{SPT} (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Drained Friction Angle, φ (°) ::

$$\varphi = \varphi'_{cv} + 15.94 \cdot \log(Q_{tn,cs}) - 26.88$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$
$\alpha = 14$ for $Q_{tn} > 14$
$\alpha = Q_{tn}$ for $Q_{tn} \leq 14$
$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$

If $I_c \geq 2.20$

$$M_{CPT} = 0.03 \cdot (q_t - \sigma_v) \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Small strain shear Modulus, Go (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Shear Wave Velocity, Vs (m/s) ::

$$V_s = \left(\frac{G_0}{\rho}\right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot + 7 \cdot \log(F_r))}\right]^{1.25} \text{ or user defined}$$

$$OCR = k_{OCR} \cdot Q_{tn}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_O = (1 - \sin\varphi') \cdot OCR^{\sin\varphi'}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Soil Sensitivity, S_t ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Peak Friction Angle, φ' (°) ::

$$\varphi' = 29.5^\circ \cdot B_q^{0.121} \cdot (0.256 + 0.336 \cdot B_q + \log Q_t)$$

(applicable for $0.10 < B_q < 1.00$)

References

- Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 5th Edition, November 2012
- Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)
- N Barounis, J Philpot, Estimation of in-situ water content, void ratio, dry unit weight and porosity using CPT for saturated sands, Proc. 20th NZGS Geotechnical Symposium

Middle Earth
GEO TESTING INC.

Haley & Aldrich Inc.

https://www.haleyaldrich.com/

CPT: CPT-06

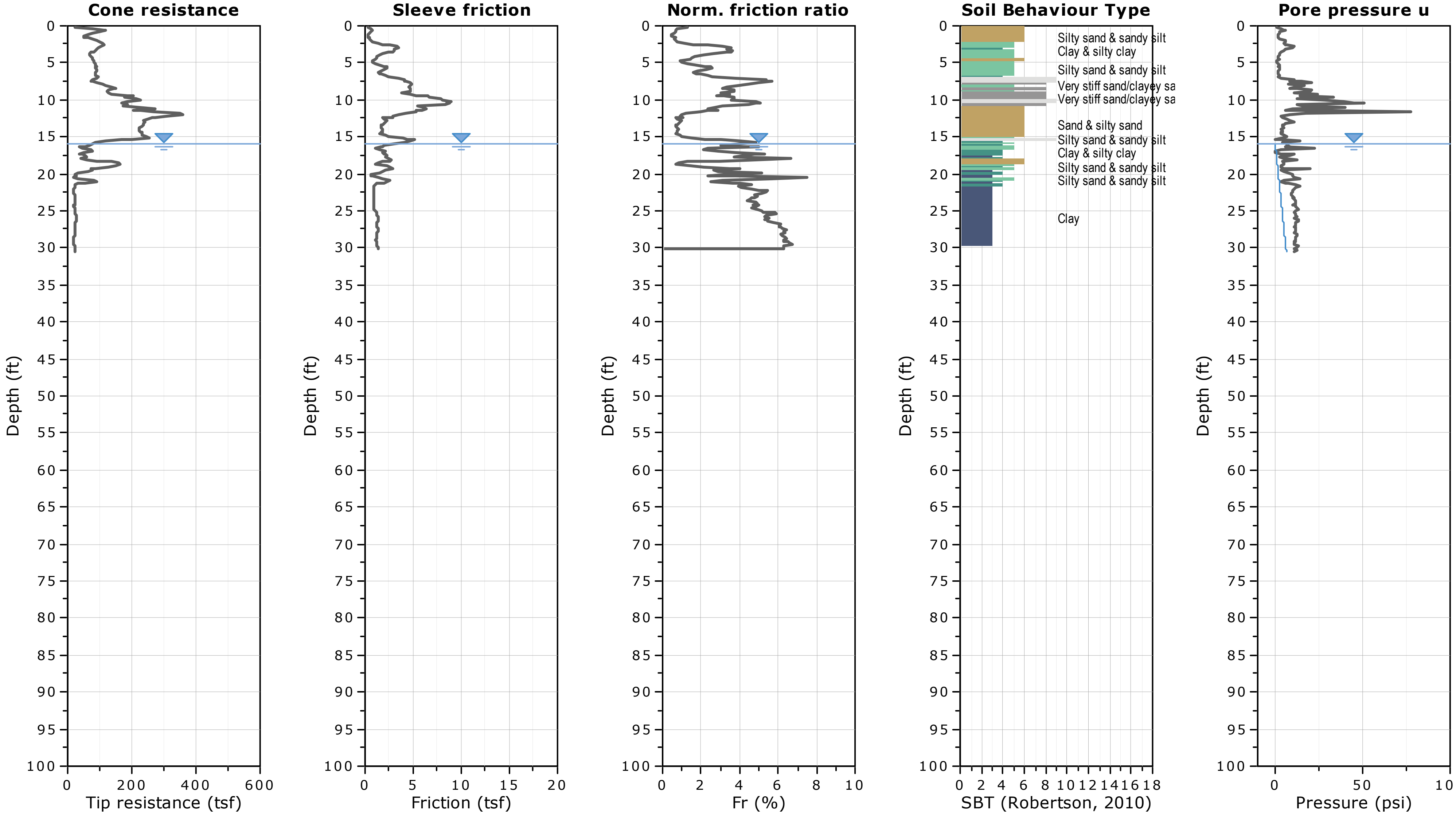
Total depth: 30.51 ft, Date: 11/1/2024

Cone Type: 15cm²

Cone Operator: JM-IY

Project: New Leaf Minto Road Watsonville

Location: 90 Minto Road, Watsonville, CA 95076



APPENDIX B

Laboratory Test Results

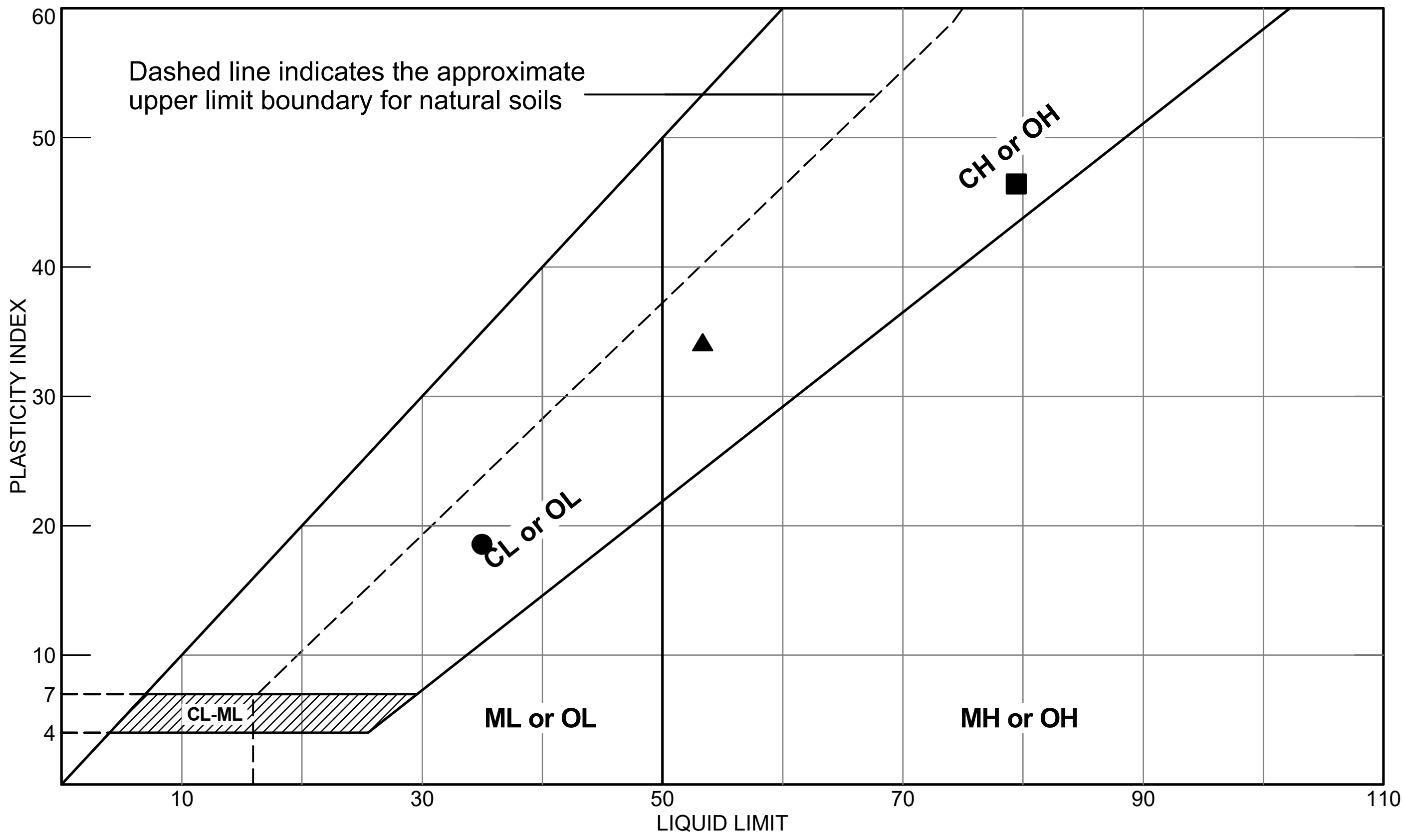
Exploration	Sample ID	Depth	Water Content (%)	Dry Density (pcf)	Fines (%)	Sand (%)	Gravel (%)	Liquid Limit	Plastic Limit	Plasticity Index	Organic Content (%)	Pocket Pen (tsf)	Torvane (tsf)	
B-1	G-1	0.0						35	16	19				
B-1	S-6	12.5	11											
B-1	S-7	15.0	21		41	57	2							
B-1	S-8	20.0	37					79	33	46				
HB-02	G-1	1.5	23					53	19	34				
IT-1	BULK	0-2						30	14	16				

HALEY ALDRICH

Project: NEW LEAF SEAHAWK WATSONVILLE
Location: WATSONVILLE, CALIFORNIA
Project No.: 0210659-004

Summary of Laboratory Results

Figure **B-1**
Sheet **1 of 1**



Location and Description	LL	PL	PI	-#200	MC%	USCS
● **Source:** B-1 **Sample No.:** G-1 **Depth:** 0.0 to 2.5 LEAN CLAY	35	16	19	NT	NT	CL
■ **Source:** B-1 **Sample No.:** S-8 **Depth:** 20.0 to 21.5 FAT CLAY	79	33	46	NT	37	CH
▲ **Source:** HB-02 **Sample No.:** G-1 **Depth:** 1.5 to 3.0 FAT CLAY	53	19	34	NT	23	CH

Remarks:

●

■

▲

HALEY ALDRICH

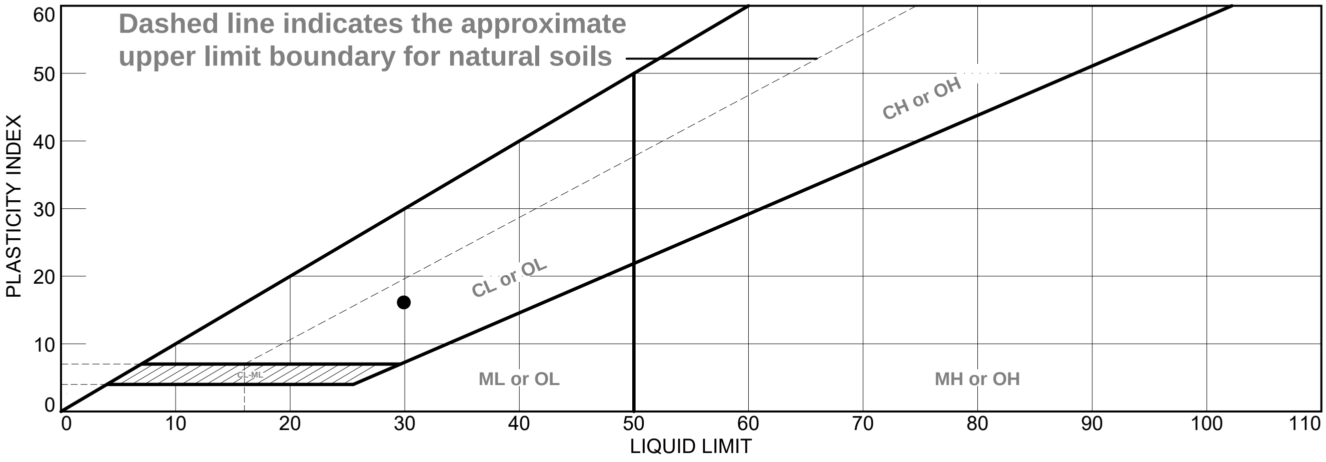
Project: NEW LEAF SEAHAWK WATSONVILLE
Location: WATSONVILLE, CALIFORNIA
Project No.: 0210659-004

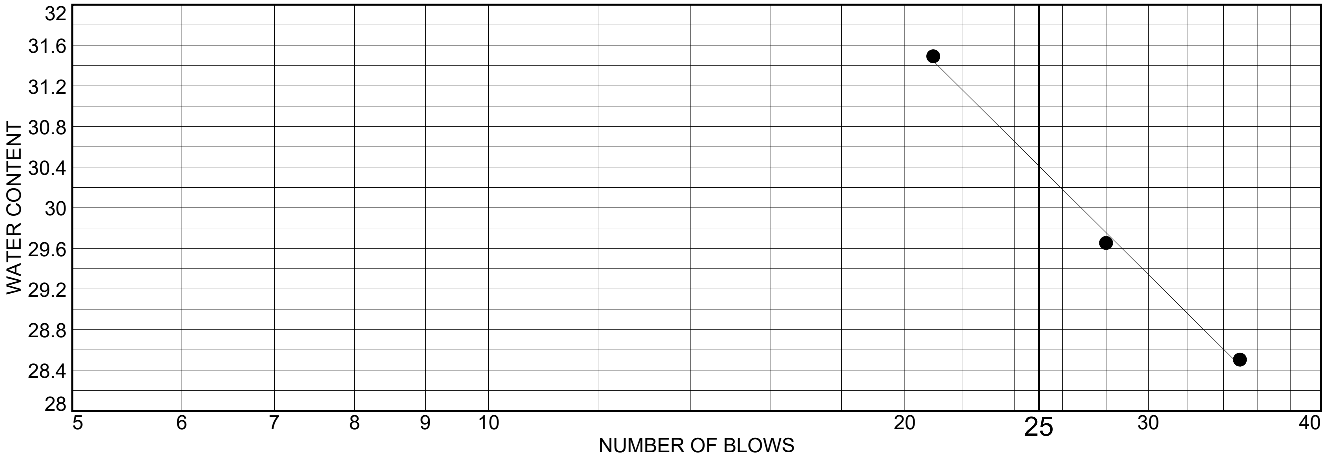
Liquid Limit, Plastic Limit, and Plasticity Index

Figure **B-2**
Sheet **1 of 1**

HA ATTERBERG LIMITS - STD - \\HALEYALDRICH.COM\SHARE\PDX_DATA\GEOMATICS\GINT\HC_LIBRARY.GLB - 12/16/25 14:12 - \\HALEYALDRICH.COM\SHARE\CF\PROJECTS\0210659\GINT\0210659-004_GINT.GPJ - mschweitzer

LIQUID AND PLASTIC LIMITS TEST REPORT D4318





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Grayish brown clay with sand	30	14	16			CL

Project No. 2831-097.0 **Client:** Haley & Aldrich, Inc.

Project: 90 Minto Rd.

02010659-003

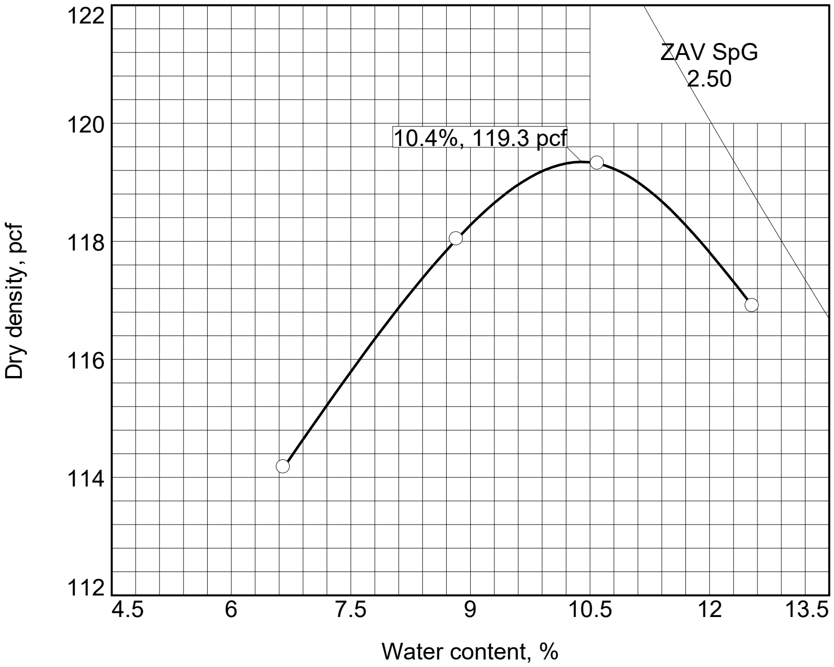
● **Source of Sample:** Onsite (IT-1) **Depth:** Bulk (0-2') **Sample Number:** 1

Remarks:

Figure

Tested By: JH **Checked By:** JH

COMPACTION TEST REPORT



Curve No.
G-69791

Test Specification:
ASTM D 1557-12 Method B Modified

Hammer Wt.: 10 lb.
Hammer Drop: 18 in.
Number of Layers: five
Blows per Layer: 25
Mold Size: 0.03333 cu. ft.
Test Performed on Material Passing 3/8 in. **Sieve**

Soil Data

NM ______ **Sp.G.** ______
LL ______ **PI** ______
%>3/8 in. 0.1 **%<#200** ______
USCS ______ **AASHTO** ______

TESTING DATA

	1	2	3	4	5	6
WM + WS	3932.0	3985.0	3979.0	3831.0		
WM	1990.0	1990.0	1990.0	1990.0		
WW + T #1	568.3	554.2	579.8	547.8		
WD + T #1	522.2	501.1	515.2	513.6		
TARE #1	0.0	0.0	0.0	0.0		
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	8.8	10.6	12.5	6.7		
DRY DENSITY	118.0	119.3	116.9	114.2		

TEST RESULTS

Maximum dry density = 119.3 pcf

Optimum moisture = 10.4 %

Material Description

Brown sandy clay

Project No. 2831-097.0 **Client:** Haley & Aldrich, Inc.
Project: 90 Minto Rd. Watsonville
02010659-003
○ **Source of Sample:** Onsite **Depth:** Bulk **Sample Number:** 1

Remarks:

Figure

Wet Prep
Mechanical Rammer Used

Tested By: BF **Checked By:** JH

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

13 November, 2024

Job No. 2411013
Cust. No. 13349

Mr. Micah Hintz
Haley & Aldrich
6420 S. Macadam Avenue, Suite 100
Portland, OR 97239

Subject: Project No.: 0210659-003
Project Name: New Leaf 90 Minto Rd Watsonville
Corrosivity Analysis –ASTM Test Methods

Dear Mr. Hintz:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on November 07, 2024. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

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

The chloride ion concentration reflects none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentration reflects none detected with a reporting limit of 15 mg/kg.

The pH of the soil is 6.91, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 280-mV and is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

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

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

Very truly yours,
CERCO ANALYTICAL, INC.
Juli Ch for
J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

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

Client:	Haley & Aldrich, Inc.
Client's Project No.:	0210659-003
Client's Project Name:	New Leaf 90 Minto Rd Watsonville
Date Sampled:	1-Nov-24
Date Received:	7-Nov-24
Matrix:	Soil
Authorization:	Signed Chain of Custody

Date of Report: 13-Nov-2024

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2411013-001	CPT-2 @ 0-5'	280	6.91	-	2,100	-	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	13-Nov-2024	8-Nov-2024		12-Nov-2024		8-Nov-2024	8-Nov-2024

* Results Reported on "As Received" Basis

N.D. - None Detected

Julia Clauson
Chemist

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

Chain of Custody

2411013

Page of

1100 Willow Pass Court
Concord, CA 94520-1005
925 462-2771
Fax: 925 462-2775
www.cercoanalytical.com

CERCO analytical

Job No.	CU#		Client Project I.D.	Schedule	Date Sampled	Date Due
0210659-003	13349		New Leaf 90 Minto Rd Watsonville	Analyte	11/1/2024	

Full Name
Micah Hintz

Phone X

Fax

Company and/or Mailing Address
Haley & Aldrich, 6420 S Macadam Ave # 100, Portland, OR 97239

Cell
503-310-3821 ☒

Sample Source
Bulk Sample CPT-2

ANALYSIS

ASTM w/Brief Evaluation

Lab No.	Sample I.D.	Date	Time	Matrix	Contain.	Size	Preserv.	Qty.	Redox Potential	pH	Sulfate	Chloride	Resistivity-100% Saturated		Brief Evaluation							
1	CPT-2 @0-5'								x	x	x	x	x		x							

MATRIX
- DW - Drinking Water
- GW - Ground Water
- SW - Surface Water
- WW - Waste Water
- SL - Sludge
- S - Soil
- Product

ABBREVIATIONS
- HB - Hosebib
- PV - Petcock Valve
- PT - Pressure Tank
- PH - Pump House
- RR - Restroom
- GL - Glass
- PL - Plastic
- ST - Sterile

SAMPLE RECEIPT

Total No. of Containers	
Rec'd Good Cond/Cold	
Conforms to Record	
Temp. a t Lab -°C	
Sampler	

Relinquished By:	Date 11-7-2024	Time 1459
Received By: Nicole Driep	Date 11/7/24	Time 14:59
Relinquished By:	Date	Time
Received By:	Date	Time
Relinquished By:	Date	Time
Received By:	Date	Time

Comments:
THERE IS AN ADDITIONAL CHARGE FOR EXTRUDING SOIL FROM TUBES

Email Address: MHintz@haleyaldrich.com

Geotherm

21239 FM529 Rd., Bldg F
Cypress, TX 77433
Office: 291-985-9344
info@geothermusa.com
http://www.geothermusa.com

November 22, 2024

Haley & Aldrich, Inc.
6420 S. Macadam Avenue, Suite 100
Portland, Oregon 97239-3517
Attn: Micah Hintz, PE, GE

Re: Thermal Analysis of Native Soil Sample (Project No. 0210659-003)
New Leaf 90 Minto Rd. Watsonville BESS – Watsonville, CA

The following is the report of thermal dryout characterization tests conducted on one (1) bulk sample of native soil from the referenced project sent to our laboratory.

Thermal Resistivity Tests: The sample was tested at the 'optimum' moisture content and at 95% of the standard Proctor dry density ***provided by Haley & Aldrich***. The tests were conducted in accordance with the **IEEE standard 442-2017**. The results are tabulated below and the thermal dryout curve is presented in **Figure 1.**

Sample ID, Description, Thermal Resistivity, Moisture Content and Density

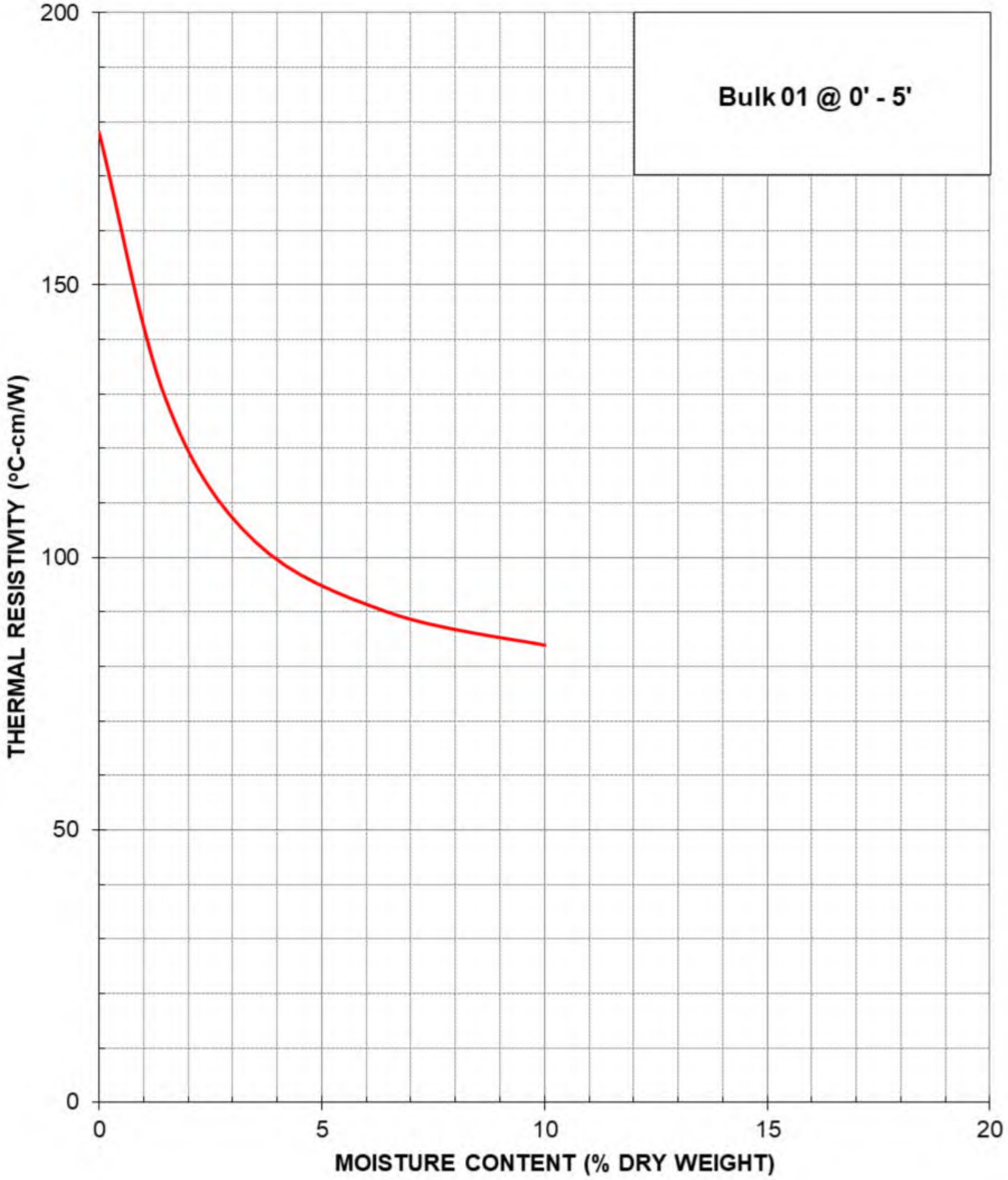
Sample ID	Depth (ft)	Effort (%)	Description (Haley & Aldrich)	Thermal Resistivity (°C-cm/W)		Moisture Content (%)	Dry Density (lb/ft^3)
				Wet	Dry		
Bulk 01	0 - 5	90	Brown Sandy Clay	84	178	10	107

Please contact us if you have any questions or if we can be of further assistance.

Geotherm USA, LLC

Nimesh Patel

THERMAL DRYOUT CURVE



Haley & Aldrich, Inc. (Project No. 0210659-003)

New Leaf 90 Minto Rd Watsonville BESS – Watsonville, CA

Thermal Analysis of Native Soil Sample

November 2024 **Figure 1**

APPENDIX C
Infiltration Test Data

HALEY ALDRICH

INFILTRATION TEST DATA

File No.:	0210659-003
Sheet:	1 of 1
Date:	1-Nov-2024
Field Rep.	W. Passman

Client: New Leaf Energy
Project: 90 Minto Road, Watsonville
Subject: Infiltration Test

Boring/Test Number	IT-1		Length of Casing	60	inches
Diameter of Boring	4	inches	Depth of Casing Below Ground Surface	3.00	feet
Diameter of Casing	2	inches	Depth to Initial Water from top of casing (d1)	24	inches

Reading Number	Time Start/End	Elapsed Time (min)	Final Water Depth (in)	Water Drop (in)	Direct Percolation Rate (in/hr)
1	1030 1100	30	22.00	2.00	4.00
2	1100 1130	30	23.00	1.40	2.80
3	1130 1200	30	22.50	0.60	1.20
4	1200 1230	30	23.00	0.60	1.20
5	1230 1300	30	23.00	0.60	1.20
6	1300 1330	30	24.60	0.60	1.20
7	1330 1400	30	24.40	0.40	0.80
8	1400 1430	30	24.40	0.40	0.80
9	1430 1500	30	24.40	0.40	0.80
10	1500 1530	30	24.40	0.40	0.80
11	1530 1600	30	24.40	0.40	0.80
12	1600 1630	30	24.40	0.40	0.80
				Average of Last 3 Readings:	0.80

Reduction Factors

RF_t =	2	*(test method factor)*
RF_v =	1.5	*(low site variability, low number of tests)*
RF_s =	2	*(moderate long-term siltation)*
RF_{total} =	**5.5**	***(sum of RF values)***

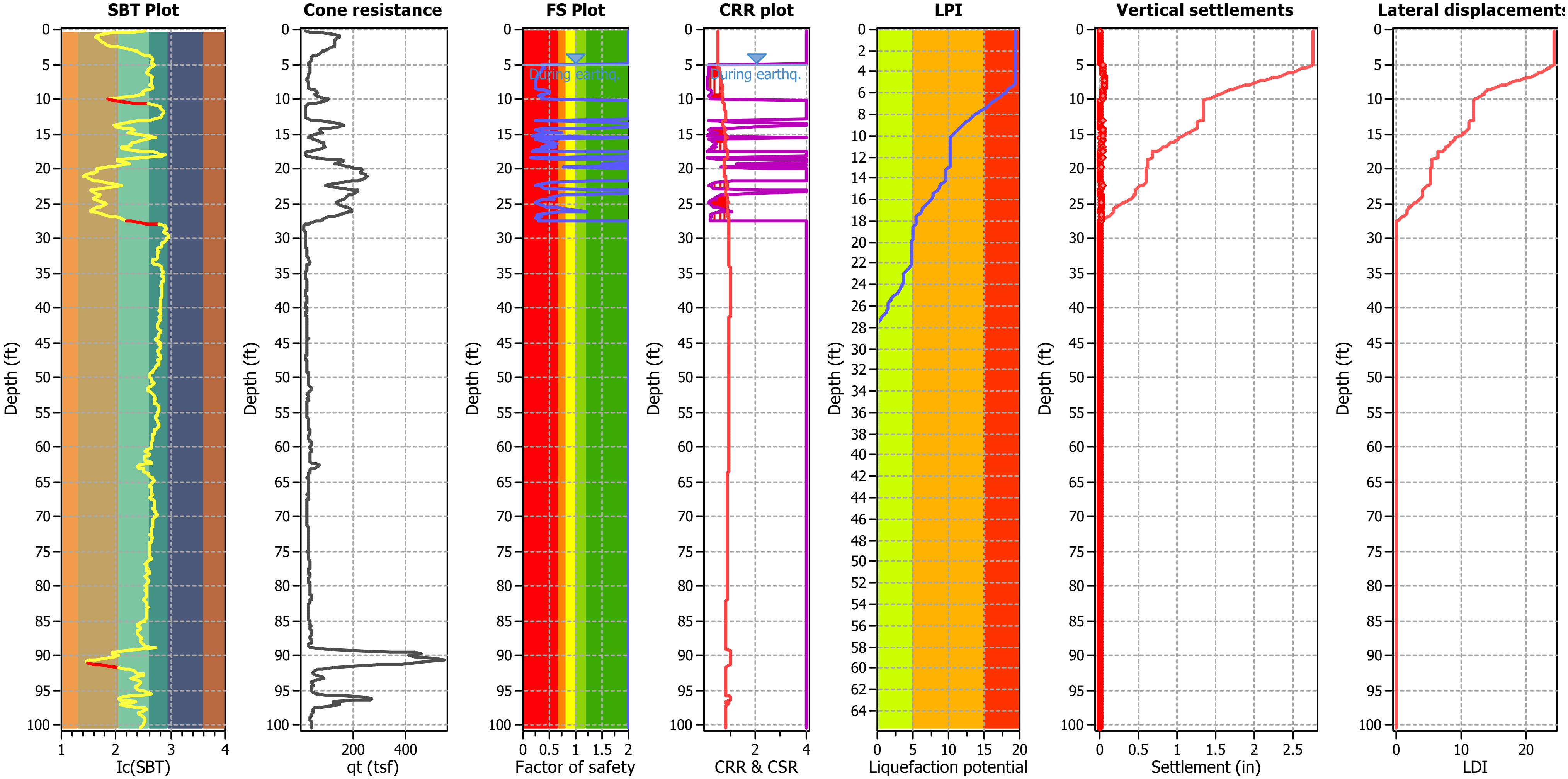
Design Percolation Rate

Unadjusted Percolation Rate (P_R) =	0.80	in/hour *(average of last three)*
Design Percolation Rate (P_R / Cf_{total}) =	0.15	in/hour
	(1.1E-04	*cm/s)*

APPENDIX D
Liquefaction

Project: **New Leaf - Seahawk**
Location: **90 Minto Road, Watsonville, CA**

CPT: CPT-1
Total depth: 100.39 ft

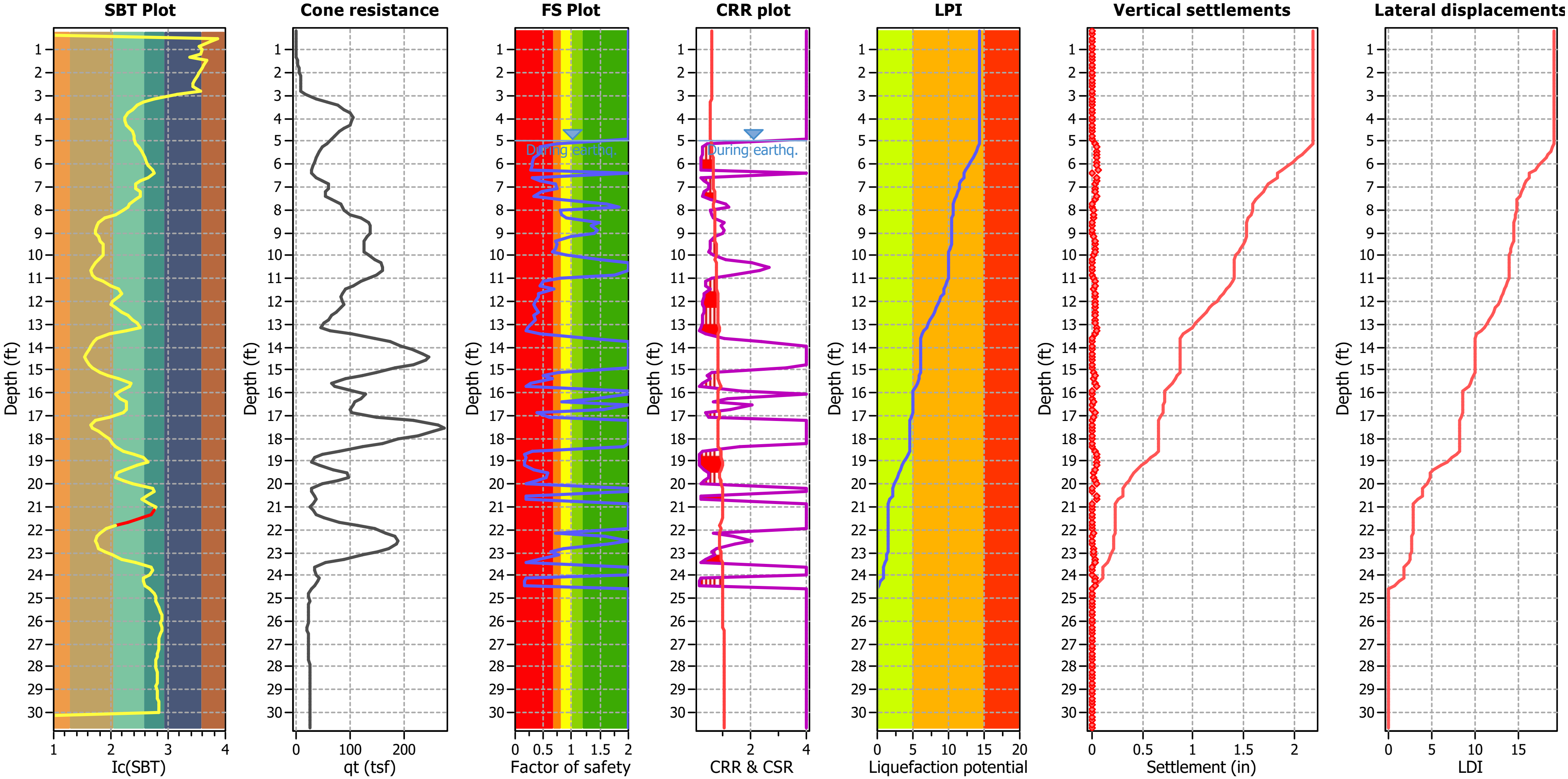


Analysis method:	B&I (2014)	G.W.T. (in-situ):	10.20 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	Yes
Earthquake magnitude M_w:	7.23	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	50.00 ft
Peak ground acceleration:	1.03	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

Haley & Aldrich
Geotechnical Engineers
2017 N First Street #380, San Jose, CA 95131
https://www.haleyaldrich.com/

Project: New Leaf - Seahawk
Location: 90 Minto Road, Watsonville, CA

CPT: CPT-2
Total depth: 30.68 ft



Analysis method: B&I (2014)
Fines correction method: B&I (2014)
Points to test: Based on Ic value
Earthquake magnitude M_w: 7.23
Peak ground acceleration: 1.03

G.W.T. (in-situ): 5.20 ft
G.W.T. (earthq.): 5.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT

Use fill: No
Fill height: N/A
Fill weight: N/A
Trans. detect. applied: Yes
K_σ applied: Yes

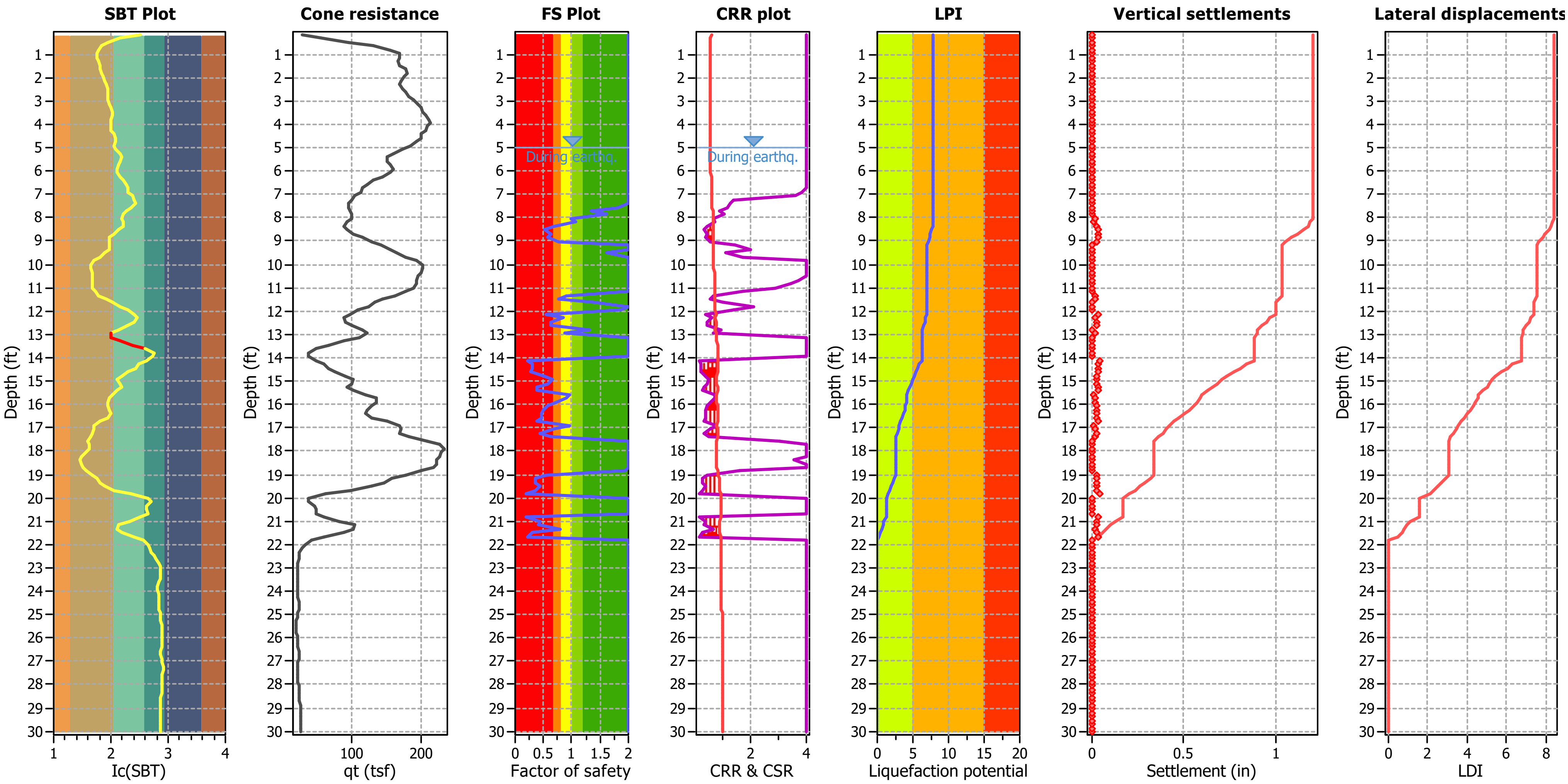
Clay like behavior applied: .
Limit depth applied: No
Limit depth: N/A
MSF method: Method based

Haley & Aldrich
Geotechnical Engineers
2017 N First Street #380, San Jose, CA 95131
https://www.haleyaldrich.com/

Project: **New Leaf - Seahawk**
Location: **90 Minto Road, Watsonville, CA**

CPT: CPT-3

Total depth: 30.02 ft

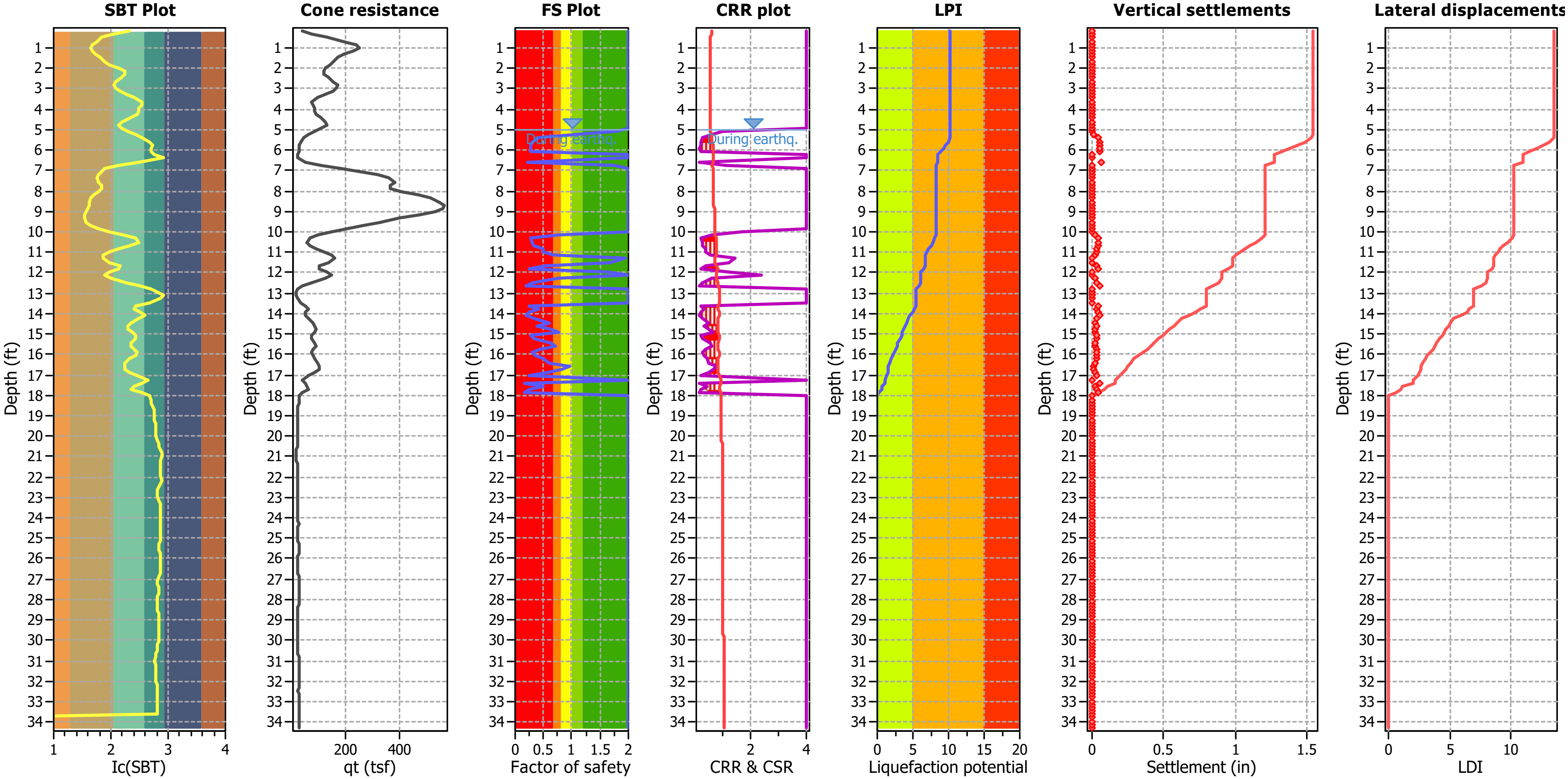


Analysis method:	B&I (2014)	G.W.T. (in-situ):	13.70 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.23	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	1.03	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Haley & Aldrich
Geotechnical Engineers
2017 N First Street #380, San Jose, CA 95131
https://www.haleyaldrich.com/

Project: **New Leaf - Seahawk**
Location: **90 Minto Road, Watsonville, CA**

CPT: CPT-4
Total depth: 34.28 ft

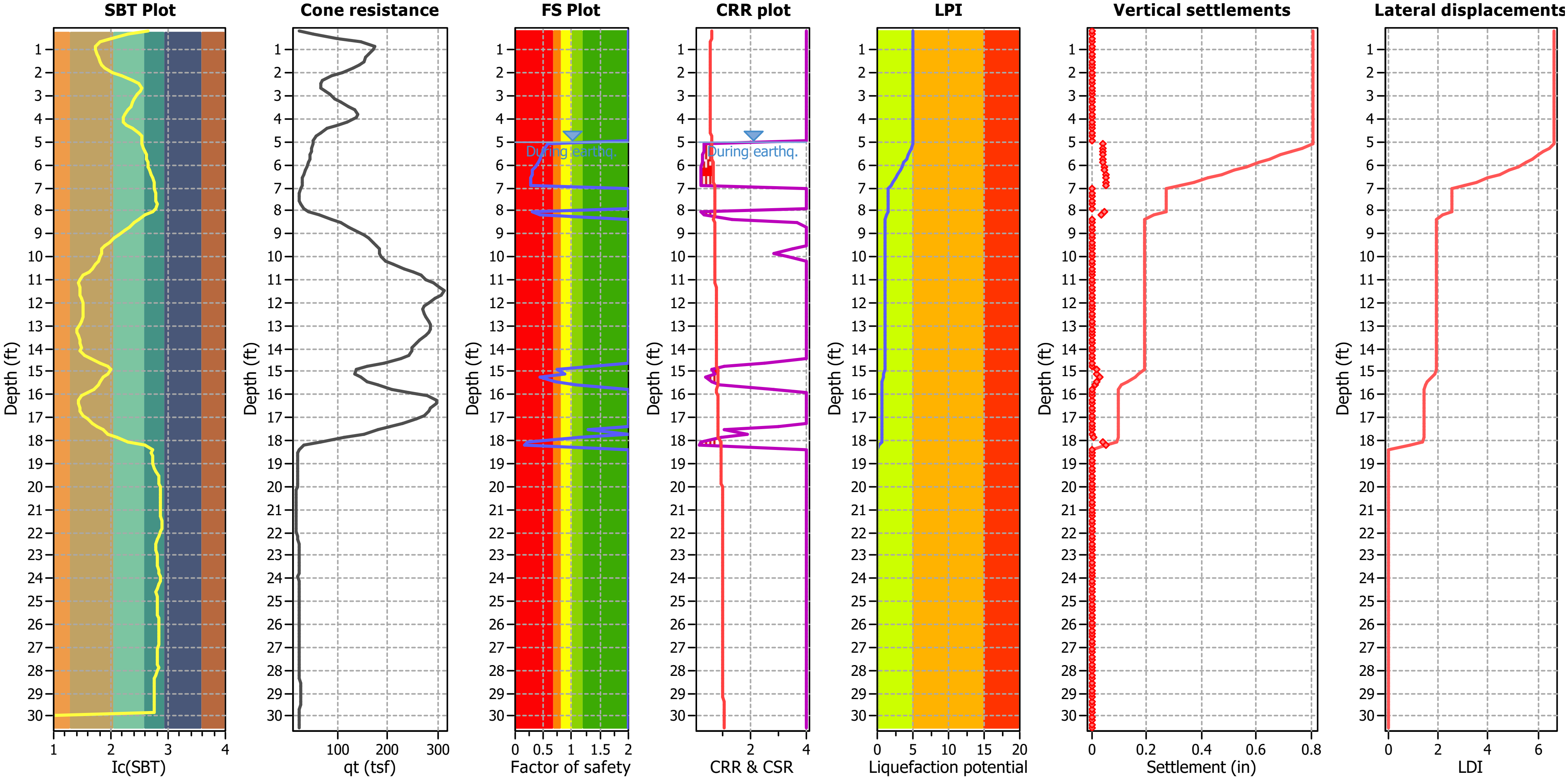


Analysis method:	B&I (2014)	G.W.T. (in-situ):	12.00 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.23	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	1.03	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Haley & Aldrich
Geotechnical Engineers
2017 N First Street #380, San Jose, CA 95131
https://www.haleyaldrich.com/

Project: **New Leaf - Seahawk**
Location: **90 Minto Road, Watsonville, CA**

CPT: CPT-5
Total depth: 30.51 ft

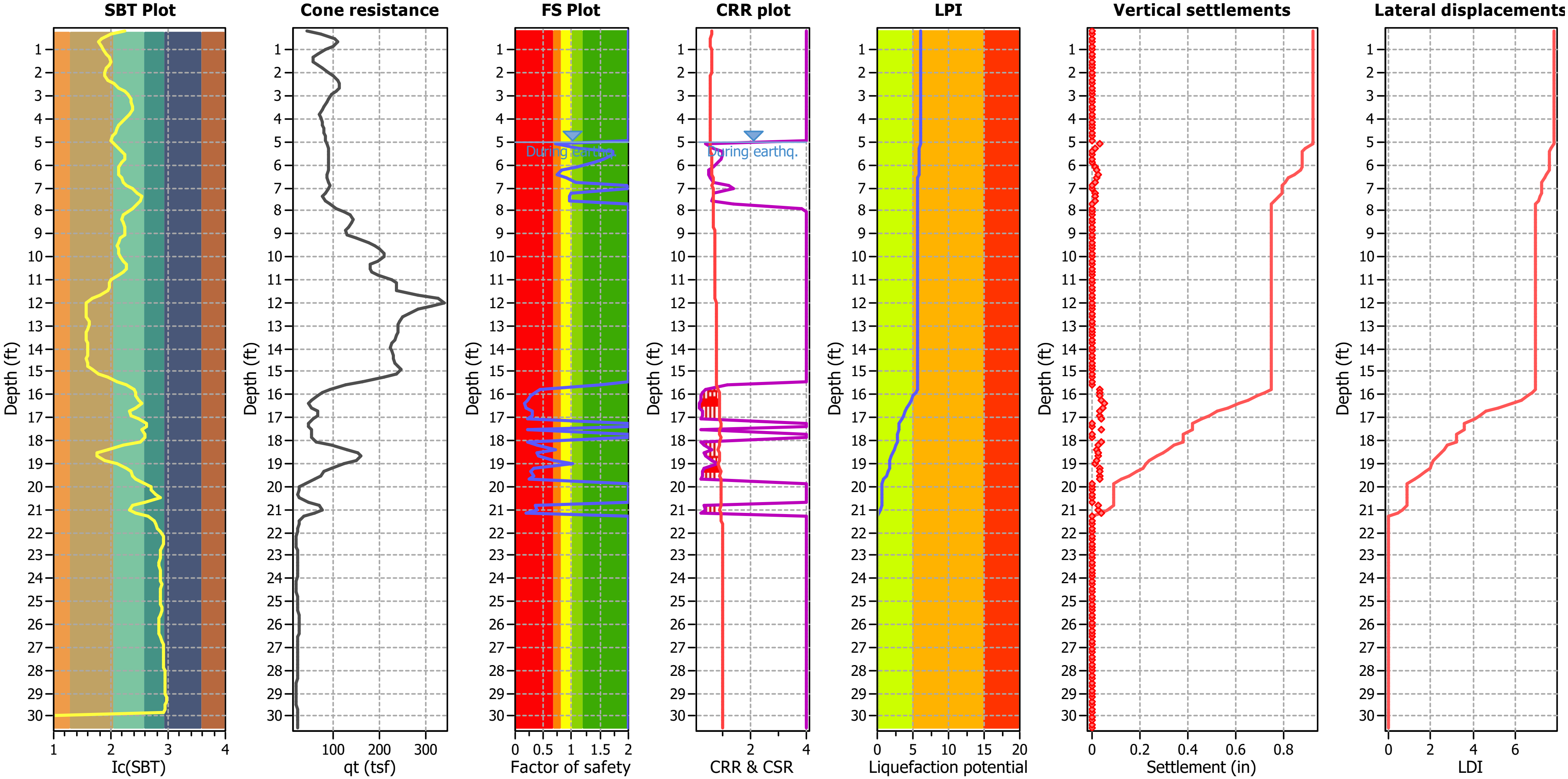


Analysis method:	B&I (2014)	G.W.T. (in-situ):	11.60 ft	Use fill:	No	Clay like behavior	
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	applied:	.
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	7.23	Ic cut-off value:	2.60	Trans. detect. applied:	No	Limit depth:	N/A
Peak ground acceleration:	1.03	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

Haley & Aldrich
Geotechnical Engineers
2017 N First Street #380, San Jose, CA 95131
https://www.haleyaldrich.com/

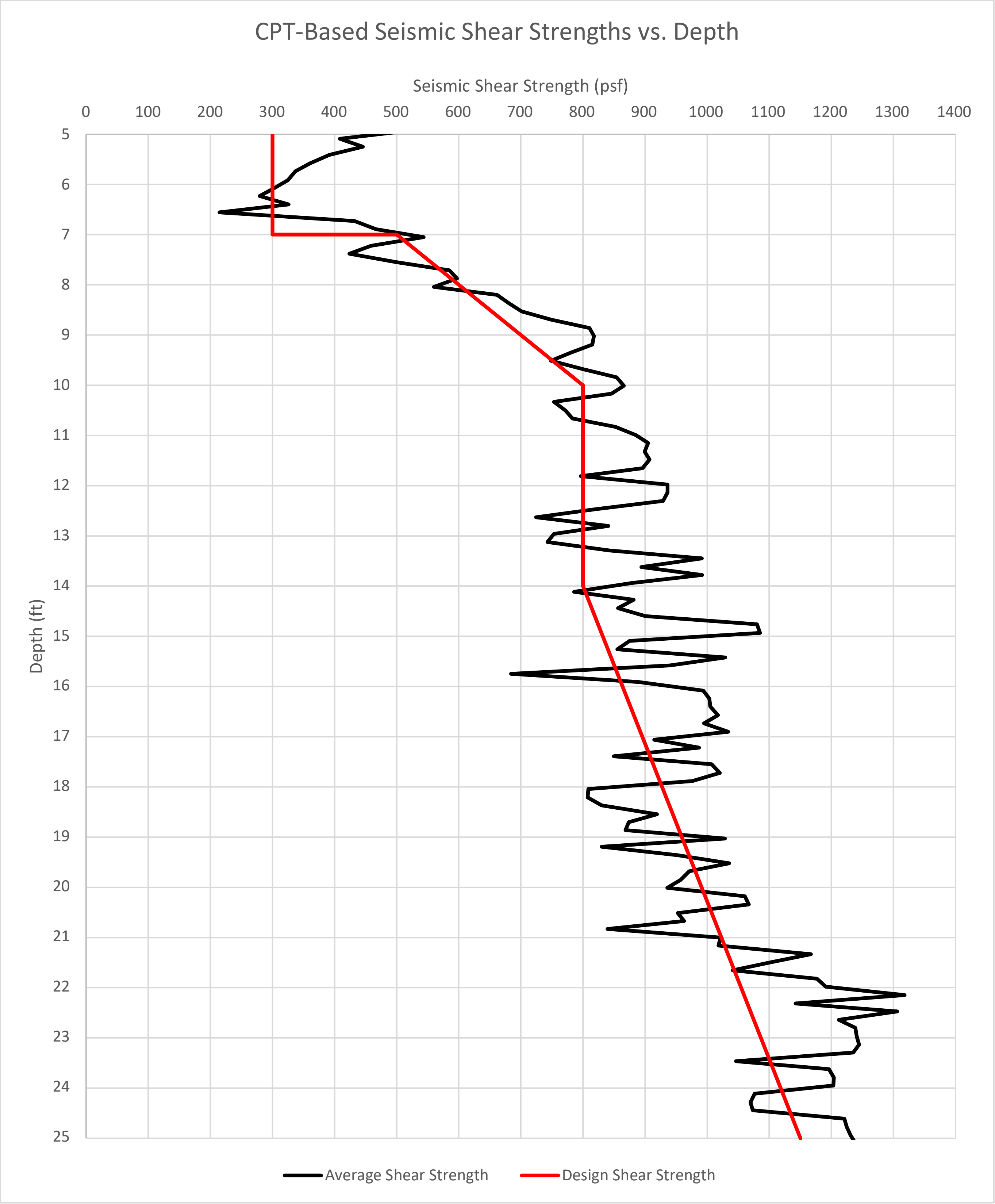
Project: **New Leaf - Seahawk**
Location: **90 Minto Road, Watsonville, CA**

CPT: CPT-6
Total depth: 30.51 ft



Analysis method:	B&I (2014)	G.W.T. (in-situ):	16.00 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	7.23	Ic cut-off value:	2.60	Trans. detect. applied:	No	Limit depth:	N/A
Peak ground acceleration:	1.03	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

NEW LEAF WATSONVILLE
0210659-004

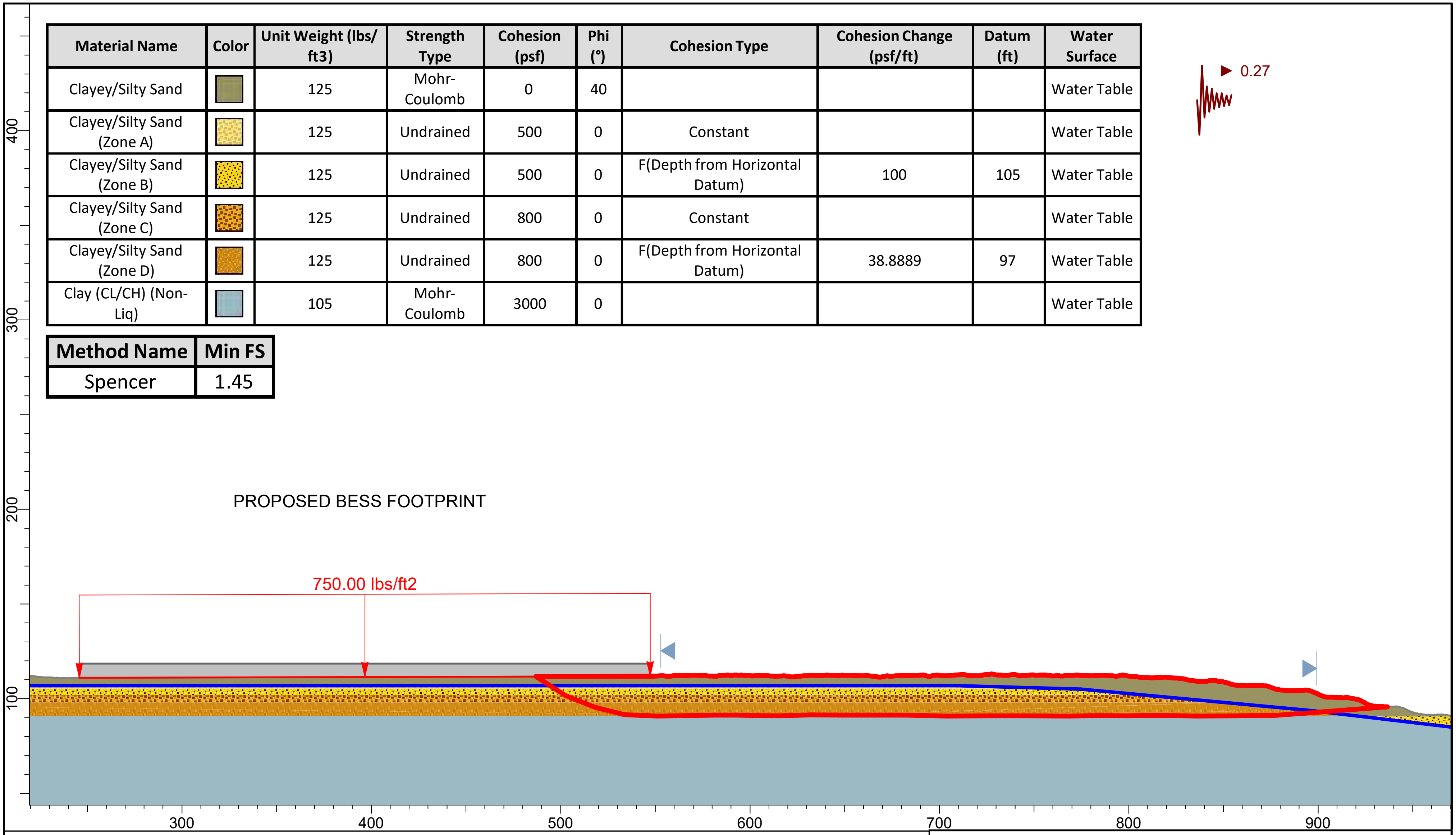


APPENDIX E

Slope Stability

Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (°)	Cohesion Type	Cohesion Change (psf/ft)	Datum (ft)	Water Surface
Clayey/Silty Sand	🟫	125	Mohr-Coulomb	0	40				Water Table
Clayey/Silty Sand (Zone A)	🟨	125	Undrained	500	0	Constant			Water Table
Clayey/Silty Sand (Zone B)	🟨	125	Undrained	500	0	F(Depth from Horizontal Datum)	100	105	Water Table
Clayey/Silty Sand (Zone C)	🟧	125	Undrained	800	0	Constant			Water Table
Clayey/Silty Sand (Zone D)	🟧	125	Undrained	800	0	F(Depth from Horizontal Datum)	38.8889	97	Water Table
Clay (CL/CH) (Non-Liq)	🟦	105	Mohr-Coulomb	3000	0				Water Table

Method Name	Min FS
Spencer	1.45



SEE APPENDIX D FOR A GRAPH OF DESIGN RESIDUAL SHEAR STRENGTHS

HALEY ALDRICH

NEW BATTERY ENERGY STORAGE SYSTEM INSTALLATION
90 MINTO ROAD
WATSONVILLE, CALIFORNIA

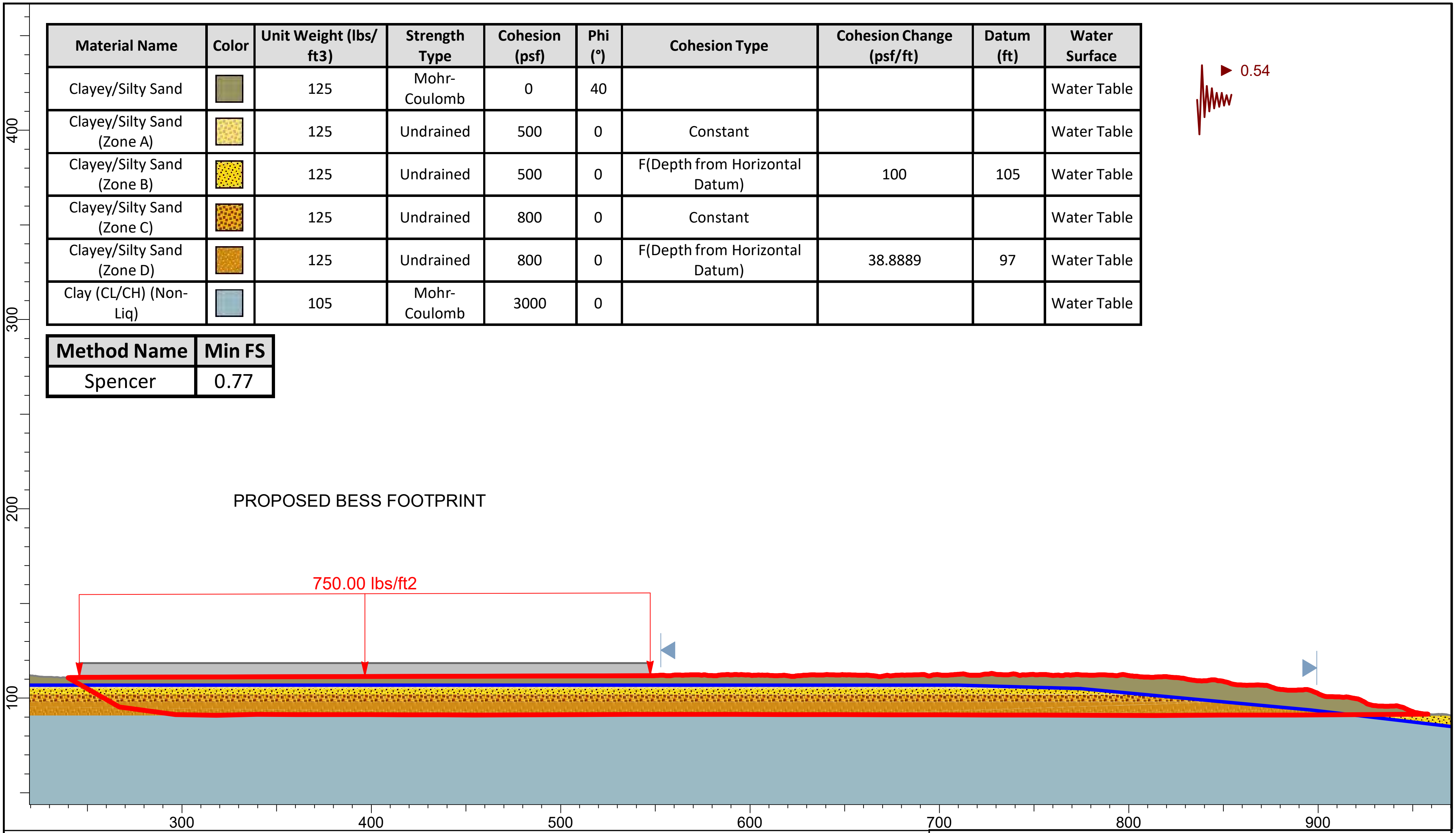
SLOPE STABILITY ANALYSIS
SECTION A-A'
SEISMIC AT KH = 1/4 PGA

SCALE: AS SHOWN
DECEMBER 2025

FIGURE E1

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Cohesion Type	Cohesion Change (psf/ft)	Datum (ft)	Water Surface
Clayey/Silty Sand	▨	125	Mohr-Coulomb	0	40				Water Table
Clayey/Silty Sand (Zone A)	▨	125	Undrained	500	0	Constant			Water Table
Clayey/Silty Sand (Zone B)	▨	125	Undrained	500	0	F(Depth from Horizontal Datum)	100	105	Water Table
Clayey/Silty Sand (Zone C)	▨	125	Undrained	800	0	Constant			Water Table
Clayey/Silty Sand (Zone D)	▨	125	Undrained	800	0	F(Depth from Horizontal Datum)	38.8889	97	Water Table
Clay (CL/CH) (Non-Liq)	▨	105	Mohr-Coulomb	3000	0				Water Table

Method Name	Min FS
Spencer	0.77



SEE APPENDIX D FOR A GRAPH OF DESIGN RESIDUAL SHEAR STRENGTHS

HALEY ALDRICH

NEW BATTERY ENERGY STORAGE SYSTEM INSTALLATION
90 MINTO ROAD
WATSONVILLE, CALIFORNIA

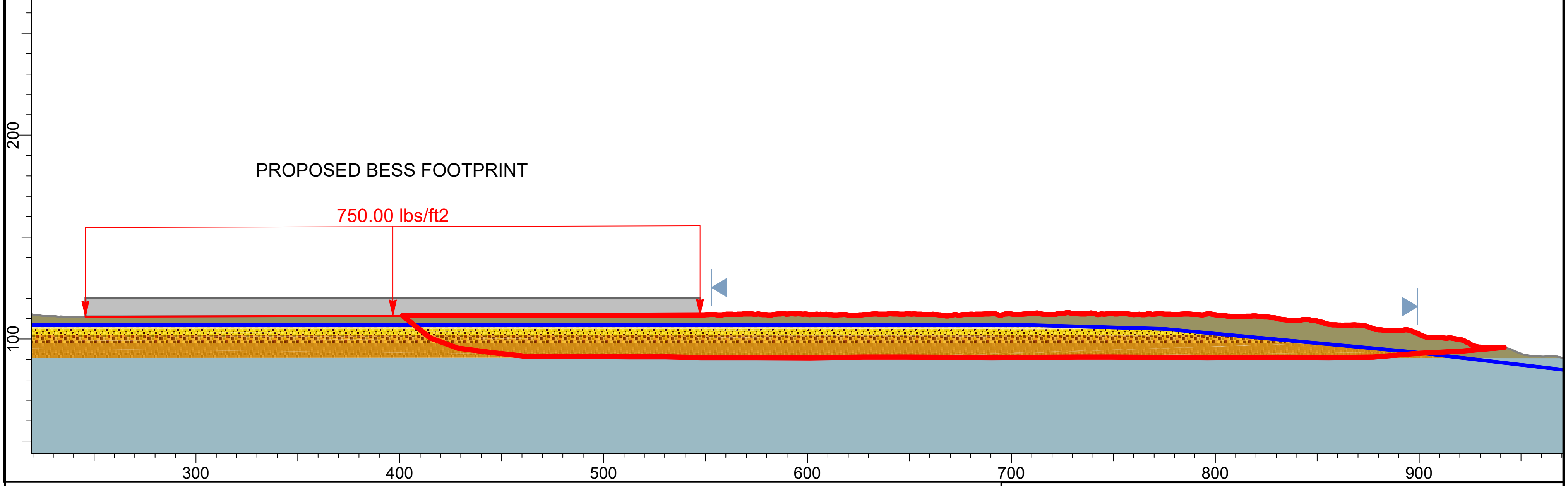
SLOPE STABILITY ANALYSIS
SECTION A-A'
SEISMIC AT KH = 1/2 PGA

SCALE: AS SHOWN
DECEMBER 2025

FIGURE E2

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Cohesion Type	Cohesion Change (psf/ft)	Datum (ft)	Water Surface
Clayey/Silty Sand	🟫	125	Mohr-Coulomb	0	40				Water Table
Clayey/Silty Sand (Zone A)	🟨	125	Undrained	500	0	Constant			Water Table
Clayey/Silty Sand (Zone B)	🟨	125	Undrained	500	0	F(Depth from Horizontal Datum)	100	105	Water Table
Clayey/Silty Sand (Zone C)	🟫	125	Undrained	800	0	Constant			Water Table
Clayey/Silty Sand (Zone D)	🟧	125	Undrained	800	0	F(Depth from Horizontal Datum)	38.8889	97	Water Table
Clay (CL/CH) (Non-Liq)	🟦	105	Mohr-Coulomb	3000	0				Water Table

Method Name	Min Ky
Spencer	0.41



SEE APPENDIX D FOR A GRAPH OF DESIGN RESIDUAL SHEAR STRENGTHS

HALEY ALDRICH

NEW BATTERY ENERGY STORAGE SYSTEM INSTALLATION
90 MINTO ROAD
WATSONVILLE, CALIFORNIA

SLOPE STABILITY ANALYSIS
SECTION A-A'
SEISMIC YIELD ACCELERATION

SCALE: AS SHOWN
DECEMBER 2025

FIGURE E3

APPENDIX F

Report of Geologic Fault Investigation

BAYSIDE GEOLOGY
ENGINEERING GEOLOGISTS

GEOLOGIC FAULT INVESTIGATION
NEW LEAF ENERGY INC. PROPERTY
90 MINTO ROAD
WATSONVILLE, CALIFORNIA
SANTA CRUZ COUNTY APN 051-101-77 & 78

Job No. 24017-SC
August 1, 2025

202 Anita Street
Santa Cruz, California 95060

jim@baysidegeology.com
(831) 234-7825

BAYSIDE GEOLOGY
ENGINEERING GEOLOGISTS

August 1, 2025
Job No. 24017-SC

New Leaf Energy, Inc.
55 Technology Drive, Suite 102
Lowell, Massachusetts 01851

ATTENTION: William Peregoy

SUBJECT: **GEOLOGIC FAULT INVESTIGATION**
Proposed Battery Energy Storage System Facility
90 Minto Road Watsonville, California
Santa Cruz County APN 051-101-77 & 78

Dear Mr. Peregoy:

As requested, we have completed our geologic fault evaluation of the above-referenced property. The intent of our investigation was to provide a detailed subsurface trenching evaluation of the potential for tectonic faulting relative to the development of the proposed Battery Energy Storage System (BESS) facility. This report details the findings and conclusions from our literature review, field exploration, and geologic analysis.

It is our opinion that the proposed development will be subject to an "ordinary" level of risk relative to the tectonic faulting hazard, provided our recommendations and the recommendations of your project geotechnical and civil engineers are followed.

Please contact us if you have any questions regarding this report.

Sincerely,
BAYSIDE GEOLOGY

James A. Olson
Principal Geologist
Professional Geologist #7244
Certified Engineering Geologist #2267

Copies: Addressee (electronic copy)

202 Anita Street
Santa Cruz, California 95060
jim@baysidegeology.com
(831) 234-7825

TABLE OF CONTENTS

NOTE: Plates must accompany text of report in order for report to be considered complete.

INTRODUCTION

This report presents the results of our evaluation of the New Leaf Energy Inc. property (APN 051-101-77 & 78). The property is located southwest of the Santa Cruz Mountains in Watsonville, California (Figure 1; Site Location Map). The north west corner of the site is currently occupied with a large barn and several smaller structures. The bulk of the property is in production as an apple orchard. It is our understanding that the proposed project consists of the development of a BESS facility.

Previously, a Geologic Hazards Assessment (GHA) for the proposed project was completed by Haley & Aldrich, Inc. (2024). They identified surface fault rupture, expansive soils, liquefaction induced settlement and lateral spreading (landsliding), and seismic shaking as hazards that present a potential risk to the proposed development. Review of the GHA by the Santa Cruz County Geologist concurred with their finding (Santa Cruz County, 2024). As part of the conditions for the development of the proposed project, Santa Cruz County has required a focused geologic fault investigation performed by the Project Geologist. They have not required a full geologic investigation typical for development of habitable space. The additional geologic hazards of expansive soils, liquefaction induced settlement and lateral spreading (landsliding), and seismic shaking that present a potential risk to the proposed project are required to be evaluated by the Project Geotechnical Engineer.

In accordance with your authorization, we have completed a detailed, subsurface trenching investigation of the potential for tectonic fault rupture to impact the proposed project. We have not evaluated the risk to the proposed project by the additional identified hazards of expansive soils, liquefaction induced settlement and lateral spreading (landsliding), and seismic shaking.

The scope of work performed for this evaluation included: 1) review of published and unpublished literature relevant to the site and vicinity; 2) analysis of stereo-paired, aerial photographs; 3) review of Google Earth aerial imagery; 4) reprocessing and review of Santa Cruz County LiDAR imagery of the property; 5) geologic reconnaissance mapping of the site; 6) coordination and discussion of our proposed subsurface exploration plan with the Santa Cruz County Geologist; 7) excavation and logging of approximately 1,300 feet of exploratory fault trench; 8) compilation and analysis of the resulting data; and 9) preparation of this report and accompanying illustrations.

REGIONAL GEOLOGIC SETTING

The subject property is located on the southwest margin of the Santa Cruz Mountains of the California Coast Ranges (Figure 1). The Santa Cruz Mountains are formed by a series of rugged, linear ridges and valleys following the pronounced northwest to southeast structural grain of central California geology. Underlying the Santa Cruz Mountains is a large, elongate prism of granitic and metamorphic basement rocks. These rocks are separated from contrasting basement rock types to the northeast and southwest by the San Andreas and Monterey Bay/Tularcitos strike-slip fault systems, respectively. Overlying the granitic basement rocks is a sequence of dominantly marine sedimentary rocks of Paleocene to Pliocene age and non-marine sediments of Pliocene to Pleistocene age (Figure 2).

Throughout the Cenozoic Era, this portion of California was dominated by tectonic forces associated with lateral or "transform" motion between the North American and Pacific crustal plates, producing long, northwest-trending faults such as the San Andreas, with horizontal displacements measured in tens to hundreds of miles. Accompanying the northwest direction of the horizontal (strike-slip) movement of the plates were episodes of compressive stress, causing repeated uplift, deformation, erosion and subsequent redeposition of sedimentary rocks. This tectonic deformation is most evident in sedimentary rocks older than the middle Miocene, and consists of steeply dipping folds, overturned bedding, faulting, jointing, and fracturing. Along the coast, the ongoing tectonic activity is most evident in the formation of a series of uplifted marine terraces. The Loma Prieta earthquake of 1989 and its aftershocks are the most recent reminders of the geologic unrest in the region.

REGIONAL SEISMIC SETTING

California's broad system of strike-slip faulting has a long and complex history. Some of these faults present a seismic hazard to the subject property. The most important of these are the San Andreas, Zayante-Vergeles and San Gregorio faults (Figure 2). These faults are either active or considered potentially active (Buchanan-Banks et al., 1978; Burkland and Associates, 1975; Jennings et al., 1975; Greene, 1977; Hall et al., 1974; Schwartz et al., 1990; Wallace, 1990; and Working Group on Northern California Earthquake Potential [WGNCEP], 1996). Each fault is discussed below.

San Andreas Fault

The San Andreas fault is active and represents the major seismic hazard in northern California (Jennings et al., 1975; Buchanan-Banks et al., 1978; Hall et al., 1974). The main trace of the San Andreas fault trends northwest-southeast and extends over 700 miles from the Gulf of California through the Coast Ranges to Point Arena, where the fault extends offshore.

Geologic evidence suggests that the San Andreas fault has experienced right-lateral, strike-slip movement throughout the latter portion of Cenozoic time, with cumulative offset of hundreds of miles. Surface rupture during historical earthquakes, fault creep, and historical seismicity confirm that the San Andreas fault and its branches, the Hayward, Calaveras, and San Gregorio faults, are all active today.

Historical earthquakes along the San Andreas fault and its branches have caused significant seismic shaking in the Santa Cruz County area. The two largest historical earthquakes on the San Andreas to affect the area were the moment magnitude (M_w) 7.9 San Francisco earthquake of April 18, 1906 (actually centered near Olema) and the M_w 6.9 Loma Prieta earthquake of October 17, 1989. The San Francisco earthquake caused severe seismic shaking and structural damage to many buildings in Santa Cruz County. The Loma Prieta earthquake appears to have caused more intense seismic shaking than the 1906 event in localized areas of the Santa Cruz Mountains, even though its regional effects were not as extensive. There were also significant earthquakes in northern California along or near the San Andreas fault in 1838, 1865 and possibly 1890 (Sykes and Nishenko, 1984; Working Group on Northern California Earthquake Potential, 1996).

Geologists have recognized that the San Andreas fault system can be divided into segments with earthquakes of different magnitudes and recurrence intervals (Working Group on California Earthquake Probabilities, 1988 and 1990; Working Group on Northern California Earthquake Potential (WGNCEP), 1996). The most recent work has redefined the segments and defines 10 sections: Shelter Cove, North Coast, Peninsula, Santa Cruz Mountains, Creeping, Parkfield, Cholame-Carrizo, Mojave, San Bernardino Mountains, and Coachella sections (Bryant, 2000). In theory, the segments rupture independently or combine to produce earthquakes with larger magnitudes.

In summary, the San Andreas fault should be considered active. It is considered capable of generating a magnitude 8.1 earthquake (USGS, 2008) with a recurrence interval of 100-450 years (Bryant, 2000).

Zayante-Vergeles Fault

The Zayante fault lies west of the San Andreas fault and trends about 50 miles northwest from the Watsonville lowlands into the Santa Cruz Mountains. The southern extension of the Zayante fault, known as the Vergeles fault, merges with the San Andreas fault south of San Juan Bautista.

The Zayante fault has a long, well-documented history of vertical movement (Clark and Reitman, 1973), probably accompanied by right-lateral, strike-slip movement (Hall et al., 1974; Ross and Brabb, 1973). Stratigraphic and geomorphic evidence indicates the Zayante fault has undergone late Pleistocene and Holocene movement and is potentially active (Buchanan-Banks et al., 1978; Coppersmith, 1979).

Some historical seismicity may be related to the Zayante fault (Griggs, 1973). For instance, the Zayante fault may have undergone sympathetic fault movement during the 1906 earthquake centered on the San Andreas fault, although this evidence is equivocal (Coppersmith, 1979). Seismic records strongly suggest that a section of the Zayante fault approximately 3 miles long underwent sympathetic movement in the 1989 earthquake. The earthquake hypocenters tentatively correlated to the Zayante fault occurred at a depth of 5 miles; no instances of surface rupture on the fault have been reported.

In summary, the Zayante-Vergeles fault should be considered potentially active. It is considered capable of generating a magnitude 7.0 earthquake (USGS, 2008) with a recurrence interval of 3,130 years (Wesnousky, 1986).

San Gregorio Fault

The San Gregorio fault, as mapped by Greene (1977), Weber et al. (1979), Weber and Lajoie (1974), and Weber et al. (1995), skirts the coastline of Santa Cruz County northward from Monterey Bay and trends onshore at Point Año Nuevo. Northward from Año Nuevo, it passes offshore again, touching onshore briefly at Seal Cove just north of Half Moon Bay, and eventually connects with the San Andreas fault near Bolinas. Southward from Monterey Bay, it may trend onshore north of Big Sur (Greene, 1977) to connect with the Palo Colorado fault, or it may continue southward through Point Sur to connect with the Hosgri fault in south-central California. Based on these two proposed correlations, the San Gregorio fault zone has a length of at least 100 miles and possibly as much as 250 miles.

The on land exposures of the San Gregorio fault at Point Año Nuevo and Seal Cove show evidence of late Pleistocene displacement (Jennings, 1975; and Buchanan-Banks et al., 1978) and Holocene displacement (Weber and Cotton, 1981; Simpson et al., 1997). Although stratigraphic offsets indicate a history of horizontal and vertical displacements, the San Gregorio is considered predominantly right-lateral strike slip by most researchers (Greene, 1977; Weber and Lajoie, 1974; and Graham and Dickinson, 1978).

In addition to stratigraphic evidence for Holocene activity, the historical seismicity in the region is partially attributed to the San Gregorio fault (Greene, 1977). Due to inaccuracies of epicenter locations, even the magnitude the 6+ earthquakes of 1926, tentatively assigned to the Monterey Bay fault zone, may have actually occurred on the San Gregorio fault (Greene, 1977).

The WGNCEP (1996) divided the San Gregorio fault into the "San Gregorio" and "San Gregorio, Sur Region" segments. The segmentation boundary is located west of Monterey Bay, where the fault appears to have a right step-over (Figure 2). The San Gregorio segment is assigned a slip rate that results in a M_w 7.3 earthquake with a recurrence interval of 400 years. This value was assigned based on the preliminary results of a paleoseismic investigation at Seal Cove by Lettis and Associates (see Simpson et al., 1997) and on regional mapping by Weber et al. (1995). Simpson et al. (1997) discovered prior displacements consistent with a moment magnitude of 7 to 7¼ in their paleoseismic study at Seal Cove. The Sur Region segment is assigned a slip rate that results in a M_w 7.0 earthquake with an effective recurrence interval of 411 years. Within the Sur Region many geologists, including Greene (1977), map the San Gregorio fault zone as continuing along the Palo Colorado fault. Graham and Dickinson (1978) show the San Gregorio fault continuing along the Sur fault zone.

Bryant (2000) has adopted a model similar to the WGNCEP (1996). They split the fault into the San Gregorio section and the Sur Region section. The forecasted earthquake on the combined fault is a Mw 7.5 (USGS, 2008).

DESCRIPTION OF SITE AND VICINITY

The Site Location Map (Figure 1), Regional Geologic Map (Figure 2), USGS Geologic Map (Figure 3), State Fault Map (Alquist Priolo) (Figure 4), County Fault Map (Figure 5), Site Subsurface Exploration Map (Plate 1), and Trench Logs (Plates 2 thru 5) depict the relevant topographic and geologic information on the subject property. We initiated our trenching this past January and completed it in May and June (2025).

Geomorphology

The subject property is approximately 2,200 feet long and 1,000 feet wide, encompassing about 47 acres. Total relief across the site is about 20 feet and slopes are gentle. A northwest trending ridge and swale traverse the property. The morphology of the site is unaltered by grading.

The subject site is located within the Watsonville lowlands; a backfilled sedimentary basin. The morphology of the region is controlled by the combined forces of plate tectonics (faulting) with sedimentation/erosion driven by glacio-eustatic cycling (ice ages and sea-level change) [Dupré, 1975]. Over time, this has resulted in a complex assemblage of folded and faulted marine, fluvial (river), estuarine (lagon), alluvial fan (sheet-wash and debris flow) and eolian (dune) deposits within the basin. Most recently, as the sea-level has risen, entrenched rivers and streams have been backfilled with a sequence of marine to terrestrial deposits. The backfilling along Corralitos and Salsipuedes Creeks has dammed side streams, forming several of the lakes in the area (Pinto Lake, College Lake, etc.).

Earth Materials and Geologic Structure

The published geologic map of the area (Figure 3) shows the site and surrounding area as underlain by an Terrace deposits of Watsonville - Fluvial facies (Qwf). The formation is described as “Semiconsolidated, moderately to poorly sorted silt, sand, silty clay, and gravel. May be more than 200 ft thick...” (Brabb, 1997).

Our observations of the earth materials at the site are in general agreement with the published geologic map. Our subsurface exploratory trenching exposed a disturbed “A” soil horizon overlying interbedded fluvial subunits of the Fluvial (river) facies of the Terrace deposits of Watsonville (Qwf). We mapped 4 distinct subunits that were generally present across the entire site, although occasionally discontinuous. Bedding is approximately horizontal. Contacts between the subunits are over lapping and interfingered. Please see the trench logs (Plates 2 thru 5) for a detailed description and a graphic depiction of the fluvial subunits.

GEOLOGIC HAZARDS

Faulting

The north-eastern boundary of the property is located within the State of California Special Studies Zone (Alquist Priolo Act), for the Zayante Fault (Parrish, 2018; Gay, 1976; Davis, 1982). The entire property is located within the Santa Cruz County Fault Zone (Hall et al., 1974).

During our aerial photographic and LiDAR review we mapped a lineament within the swale that runs the length of the property. Hall et al., (1974) map this same lineament as a possible fault (Figure 5). To further evaluate the potential for faulting we advanced, scraped and logged about 1,300 feet of exploratory trench, distributed across 5 separate trenches (Plate 1). The trenches were spread across the proposed development. As required for critical structures, the length of the trenching incorporated a minimum of 100 feet of overlap on both ends given a +/-10-degree variation in regional fault orientation. We evaluated the trenches for evidence for faulting. The contacts between the terrace deposit subunits are distinct and continuous or over lapping along the entire length of the exploratory trenching. Overall, the exploratory trenching revealed no indication of tectonic faulting of any kind; no offset or truncation of units, no soil mixing or colluvial wedges, no continuous (through going) shear planes, no alignment of gravels, etcetera.

CONCLUSIONS

This report presents the results of our evaluation of the New Leaf Energy Inc. property (APN 051-101-77 & 78). The property is located southwest of the Santa Cruz Mountains in Watsonville, California (Figure 1; Site Location Map). The north west corner of the site is currently occupied with a large barn and several smaller structures. The bulk of the property is in production as an apple orchard. It is our understanding that the proposed project consists of the development of a BESS facility.

The property is located within both the Santa Cruz County Fault Zone and within the State of California Special Studies Zone (Alquist Priolo Act), for the Zayante Fault (Hall et al., 1974; Parrish, 2018; Gay, 1976; Davis, 1982). Our exploratory trenching revealed no indication of tectonic faulting of any kind. Given our evaluation, there is a low likelihood that the proposed development will be damaged by surface ground rupture due to tectonic faulting within the lifetime of the project.

We have not evaluated the risk to the proposed project by the additional identified hazards of expansive soils, liquefaction induced settlement and lateral spreading (landsliding), and seismic shaking.

It is our opinion that the proposed development will be subject to an “ordinary” level of risk (as defined in Appendix A) relative to the tectonic faulting hazard, provided our recommendations and the recommendations of your project geotechnical and civil engineers are followed. Appendix A should be reviewed in detail by the owner to determine whether this risk, as defined in the appendix, is acceptable. If this level of risk is unacceptable to the owner, then the risk should be further mitigated to an acceptable level.

RECOMMENDATIONS

1. The proposed BESS facility should be generally located as currently planned and as shown on our Site Subsurface Exploration Map (Plate 1).

2. Drainage from improved surfaces, such as walkways, patios, roofs and roads, should generally be dispersed while maintaining a natural path and remain within the basin of origin.

   Engineered, site-specific drainage and erosion control plans should be reviewed and approved by the project geologist and project geotechnical engineer. Proper drainage control design, implementation and maintenance throughout the lifetime of the development is critical.

3. We request the privilege of reviewing all geotechnical engineering reports and letters, civil engineering, drainage, and landscaping plans pertaining to the proposed development. If our firm is not afforded the privilege of providing these review services we can assume no responsibility for misinterpretation of our recommendations.

4. **We recommend that Bayside Geology be retained to observe any and all excavations.** Field observation must be provided by a representative of Bayside Geology

to enable us to form an opinion as to the degree of conformance of the site conditions exposed during excavation to those described in our geologic report. Any excavation performed without the full knowledge and direct observation of Bayside Geology, the project geologist of record, will render the recommendations of our report invalid.

INVESTIGATION LIMITATIONS

1. The conclusions and recommendations contained herein are based on probability and in no way imply that the proposed improvements will not possibly be subjected to ground failure, seismic shaking or faulting of such a magnitude that it overwhelms the site. The report does suggest that using the site for the proposed purpose in compliance with the recommendations contained herein represents an “ordinary” risk.

2. This report is issued with the understanding that it is the duty and responsibility of the owner or his representative or agent to ensure that the recommendations contained in this report are brought to the attention of the architect and engineers for the project, incorporated into the plans and specifications, and that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. If any unexpected variations in soil conditions or if any undesirable conditions are encountered during construction, Bayside Geology should be notified so that supplemental recommendations may be given.

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APPENDIX A

SCALE OF ACCEPTABLE RISKS FROM GEOLOGIC HAZARDS

SCALE OF ACCEPTABLE RISKS FROM SEISMIC GEOLOGIC HAZARDS

Risk Level	Structure Types	Extra Project Cost Probably Required to Reduce Risk to an Acceptable Level
Extremely low[1]	Structures whose continued functioning is critical, or whose failure might be catastrophic: nuclear reactors, large dams, power intake systems, plants manufacturing or storing explosives or toxic materials.	No set percentage (whatever is required for maximum attainable safety).
Slightly higher than under "Extremely low" level.[1]	Structures whose use is critically needed after a disaster: important utility centers; hospitals; fire, police and emergency communication facilities; fire station; and critical transportation elements such as bridges and overpasses; also dams.	5 to 25 percent of project cost.[2]
Lowest possible risk to occupants of the structure.[3]	Structures of high occupancy, or whose use after a disaster would be particularly convenient: schools, churches, theaters, large hotels, and other high rise buildings housing large numbers of people, other places normally attracting large concentrations of people, civic buildings such as fire stations, secondary utility structures, extremely large commercial enterprises, most roads, alternative or non-critical bridges and overpasses.	5 to 15 percent of project cost.[4]
An "ordinary" level of risk to occupants of the structure.[3,5]	The vast majority of structures: most commercial and industrial buildings, small hotels and apartment buildings, and single family residences.	1 to 2 percent of project cost, in most cases (2 to 10 percent of project cost in a minority of cases).[4]

[1] Failure of a single structure may affect substantial populations.

[2] These additional percentages are based on the assumptions that the base cost is the total cost of the building or other facility when ready for occupancy. In addition, it is assumed that the structure would have been designed and built in accordance with current California practice. Moreover, the estimated additional cost presumes that structures in this acceptable risk category are to embody sufficient safety to remain functional following an earthquake.

[3] Failure of a single structure would affect primarily only the occupants.

[4] These additional percentages are based on the assumption that the base cost is the total cost of the building or facility when ready for occupancy. In addition, it is assumed that the structures would have been designed and built in accordance with current California practice. Moreover the estimated additional cost presumes that structures in this acceptable-risk category are to be sufficiently safe to give reasonable assurance of preventing injury or loss of life during and following an earthquake, but otherwise not necessarily to remain functional.

[5] "Ordinary risk": Resist minor earthquakes without damage: resist moderate earthquakes without structural damage, but with some non-structural damage; resist major earthquakes of the intensity or severity of the strongest experienced in California, without collapse, but with some structural damage as well as non-structural damage. In most structures it is expected that structural damage, even in a major earthquake, could be limited to repairable damage. (Structural Engineers Association of California)

Source: *Meeting the Earthquake,* Joint Committee on Seismic Safety of the California Legislature, Jan. 1974, p.9.

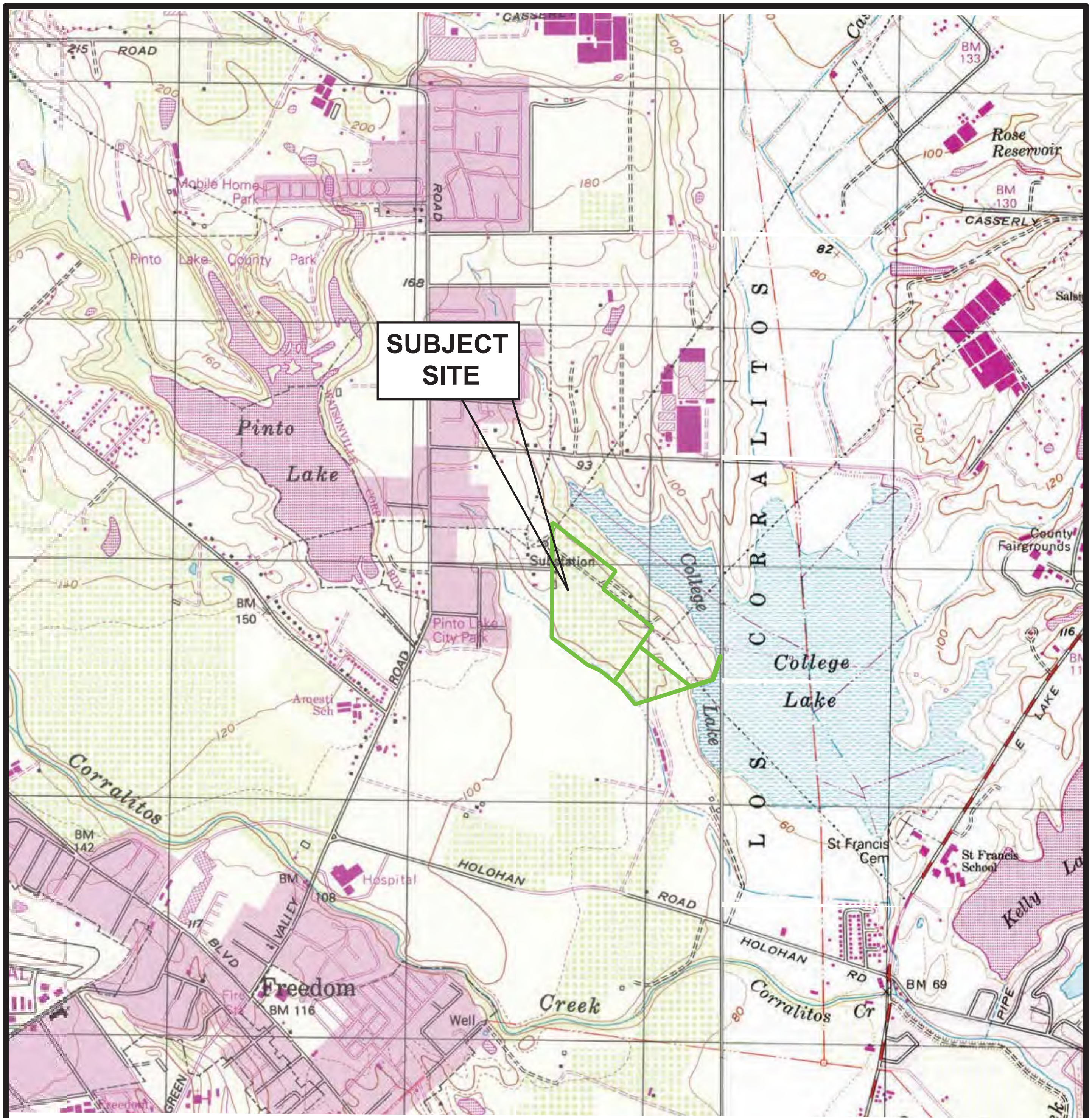
SCALE OF ACCEPTABLE RISKS FROM NON-SEISMIC GEOLOGIC HAZARDS[6]

Risk Level	Structure Type	Risk Characteristics
Extremely low risk	Structures whose continued functioning is critical, or whose failure might be catastrophic: nuclear reactors, large dams, power intake systems, plants manufacturing or storing explosives or toxic materials.	1. Failure affects substantial populations, risk nearly equals nearly zero.
Very low risk	Structures whose use is critically needed after a disaster: important utility centers; hospitals; fire, police and emergency communication facilities; fire station; and critical transportation elements such as bridges and overpasses; also dams.	1. Failure affects substantial populations. Risk slightly higher than 1 above.
Low risk	Structures of high occupancy, or whose use after a disaster would be particularly convenient: schools, churches, theaters, large hotels, and other high rise buildings housing large numbers of people, other places normally attracting large concentrations of people, civic buildings such as fire stations, secondary utility structures, extremely large commercial enterprises, most roads, alternative or non-critical bridges and overpasses.	1. Failure of a single structure would affect primarily only the occupants.
"Ordinary" risk	The vast majority of structures: most commercial and industrial buildings, small hotels and apartment buildings, and single family residences.	1. Failure only affects owners /occupants of a structure rather than a substantial population. 2. No significant potential for loss of life or serious physical injury. 3. Risk level is similar or comparable to other ordinary risks (including seismic risks) to citizens of coastal California. 4. No collapse of structures; structural damage limited to repairable damage in most cases. This degree of damage is unlikely as a result of storms with a repeat time of 50 years or less.
Moderate risk	Fences, driveways, non-habitable structures, detached retaining walls, sanitary landfills, recreation areas and open space.	1. Structure is not occupied or occupied infrequently. 2. Low probability of physical injury. 3. Moderate probability of collapse.

[6] Non-seismic geologic hazards include flooding, landslides, erosion, wave runup and sinkhole collapse

APPENDIX B

Figures 1 through 5



BASE MAP: WATSONVILLE WEST QUADRANGLE, California, 7.5 Minute Series, United States Geological Survey, 1995, scale 1:24,000 and WATSONVILLE EAST QUADRANGLE, California, 7.5 Minute Series, United States Geological Survey, 1995, scale 1:24,000.

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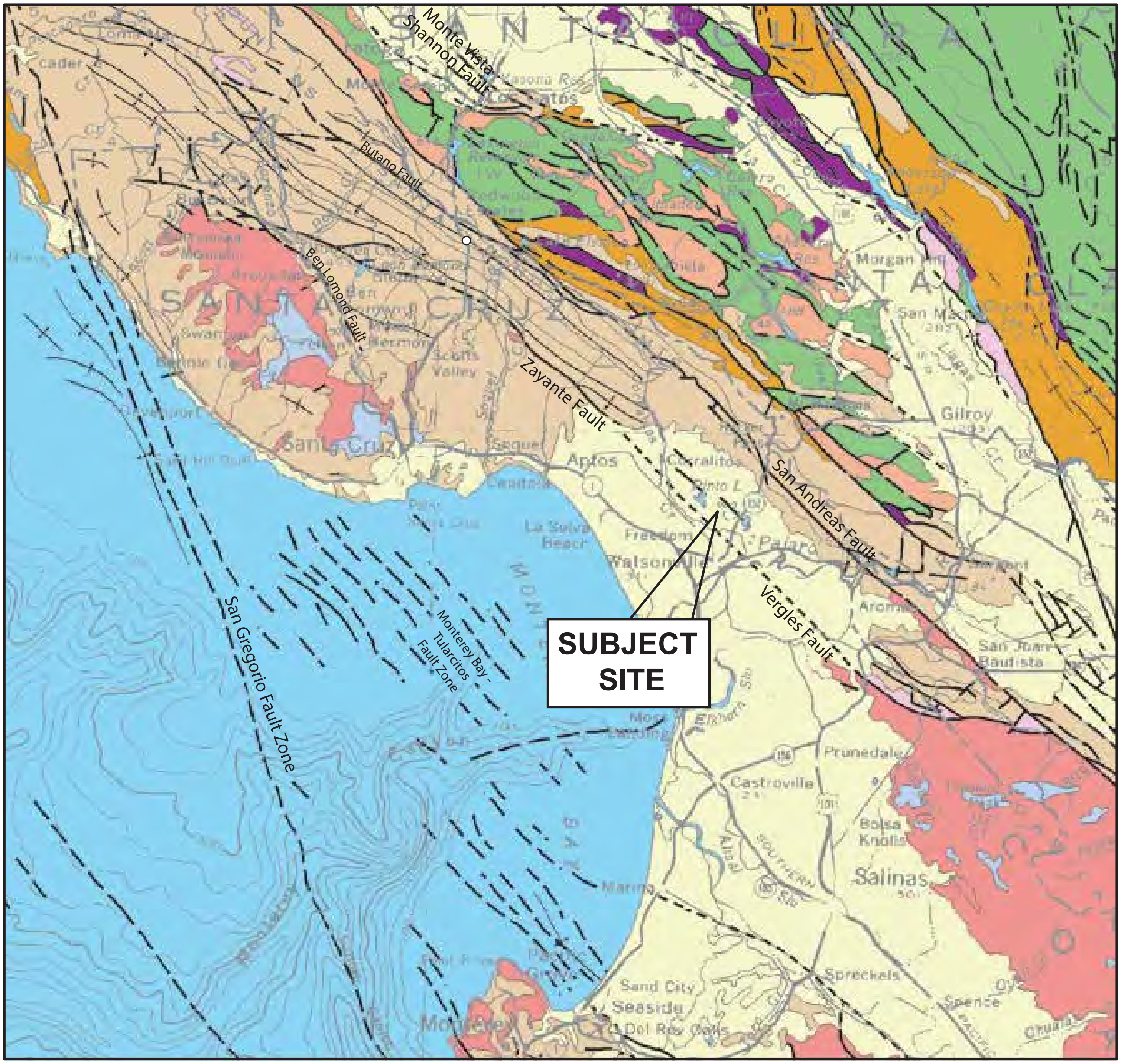
BAYSIDE GEOLOGY

SITE LOCATION MAP
90 Minto Road, Watsonville, California
Santa Cruz County APN 051-101-77 & 78

FIGURE
1



EXPLANATION

Geologic Units

- 🟨 Quaternary Deposits
- 🟨 Quaternary Volcanics
- 🟫 Tertiary Sedimentary Rocks
- ▭ Tertiary Volcanic Rocks
- 🟧 Pre-Tertiary Sedimentary Rocks
- ▭ Pre-Tertiary Volcanic Rocks
- 🟥 Granitic Intrusive Rocks
- 🟩 Franciscan Complex
- 🟪 Ultramafic Rocks
- ▭ Pre-Tertiary Metamorphic Rock
- 🟦 Pre-Cambrian Metamorphic and Igneous Rocks

Symbols

- ╲ contact
- ╲ fault, certain
- ╲ fault,approx. located
- ╲ fault, concealed or inferred
- ✕ anticline
- ✕ monocline
- ✕ syncline

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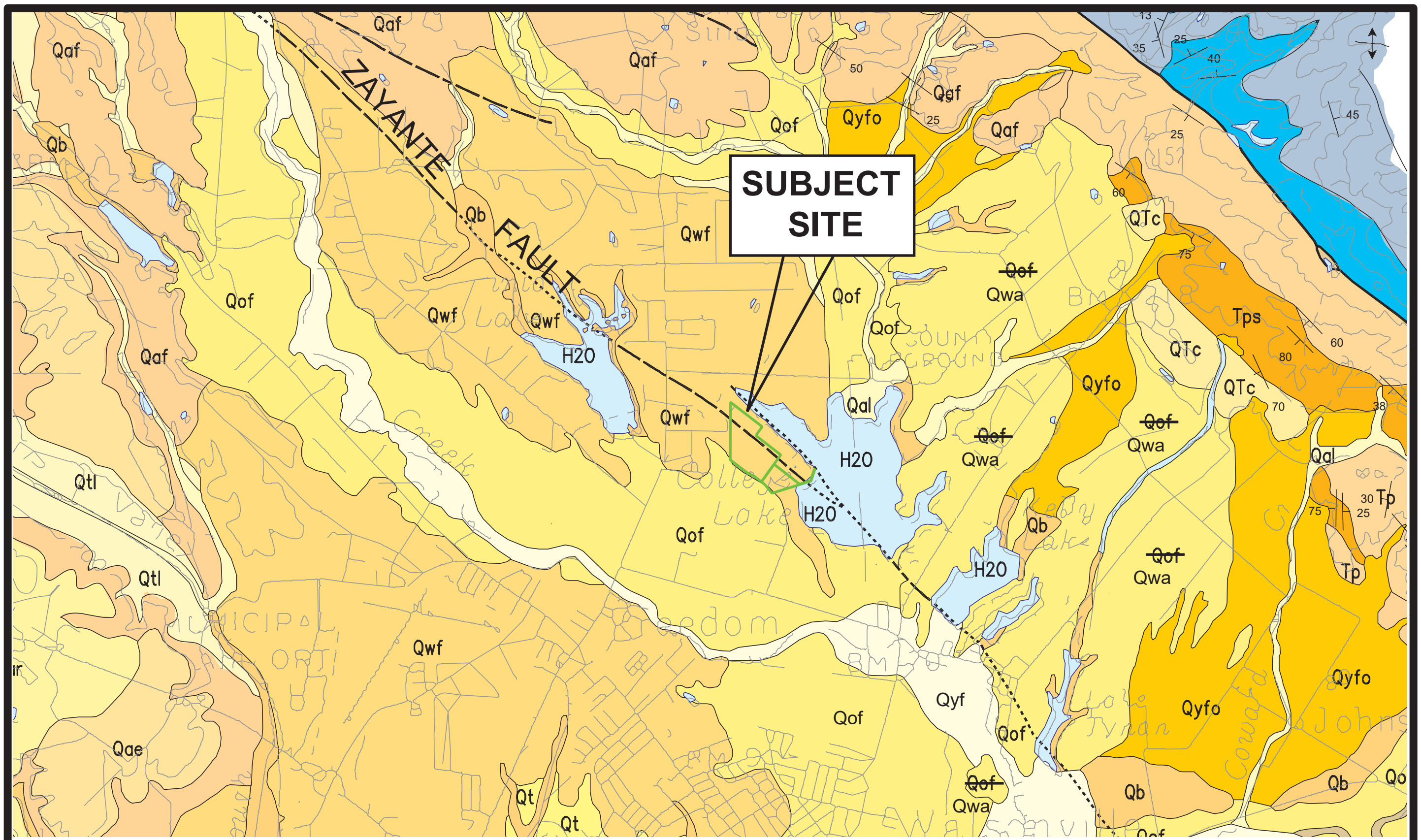
BAYSIDE GEOLOGY

REGIONAL GEOLOGIC MAP
90 Minto Road, Watsonville, California
Santa Cruz County APN 051-101-77 & 78

FIGURE
2



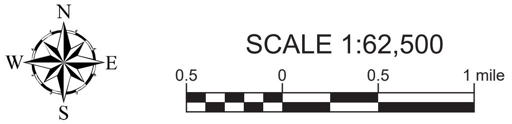
EXPLANATION

Qtl	Colluvium (Holocene)
Qal	Alluvial deposits, undifferentiated (Holocene)
Qyf	Younger flood-plain deposits (Holocene)
Qof	Older flood-plain deposits (Holocene)
Qyfo	Alluvial fan deposits (Holocene)
Qb	Basin deposits (Holocene)
Qds	Dune sand (Holocene)
Qbs	Beach sand (Holocene)
Qt	Terrace deposits, undifferentiated (Pleistocene)
Qes	Eolian deposits of Sunset Beach (Pleistocene)
Qem	Eolian deposits of Manresa Beach (Pleistocene)
	Terrace deposits of Watsonville
Qwf	Fluvial facies (Pleistocene)
Qwa	Alluvial fan facies (Pleistocene)
Qcu	Coastal terrace deposits, undifferentiated (Pleistocene)
Qcl	Lowest emergent coastal terrace deposits
Qar	Aromas Sand, undivided (Pleistocene)
Qae	Eolian lithofacies
Qaf	Fluvial lithofacies
QTc	Continental deposits, undifferentiated (Pliocene?)
Tp	Purisima Formation (Pliocene and upper Miocene)
Tps	Predominantly massive sandstone
Tsc	Santa Cruz Mudstone (upper Miocene)
Tsm	Santa Margarita Sandstone (upper Miocene)
Tm	Monterey Formation (middle Miocene)

Contact
Fault
Anticline
Syncline
Strike and dip of beds
- 80 Inclined
- 50 Approximate dip of bedding
- Vertical
- ⊕ Horizontal
- 60 Overturned

20 Strike and dip of foliation

BASE MAP: Brabb, E.E., 1997, Geologic map of Santa Cruz County, California: Digital Database, U. S. Geological Survey Open-File Report 97-489, scale 1:62,500.



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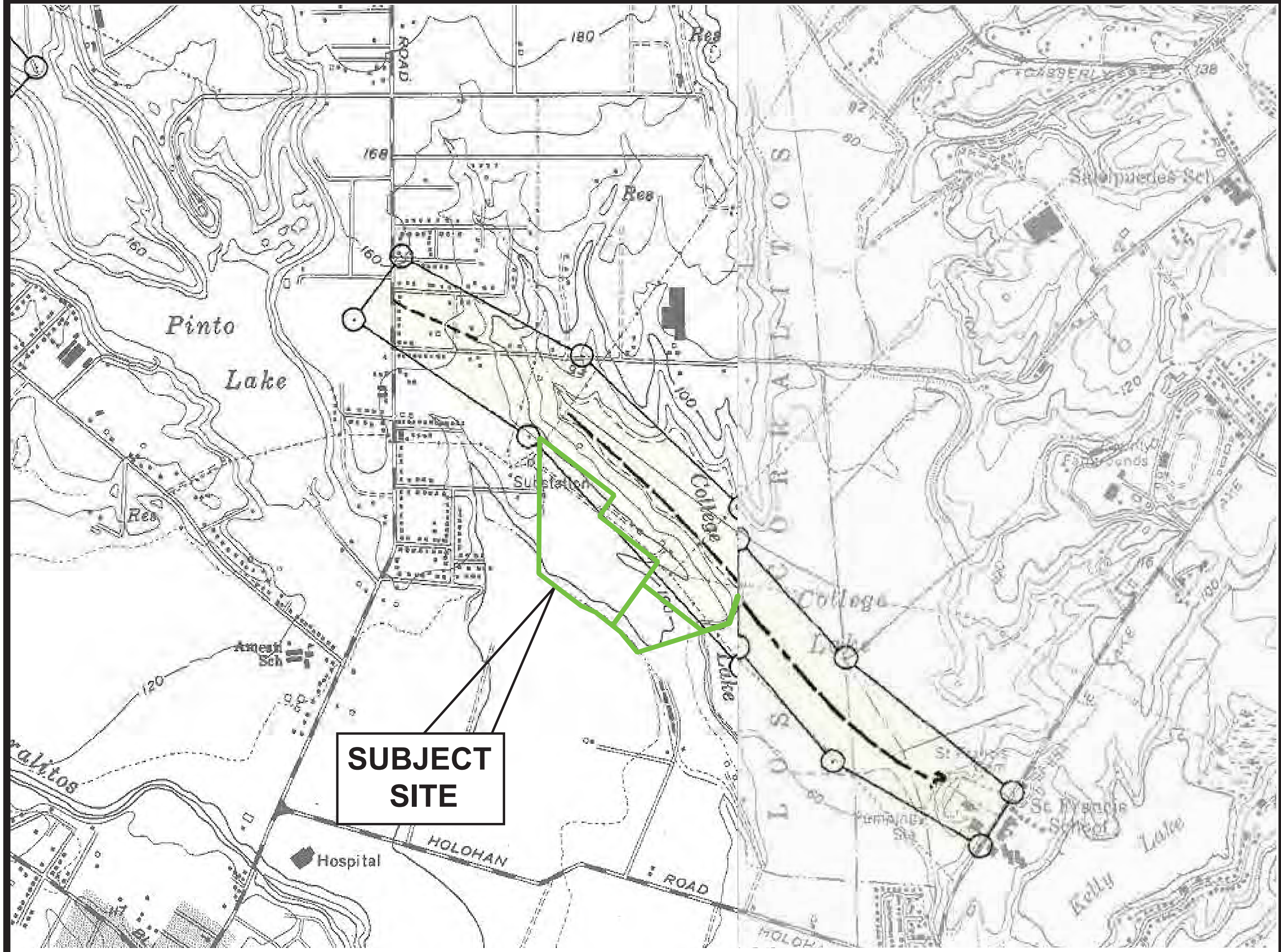
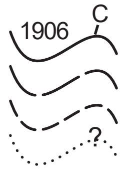
BAYSIDE GEOLOGY

USGS GEOLOGIC MAP
90 Minto Road, Watsonville, California
Santa Cruz County APN 051-101-77 & 78

FIGURE
3



EXPLANATION



ACTIVE FAULTS
Faults considered to have been active during Holocene time and to have a relatively high potential for surface rupture; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by creep or possible creep.

⊙——⊙ **FAULT ZONE BOUNDARIES**
These are delineated as straight-line segments that connect encircled turning points so as to define fault zone segments.

BASE MAP: Gay, T.E., Jr., 1976, State of California Special Studies Zones Map, Watsonville West Quadrangle, California Division of Mines and Geology, scale 1:24,000. and Davis, J.F., 1982, State of California Special Studies Zones, Watsonville East Quadrangle, Revised Official Map, California Division of Mines and Geology, scale 1:24,000.

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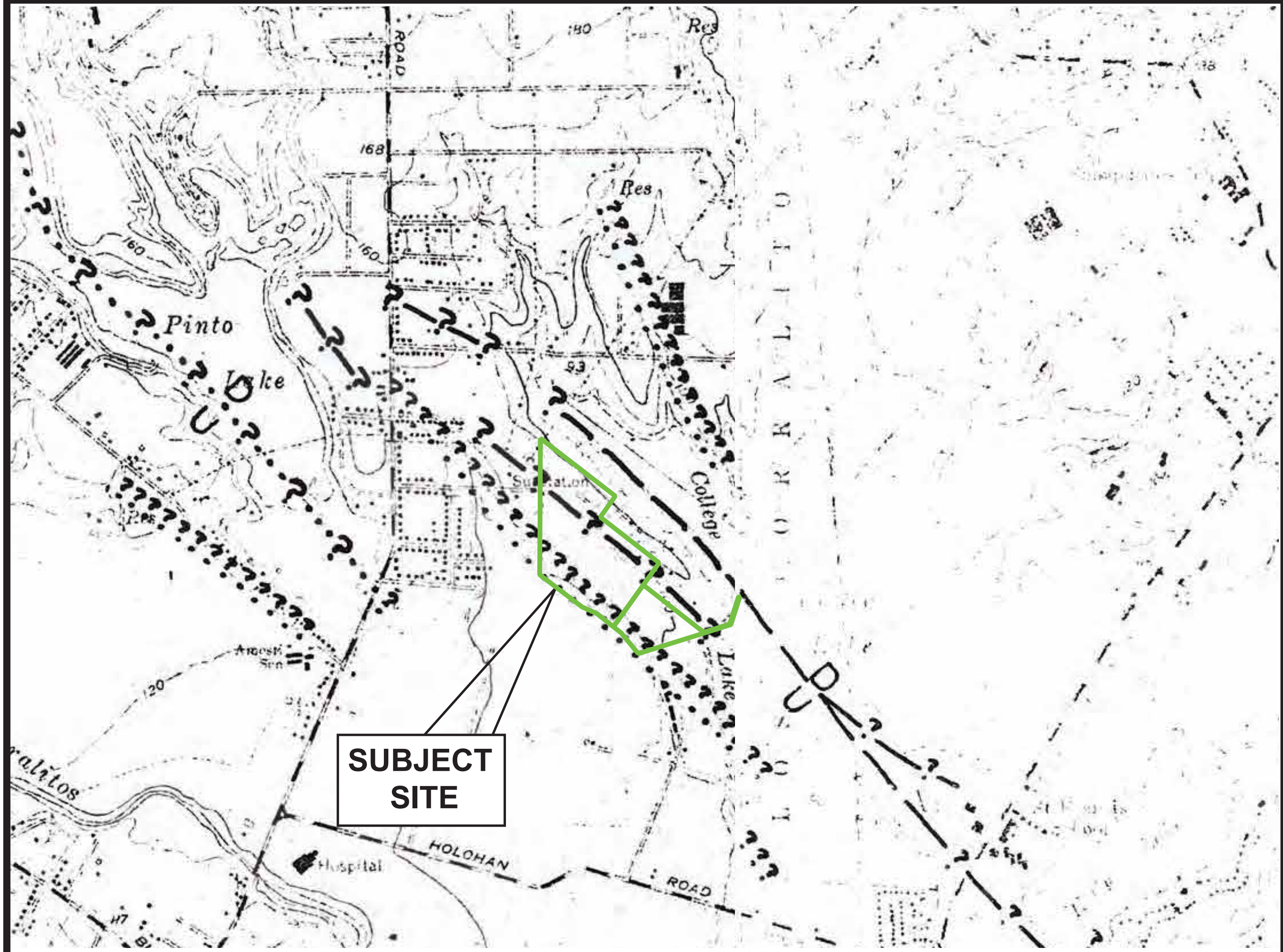
BAYSIDE GEOLOGY

STATE FAULT MAP (Alquist-Priolo)
90 Minto Road, Watsonville, California
Santa Cruz County APN 051-101-77 & 78

FIGURE
4



Classification Of Faults

— — U/D — – ?··?	Probable fault - Dashed where exposed at earth's surface, dotted where concealed beneath younger deposits, vertical movement shown by U (up) and D (down).
?—?—?—?··?	Possible fault - Dashed where exposed at earth's surface, dotted where concealed beneath younger deposits.
? ? ? ? ? ? ? ? ? ?	Photolineament of unknown origin.

BASE MAP: WATSONVILLE EAST and WEST QUADRANGLES:
Hall, N.T., Sarna-Wojcicki, A.M., and Dupré, W.R., 1974,
Faults and Their Potential Hazards in Santa Cruz County,California,
U.S. Geological Survey Miscellaneous Field Studies Map MF-626,
3 sheets, scale 1:62,500.

N
W E
S

SCALE 1:24,000

1000 0 1000 2000 feet

BAYSIDE GEOLOGY

COUNTY FAULT MAP
90 Minto Road, Watsonville, California
Santa Cruz County APN 051-101-77 & 78

FIGURE
5

BAYSIDE GEOLOGY

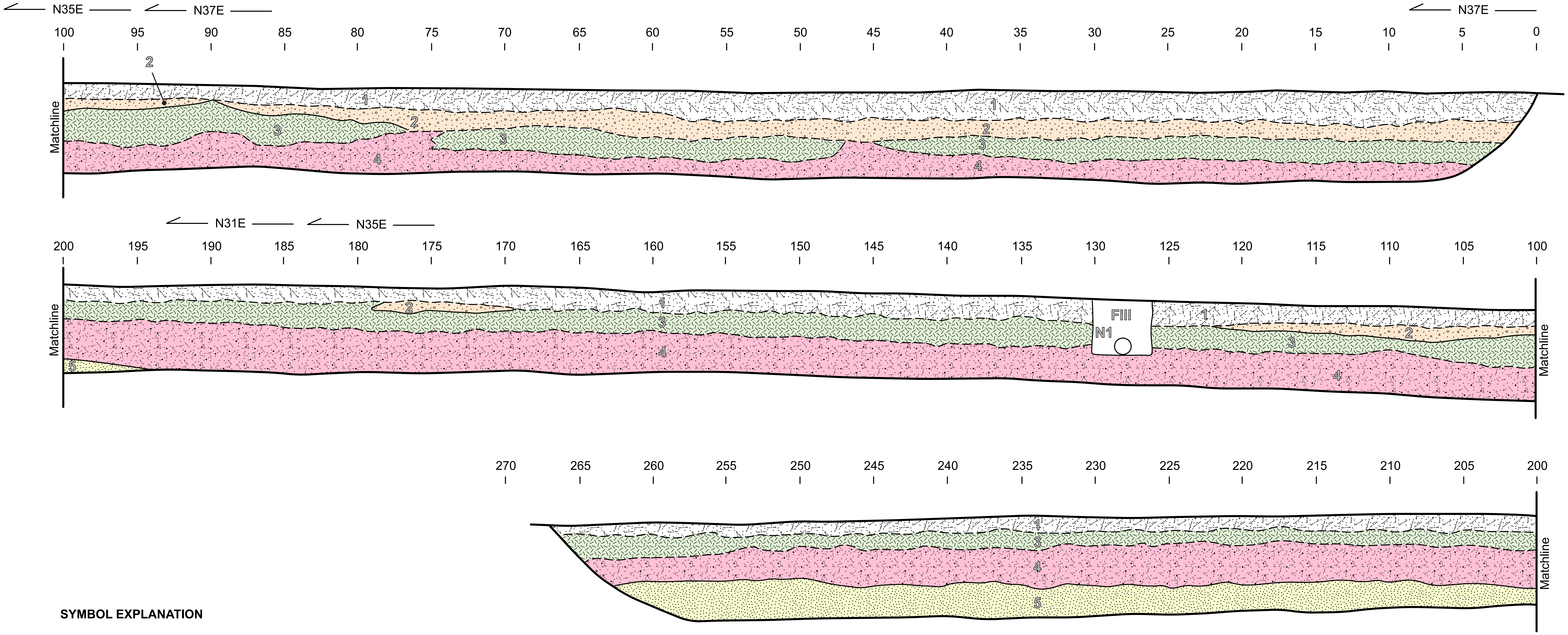
SITE SUBSURFACE EXPLORATION MAP

90 Minto Road, Watsonville, California
Santa Cruz County APN 051-101-77 & 78

PLATE
1

BAYSIDE GEOLOGY	LOG of TRENCH 1 90 Minto Road, Watsonville, California Santa Cruz County APN 051-101-77 & 78	PLATE 2

TRENCH 1
Southeast wall logged
Scale 1"=5', h=v



SYMBOL EXPLANATION

Sharp contact between units

Diffuse contact between units (between 0.1 to 2 inches); queried where uncertain

Gradational contact between units (greater than 2 inches); height of hachures approximately reflects measured transition between units

GENERAL NOTES

No evidence of faulting of any kind, no offset or truncation of units, no striated clay-lined surfaces, etc.

NOTES

N1 Unit contacts are through going and undisturbed, as observed on opposite wall.

UNIT DESCRIPTIONS

1- Sandy SILT ("A" Soil Horizon) - Very dark yellowish brown (Munsell Color: 10YR 3/2) contains few pebbles and widely scattered iron-oxide nodules, scattered micro-pores, few roots and rootlets, moist. Sand is generally fine grained. Includes 5 to 8 inch thick upper till zone.

2 - Pebbly SAND (Qwf - Terrace deposits of Watsonville - Fluvial facies, Pleistocene) - Light gray (Munsell Color: 10YR 7/2) contains some silt and clay as well as abundant pebbles and iron-oxide nodules, massive, moist. Sand is generally fine grained, rounded, and primarily composed of quartz. Pebbles are composed primarily of sandstone and mudstone with occasional chert and grantics. Iron-oxide nodules are 1/8 to 1/2 inch in diameter and concentrated at base of unit.

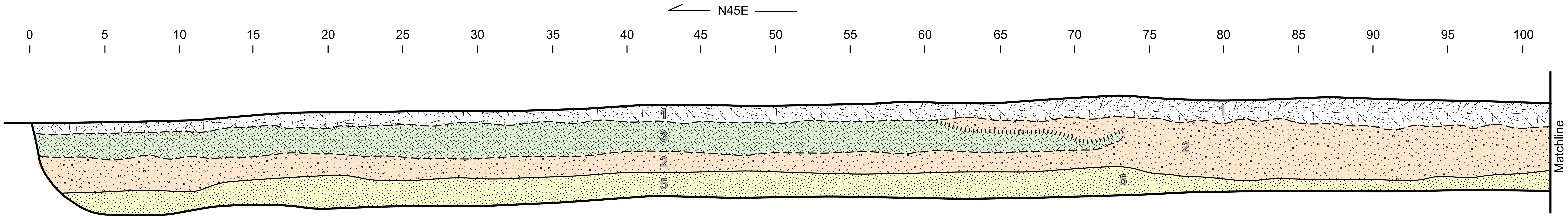
3 - Fat CLAY (Qwf - Terrace deposits of Watsonville - Fluvial facies, Pleistocene) - Olive brown (Munsell Color: 2.5Y 4/3) minor sand content varies laterally, massive, moist to wet.

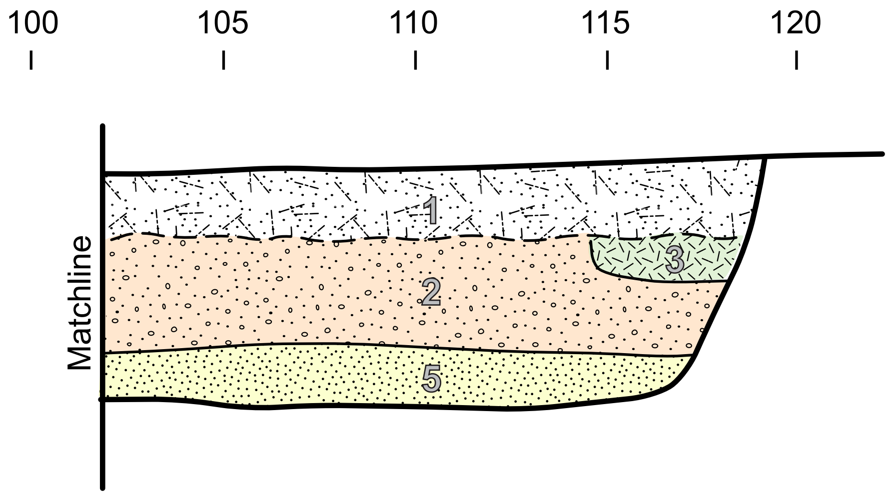
4 - Sandy CLAY / clayey SAND (Qwf - Terrace deposits of Watsonville - Fluvial facies, Pleistocene) - Brown (Munsell Color: 10YR 5/3) contains occasional pebbles, ratio of sand to clay varies vertically and laterally with structure, massive to laminated and cross bedded, well developed shrink/swell fabric, dry to wet. Sand is generally fine grained, but ranges from very fine to medium, is rounded, and primarily composed of quartz. Contains frequent paleo-liquefaction structures (sand blows, sills and dykes marked by secondary oxidation).

5 - SAND (Qwf - Terrace deposits of Watsonville - Fluvial facies, Pleistocene) - Yellowish brown (Munsell Color: 10YR 5/4) contains some silt and clay with few pebbles, thinly bedded to laminated, varies from poorly to well sorted, moist. Sand is generally fine grained, but ranges to coarse, is rounded, and primarily composed of quartz. Pebbles are composed primarily of sandstone and mudstone with occasional chert and grantics. Contains frequent paleo-liquefaction structures (sand blows, sills and dykes marked by secondary oxidation).

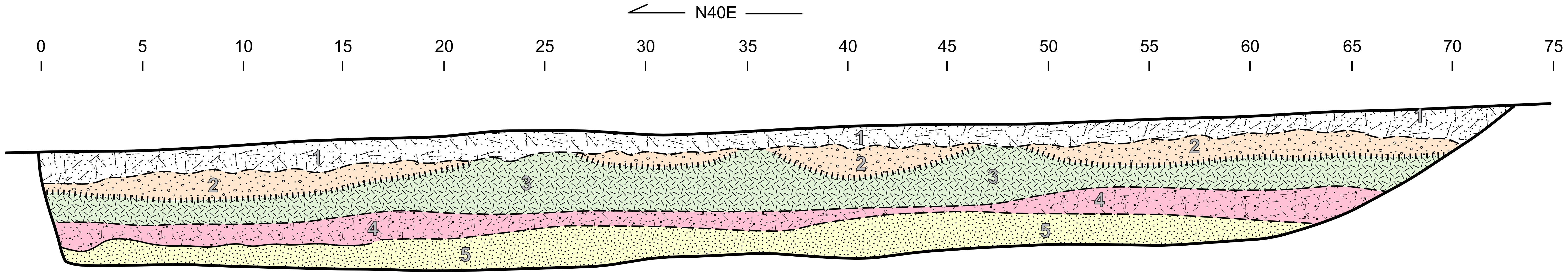
BAYSIDE GEOLOGY	LOG of TRENCH 2 & 3 90 Minto Road, Watsonville, California Santa Cruz County APN 051-101-77 & 78	PLATE 3

TRENCH 2
Southeast wall logged
Scale 1"=5', h=v





TRENCH 3
Southeast wall logged
Scale 1"=5', h=v



SYMBOL EXPLANATION

⁀	Sharp contact between units
- - - -?-?	Diffuse contact between units (between 0.1 to 2 inches); queried where uncertain
IIIIIIIIII	Gradational contact between units (greater than 2 inches); height of hachures approximately reflects measured transition between units

GENERAL NOTES

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UNIT DESCRIPTIONS

1- Sandy SILT ("A" Soil Horizon) - Very dark yellowish brown (Munsell Color: 10YR 3/2) contains few pebbles and widely scattered iron-oxide nodules, scattered micro-pores, few roots and rootlets, moist. Sand is generally fine grained. Includes 5 to 8 inch thick upper till zone.

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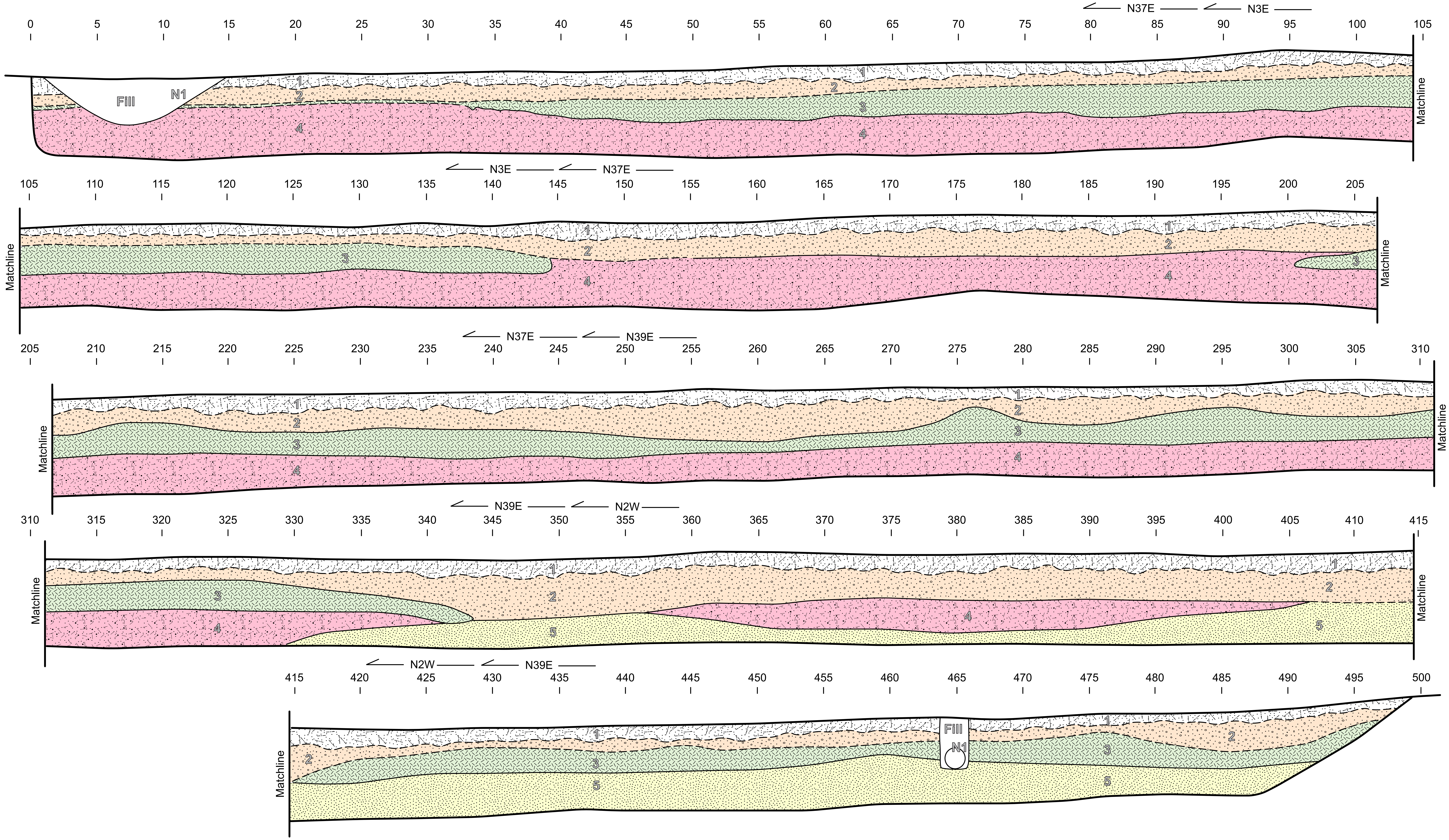
TRENCH 4
Southeast wall logged
Scale 1"=5', h=v

BAYSIDE GEOLOGY

LOG of TRENCH 4
90 Minto Road, Watsonville, California
Santa Cruz County APN 051-101-77 & 78

PLATE
4



UNIT DESCRIPTIONS

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GENERAL NOTES

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NOTES

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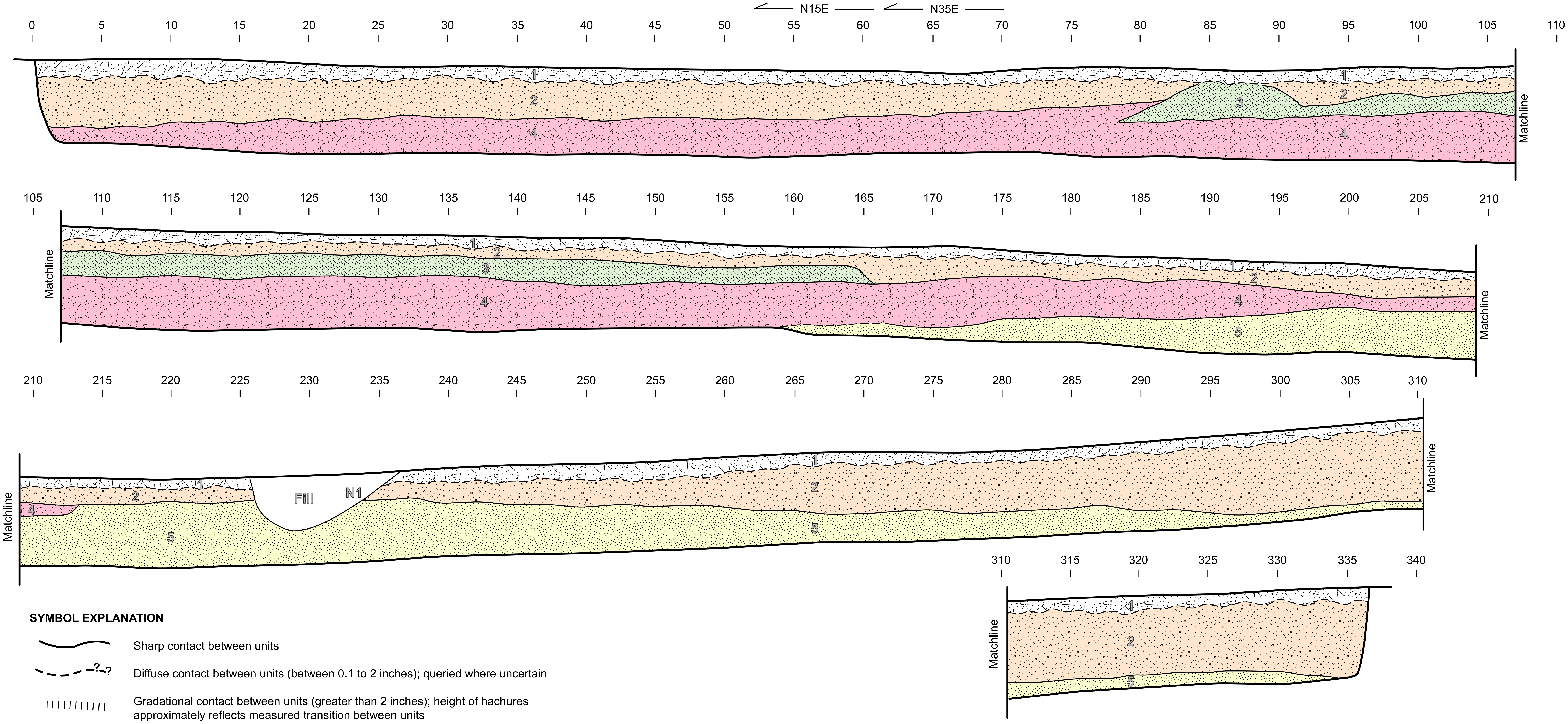
BAYSIDE GEOLOGY | **LOG of TRENCH 5** 90 Minto Road, Watsonville, California Santa Cruz County APN 051-101-77 & 78 | PLATE 5

TRENCH 5
Southeast wall logged
Scale 1”=5’, h=v



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BAYSIDE GEOLOGY
ENGINEERING GEOLOGISTS

October 27, 2025
Job No. 24017-SC

New Leaf Energy, Inc.
55 Technology Drive, Suite 102
Lowell, Massachusetts 01851

ATTENTION: William Peregoy

SUBJECT: Response to County of Santa Cruz, Planning Department, Review Letter
Santa Cruz County Application No: REV251147 (related to 241473)
Seahawk Energy Storage - Battery Energy Storage System
90 Minto Road Watsonville, California
Santa Cruz County APN 051-101-77 & 78

Dear Mr. Peregoy:

We have received the County of Santa Cruz, Planning Department review letter and request for additional information (County of Santa Cruz, 2025). As requested, we provide here our response letter.

Item #1 of the County letter states: *"Please have the project engineering geologist update the site subsurface exploration map (Plate 1) to include the recommended geologic development envelope from a fault rupture perspective (i.e., the area cleared from fault rupture hazard with an appropriate setback). Alternatively, a separate plate or figure can be provided."*

We have attached a separate plate depicting a "Critical Structure Development Envelope, Fault Rupture Hazard Only". The depicted envelope is for the fault rupture hazard only and incorporates a 100 feet setback from the limits of our subsurface exploration. As currently proposed, all "Critical Structures" within the proposed development should be located within the envelope.

As noted in our report (Bayside Geology, 2025): We have not evaluated the risk to the proposed project by the additional identified hazards of expansive soils, liquefaction induced settlement and lateral spreading (landsliding), and seismic shaking. These additional hazards and associated risks are under evaluation by the Project Geotechnical Engineer. Our depicted envelope does not incorporate these additional hazards and potential risks.

Item #2 of the County letter states: *"Please have the project engineering geologist clarify whether excavated fault trenches were backfilled as engineered fill or loose backfill. The project engineering geologist and geotechnical engineer should coordinate to provide recommendations for mitigation of loose backfill, as determined necessary to support the performance of the* *project. Areas known to require significant overexcavation and recompaction should be noted on project civil plans."*

All of the exploratory trench excavations were loosely backfilled with no compaction. We agree, where necessary, given the proposed development, loose trench backfill should be overexcavated and recompacted under the direction of the Project Geotechnical Engineer. The proposed grading plans should note such areas, if any.

INVESTIGATION LIMITATIONS

The findings of this letter are based on our subsurface exploration and our understanding of the proposed development.

Our evaluation was performed in accordance with the usual and current standards of the profession, as they relate to this and similar localities. No other warranty, expressed or implied, is provided as to the conclusions and professional advice presented in this letter.

The findings of this letter are considered valid as of the present date. However, changes in the conditions of a site can occur with the passage of time, whether they be due to natural events or to human activities on this or adjacent sites. In addition, changes in applicable or appropriate codes and standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, this letter may become invalidated wholly or partially by changes outside our control. Therefore, this letter is subject to review and revision as conditions change.

Please contact us if you have any questions regarding this letter.

Sincerely,
BAYSIDE GEOLOGY

James A. Olson
Principal Geologist
Professional Geologist #7244
Certified Engineering Geologist #2267

Distribution: Addressee (electronic copy)

REFERENCES

Bayside Geology, 2025, Geologic Fault Investigation, New Leaf Energy, Inc. Property, 90 Minto Road, Watsonville, California, Santa Cruz County APN 051-101-77 & 78, Job No. 25017-SC, unpublished consultants report dated August 1, 2025.

County of Santa Cruz, 2025, Review of Geologic and Geotechnical Investigations, 90 Minto Road, Watsonville, California, Santa Cruz County APN 051-101-77 & 78, Application No:REV251147 (related to 241473), letter dated October 23, 2025.

BAYSIDE GEOLOGY

CRITICAL STRUCTURE DEVELOPMENT ENVELOPE
90 Minto Road, Watsonville, California
Santa Cruz County APN 051-101-77 & 78

Appendix 3.4B

Geologic Fault Investigation

BAYSIDE GEOLOGY
ENGINEERING GEOLOGISTS

GEOLOGIC FAULT INVESTIGATION
NEW LEAF ENERGY INC. PROPERTY
90 MINTO ROAD
WATSONVILLE, CALIFORNIA
SANTA CRUZ COUNTY APN 051-101-77 & 78

Job No. 24017-SC
August 1, 2025

202 Anita Street
Santa Cruz, California 95060
jim@baysidegeology.com
(831) 234-7825

BAYSIDE GEOLOGY
ENGINEERING GEOLOGISTS

August 1, 2025
Job No. 24017-SC

New Leaf Energy, Inc.
55 Technology Drive, Suite 102
Lowell, Massachusetts 01851

ATTENTION: William Peregoy

SUBJECT: **GEOLOGIC FAULT INVESTIGATION**
Proposed Battery Energy Storage System Facility
90 Minto Road Watsonville, California
Santa Cruz County APN 051-101-77 & 78

Dear Mr. Peregoy:

As requested, we have completed our geologic fault evaluation of the above-referenced property. The intent of our investigation was to provide a detailed subsurface trenching evaluation of the potential for tectonic faulting relative to the development of the proposed Battery Energy Storage System (BESS) facility. This report details the findings and conclusions from our literature review, field exploration, and geologic analysis.

It is our opinion that the proposed development will be subject to an “ordinary” level of risk relative to the tectonic faulting hazard, provided our recommendations and the recommendations of your project geotechnical and civil engineers are followed.

Please contact us if you have any questions regarding this report.

Sincerely,
BAYSIDE GEOLOGY

JAMES A. OLSON
No. 2267
CERTIFIED
ENGINEERING
GEOLOGIST

James A. Olson
Principal Geologist
Professional Geologist #7244
Certified Engineering Geologist #2267

Copies: Addressee (electronic copy)

202 Anita Street
Santa Cruz, California 95060
jim@baysidegeology.com
(831) 234-7825

TABLE OF CONTENTS

NOTE: Plates must accompany text of report in order for report to be considered complete.

INTRODUCTION

This report presents the results of our evaluation of the New Leaf Energy Inc. property (APN 051-101-77 & 78). The property is located southwest of the Santa Cruz Mountains in Watsonville, California (Figure 1; Site Location Map). The north west corner of the site is currently occupied with a large barn and several smaller structures. The bulk of the property is in production as an apple orchard. It is our understanding that the proposed project consists of the development of a BESS facility.

Previously, a Geologic Hazards Assessment (GHA) for the proposed project was completed by Haley & Aldrich, Inc. (2024). They identified surface fault rupture, expansive soils, liquefaction induced settlement and lateral spreading (landsliding), and seismic shaking as hazards that present a potential risk to the proposed development. Review of the GHA by the Santa Cruz County Geologist concurred with their finding (Santa Cruz County, 2024). As part of the conditions for the development of the proposed project, Santa Cruz County has required a focused geologic fault investigation performed by the Project Geologist. They have not required a full geologic investigation typical for development of habitable space. The additional geologic hazards of expansive soils, liquefaction induced settlement and lateral spreading (landsliding), and seismic shaking that present a potential risk to the proposed project are required to be evaluated by the Project Geotechnical Engineer.

In accordance with your authorization, we have completed a detailed, subsurface trenching investigation of the potential for tectonic fault rupture to impact the proposed project. We have not evaluated the risk to the proposed project by the additional identified hazards of expansive soils, liquefaction induced settlement and lateral spreading (landsliding), and seismic shaking.

The scope of work performed for this evaluation included: 1) review of published and unpublished literature relevant to the site and vicinity; 2) analysis of stereo-paired, aerial photographs; 3) review of Google Earth aerial imagery; 4) reprocessing and review of Santa Cruz County LiDAR imagery of the property; 5) geologic reconnaissance mapping of the site; 6) coordination and discussion of our proposed subsurface exploration plan with the Santa Cruz County Geologist; 7) excavation and logging of approximately 1,300 feet of exploratory fault trench; 8) compilation and analysis of the resulting data; and 9) preparation of this report and accompanying illustrations.

REGIONAL GEOLOGIC SETTING

The subject property is located on the southwest margin of the Santa Cruz Mountains of the California Coast Ranges (Figure 1). The Santa Cruz Mountains are formed by a series of rugged, linear ridges and valleys following the pronounced northwest to southeast structural grain of central California geology. Underlying the Santa Cruz Mountains is a large, elongate prism of granitic and metamorphic basement rocks. These rocks are separated from contrasting basement rock types to the northeast and southwest by the San Andreas and Monterey Bay/Tularcitos strike-slip fault systems, respectively. Overlying the granitic basement rocks is a sequence of dominantly marine sedimentary rocks of Paleocene to Pliocene age and non-marine sediments of Pliocene to Pleistocene age (Figure 2).

Throughout the Cenozoic Era, this portion of California was dominated by tectonic forces associated with lateral or "transform" motion between the North American and Pacific crustal plates, producing long, northwest-trending faults such as the San Andreas, with horizontal displacements measured in tens to hundreds of miles. Accompanying the northwest direction of the horizontal (strike-slip) movement of the plates were episodes of compressive stress, causing repeated uplift, deformation, erosion and subsequent redeposition of sedimentary rocks. This tectonic deformation is most evident in sedimentary rocks older than the middle Miocene, and consists of steeply dipping folds, overturned bedding, faulting, jointing, and fracturing. Along the coast, the ongoing tectonic activity is most evident in the formation of a series of uplifted marine terraces. The Loma Prieta earthquake of 1989 and its aftershocks are the most recent reminders of the geologic unrest in the region.

REGIONAL SEISMIC SETTING

California's broad system of strike-slip faulting has a long and complex history. Some of these faults present a seismic hazard to the subject property. The most important of these are the San Andreas, Zayante-Vergeles and San Gregorio faults (Figure 2). These faults are either active or considered potentially active (Buchanan-Banks et al., 1978; Burkland and Associates, 1975; Jennings et al., 1975; Greene, 1977; Hall et al., 1974; Schwartz et al., 1990; Wallace, 1990; and Working Group on Northern California Earthquake Potential [WGNCEP], 1996). Each fault is discussed below.

San Andreas Fault

The San Andreas fault is active and represents the major seismic hazard in northern California (Jennings et al., 1975; Buchanan-Banks et al., 1978; Hall et al., 1974). The main trace of the San Andreas fault trends northwest-southeast and extends over 700 miles from the Gulf of California through the Coast Ranges to Point Arena, where the fault extends offshore.

Geologic evidence suggests that the San Andreas fault has experienced right-lateral, strike-slip movement throughout the latter portion of Cenozoic time, with cumulative offset of hundreds of miles. Surface rupture during historical earthquakes, fault creep, and historical seismicity confirm that the San Andreas fault and its branches, the Hayward, Calaveras, and San Gregorio faults, are all active today.

Historical earthquakes along the San Andreas fault and its branches have caused significant seismic shaking in the Santa Cruz County area. The two largest historical earthquakes on the San Andreas to affect the area were the moment magnitude (M_w) 7.9 San Francisco earthquake of April 18, 1906 (actually centered near Olema) and the M_w 6.9 Loma Prieta earthquake of October 17, 1989. The San Francisco earthquake caused severe seismic shaking and structural damage to many buildings in Santa Cruz County. The Loma Prieta earthquake appears to have caused more intense seismic shaking than the 1906 event in localized areas of the Santa Cruz Mountains, even though its regional effects were not as extensive. There were also significant earthquakes in northern California along or near the San Andreas fault in 1838, 1865 and possibly 1890 (Sykes and Nishenko, 1984; Working Group on Northern California Earthquake Potential, 1996).

Geologists have recognized that the San Andreas fault system can be divided into segments with earthquakes of different magnitudes and recurrence intervals (Working Group on California Earthquake Probabilities, 1988 and 1990; Working Group on Northern California Earthquake Potential (WGNCEP), 1996). The most recent work has redefined the segments and defines 10 sections: Shelter Cove, North Coast, Peninsula, Santa Cruz Mountains, Creeping, Parkfield, Cholame-Carrizo, Mojave, San Bernardino Mountains, and Coachella sections (Bryant, 2000). In theory, the segments rupture independently or combine to produce earthquakes with larger magnitudes.

In summary, the San Andreas fault should be considered active. It is considered capable of generating a magnitude 8.1 earthquake (USGS, 2008) with a recurrence interval of 100-450 years (Bryant, 2000).

Zayante-Vergeles Fault

The Zayante fault lies west of the San Andreas fault and trends about 50 miles northwest from the Watsonville lowlands into the Santa Cruz Mountains. The southern extension of the Zayante fault, known as the Vergeles fault, merges with the San Andreas fault south of San Juan Bautista.

The Zayante fault has a long, well-documented history of vertical movement (Clark and Reitman, 1973), probably accompanied by right-lateral, strike-slip movement (Hall et al., 1974; Ross and Brabb, 1973). Stratigraphic and geomorphic evidence indicates the Zayante fault has undergone late Pleistocene and Holocene movement and is potentially active (Buchanan-Banks et al., 1978; Coppersmith, 1979).

Some historical seismicity may be related to the Zayante fault (Griggs, 1973). For instance, the Zayante fault may have undergone sympathetic fault movement during the 1906 earthquake centered on the San Andreas fault, although this evidence is equivocal (Coppersmith, 1979). Seismic records strongly suggest that a section of the Zayante fault approximately 3 miles long underwent sympathetic movement in the 1989 earthquake. The earthquake hypocenters tentatively correlated to the Zayante fault occurred at a depth of 5 miles; no instances of surface rupture on the fault have been reported.

In summary, the Zayante-Vergeles fault should be considered potentially active. It is considered capable of generating a magnitude 7.0 earthquake (USGS, 2008) with a recurrence interval of 3,130 years (Wesnousky, 1986).

San Gregorio Fault

The San Gregorio fault, as mapped by Greene (1977), Weber et al. (1979), Weber and Lajoie (1974), and Weber et al. (1995), skirts the coastline of Santa Cruz County northward from Monterey Bay and trends onshore at Point Año Nuevo. Northward from Año Nuevo, it passes offshore again, touching onshore briefly at Seal Cove just north of Half Moon Bay, and eventually connects with the San Andreas fault near Bolinas. Southward from Monterey Bay, it may trend onshore north of Big Sur (Greene, 1977) to connect with the Palo Colorado fault, or it may continue southward through Point Sur to connect with the Hosgri fault in south-central California. Based on these two proposed correlations, the San Gregorio fault zone has a length of at least 100 miles and possibly as much as 250 miles.

The on land exposures of the San Gregorio fault at Point Año Nuevo and Seal Cove show evidence of late Pleistocene displacement (Jennings, 1975; and Buchanan-Banks et al., 1978) and Holocene displacement (Weber and Cotton, 1981; Simpson et al., 1997). Although stratigraphic offsets indicate a history of horizontal and vertical displacements, the San Gregorio is considered predominantly right-lateral strike slip by most researchers (Greene, 1977; Weber and Lajoie, 1974; and Graham and Dickinson, 1978).

In addition to stratigraphic evidence for Holocene activity, the historical seismicity in the region is partially attributed to the San Gregorio fault (Greene, 1977). Due to inaccuracies of epicenter locations, even the magnitude the 6+ earthquakes of 1926, tentatively assigned to the Monterey Bay fault zone, may have actually occurred on the San Gregorio fault (Greene, 1977).

The WGNCEP (1996) divided the San Gregorio fault into the "San Gregorio" and "San Gregorio, Sur Region" segments. The segmentation boundary is located west of Monterey Bay, where the fault appears to have a right step-over (Figure 2). The San Gregorio segment is assigned a slip rate that results in a M_w 7.3 earthquake with a recurrence interval of 400 years. This value was assigned based on the preliminary results of a paleoseismic investigation at Seal Cove by Lettis and Associates (see Simpson et al., 1997) and on regional mapping by Weber et al. (1995). Simpson et al. (1997) discovered prior displacements consistent with a moment magnitude of 7 to 7¼ in their paleoseismic study at Seal Cove. The Sur Region segment is assigned a slip rate that results in a M_w 7.0 earthquake with an effective recurrence interval of 411 years. Within the Sur Region many geologists, including Greene (1977), map the San Gregorio fault zone as continuing along the Palo Colorado fault. Graham and Dickinson (1978) show the San Gregorio fault continuing along the Sur fault zone.

Bryant (2000) has adopted a model similar to the WGNCEP (1996). They split the fault into the San Gregorio section and the Sur Region section. The forecasted earthquake on the combined fault is a Mw 7.5 (USGS, 2008).

DESCRIPTION OF SITE AND VICINITY

The Site Location Map (Figure 1), Regional Geologic Map (Figure 2), USGS Geologic Map (Figure 3), State Fault Map (Alquist Priolo) (Figure 4), County Fault Map (Figure 5), Site Subsurface Exploration Map (Plate 1), and Trench Logs (Plates 2 thru 5) depict the relevant topographic and geologic information on the subject property. We initiated our trenching this past January and completed it in May and June (2025).

Geomorphology

The subject property is approximately 2,200 feet long and 1,000 feet wide, encompassing about 47 acres. Total relief across the site is about 20 feet and slopes are gentle. A northwest trending ridge and swale traverse the property. The morphology of the site is unaltered by grading.

The subject site is located within the Watsonville lowlands; a backfilled sedimentary basin. The morphology of the region is controlled by the combined forces of plate tectonics (faulting) with sedimentation/erosion driven by glacio-eustatic cycling (ice ages and sea-level change) [Dupré, 1975]. Over time, this has resulted in a complex assemblage of folded and faulted marine, fluvial (river), estuarine (lagon), alluvial fan (sheet-wash and debris flow) and eolian (dune) deposits within the basin. Most recently, as the sea-level has risen, entrenched rivers and streams have been backfilled with a sequence of marine to terrestrial deposits. The backfilling along Corralitos and Salsipuedes Creeks has dammed side streams, forming several of the lakes in the area (Pinto Lake, College Lake, etc.).

Earth Materials and Geologic Structure

The published geologic map of the area (Figure 3) shows the site and surrounding area as underlain by an Terrace deposits of Watsonville - Fluvial facies (Qwf). The formation is described as "Semiconsolidated, moderately to poorly sorted silt, sand, silty clay, and gravel. May be more than 200 ft thick..." (Brabb, 1997).

Our observations of the earth materials at the site are in general agreement with the published geologic map. Our subsurface exploratory trenching exposed a disturbed "A" soil horizon overlying interbedded fluvial subunits of the Fluvial (river) facies of the Terrace deposits of Watsonville (Qwf). We mapped 4 distinct subunits that were generally present across the entire site, although occasionally discontinuous. Bedding is approximately horizontal. Contacts between the subunits are over lapping and interfingered. Please see the trench logs (Plates 2 thru 5) for a detailed description and a graphic depiction of the fluvial subunits.

GEOLOGIC HAZARDS

Faulting

The north-eastern boundary of the property is located within the State of California Special Studies Zone (Alquist Priolo Act), for the Zayante Fault (Parrish, 2018; Gay, 1976; Davis, 1982). The entire property is located within the Santa Cruz County Fault Zone (Hall et al., 1974).

During our aerial photographic and LiDAR review we mapped a lineament within the swale that runs the length of the property. Hall et al., (1974) map this same lineament as a possible fault (Figure 5). To further evaluate the potential for faulting we advanced, scraped and logged about 1,300 feet of exploratory trench, distributed across 5 separate trenches (Plate 1). The trenches were spread across the proposed development. As required for critical structures, the length of the trenching incorporated a minimum of 100 feet of overlap on both ends given a +/-10-degree variation in regional fault orientation. We evaluated the trenches for evidence for faulting. The contacts between the terrace deposit subunits are distinct and continuous or over lapping along the entire length of the exploratory trenching. Overall, the exploratory trenching revealed no indication of tectonic faulting of any kind; no offset or truncation of units, no soil mixing or colluvial wedges, no continuous (through going) shear planes, no alignment of gravels, etcetera.

CONCLUSIONS

This report presents the results of our evaluation of the New Leaf Energy Inc. property (APN 051-101-77 & 78). The property is located southwest of the Santa Cruz Mountains in Watsonville, California (Figure 1; Site Location Map). The north west corner of the site is currently occupied with a large barn and several smaller structures. The bulk of the property is in production as an apple orchard. It is our understanding that the proposed project consists of the development of a BESS facility.

The property is located within both the Santa Cruz County Fault Zone and within the State of California Special Studies Zone (Alquist Priolo Act), for the Zayante Fault (Hall et al., 1974; Parrish, 2018; Gay, 1976; Davis, 1982). Our exploratory trenching revealed no indication of tectonic faulting of any kind. Given our evaluation, there is a low likelihood that the proposed development will be damaged by surface ground rupture due to tectonic faulting within the lifetime of the project.

We have not evaluated the risk to the proposed project by the additional identified hazards of expansive soils, liquefaction induced settlement and lateral spreading (landsliding), and seismic shaking.

It is our opinion that the proposed development will be subject to an "ordinary" level of risk (as defined in Appendix A) relative to the tectonic faulting hazard, provided our recommendations and the recommendations of your project geotechnical and civil engineers are followed. Appendix A should be reviewed in detail by the owner to determine whether this risk, as defined in the appendix, is acceptable. If this level of risk is unacceptable to the owner, then the risk should be further mitigated to an acceptable level.

RECOMMENDATIONS

1. The proposed BESS facility should be generally located as currently planned and as shown on our Site Subsurface Exploration Map (Plate 1).

2. Drainage from improved surfaces, such as walkways, patios, roofs and roads, should generally be dispersed while maintaining a natural path and remain within the basin of origin.

   Engineered, site-specific drainage and erosion control plans should be reviewed and approved by the project geologist and project geotechnical engineer. Proper drainage control design, implementation and maintenance throughout the lifetime of the development is critical.

3. We request the privilege of reviewing all geotechnical engineering reports and letters, civil engineering, drainage, and landscaping plans pertaining to the proposed development. If our firm is not afforded the privilege of providing these review services we can assume no responsibility for misinterpretation of our recommendations.

4. **We recommend that Bayside Geology be retained to observe any and all excavations.** Field observation must be provided by a representative of Bayside Geology

to enable us to form an opinion as to the degree of conformance of the site conditions exposed during excavation to those described in our geologic report. Any excavation performed without the full knowledge and direct observation of Bayside Geology, the project geologist of record, will render the recommendations of our report invalid.

INVESTIGATION LIMITATIONS

1. The conclusions and recommendations contained herein are based on probability and in no way imply that the proposed improvements will not possibly be subjected to ground failure, seismic shaking or faulting of such a magnitude that it overwhelms the site. The report does suggest that using the site for the proposed purpose in compliance with the recommendations contained herein represents an “ordinary” risk.

2. This report is issued with the understanding that it is the duty and responsibility of the owner or his representative or agent to ensure that the recommendations contained in this report are brought to the attention of the architect and engineers for the project, incorporated into the plans and specifications, and that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. If any unexpected variations in soil conditions or if any undesirable conditions are encountered during construction, Bayside Geology should be notified so that supplemental recommendations may be given.

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APPENDIX A

SCALE OF ACCEPTABLE RISKS FROM GEOLOGIC HAZARDS

SCALE OF ACCEPTABLE RISKS FROM SEISMIC GEOLOGIC HAZARDS

Risk Level	Structure Types	Extra Project Cost Probably Required to Reduce Risk to an Acceptable Level
Extremely low[1]	Structures whose continued functioning is critical, or whose failure might be catastrophic: nuclear reactors, large dams, power intake systems, plants manufacturing or storing explosives or toxic materials.	No set percentage (whatever is required for maximum attainable safety).
Slightly higher than under "Extremely low" level.[1]	Structures whose use is critically needed after a disaster: important utility centers; hospitals; fire, police and emergency communication facilities; fire station; and critical transportation elements such as bridges and overpasses; also dams.	5 to 25 percent of project cost.[2]
Lowest possible risk to occupants of the structure.[3]	Structures of high occupancy, or whose use after a disaster would be particularly convenient: schools, churches, theaters, large hotels, and other high rise buildings housing large numbers of people, other places normally attracting large concentrations of people, civic buildings such as fire stations, secondary utility structures, extremely large commercial enterprises, most roads, alternative or non-critical bridges and overpasses.	5 to 15 percent of project cost.[4]
An "ordinary" level of risk to occupants of the structure.[3,5]	The vast majority of structures: most commercial and industrial buildings, small hotels and apartment buildings, and single family residences.	1 to 2 percent of project cost, in most cases (2 to 10 percent of project cost in a minority of cases).[4]

[1] Failure of a single structure may affect substantial populations.

[2] These additional percentages are based on the assumptions that the base cost is the total cost of the building or other facility when ready for occupancy. In addition, it is assumed that the structure would have been designed and built in accordance with current California practice. Moreover, the estimated additional cost presumes that structures in this acceptable risk category are to embody sufficient safety to remain functional following an earthquake.

[3] Failure of a single structure would affect primarily only the occupants.

[4] These additional percentages are based on the assumption that the base cost is the total cost of the building or facility when ready for occupancy. In addition, it is assumed that the structures would have been designed and built in accordance with current California practice. Moreover the estimated additional cost presumes that structures in this acceptable-risk category are to be sufficiently safe to give reasonable assurance of preventing injury or loss of life during and following an earthquake, but otherwise not necessarily to remain functional.

[5] "Ordinary risk": Resist minor earthquakes without damage: resist moderate earthquakes without structural damage, but with some non-structural damage; resist major earthquakes of the intensity or severity of the strongest experienced in California, without collapse, but with some structural damage as well as non-structural damage. In most structures it is expected that structural damage, even in a major earthquake, could be limited to repairable damage. (Structural Engineers Association of California)

Source: *Meeting the Earthquake,* Joint Committee on Seismic Safety of the California Legislature, Jan. 1974, p.9.

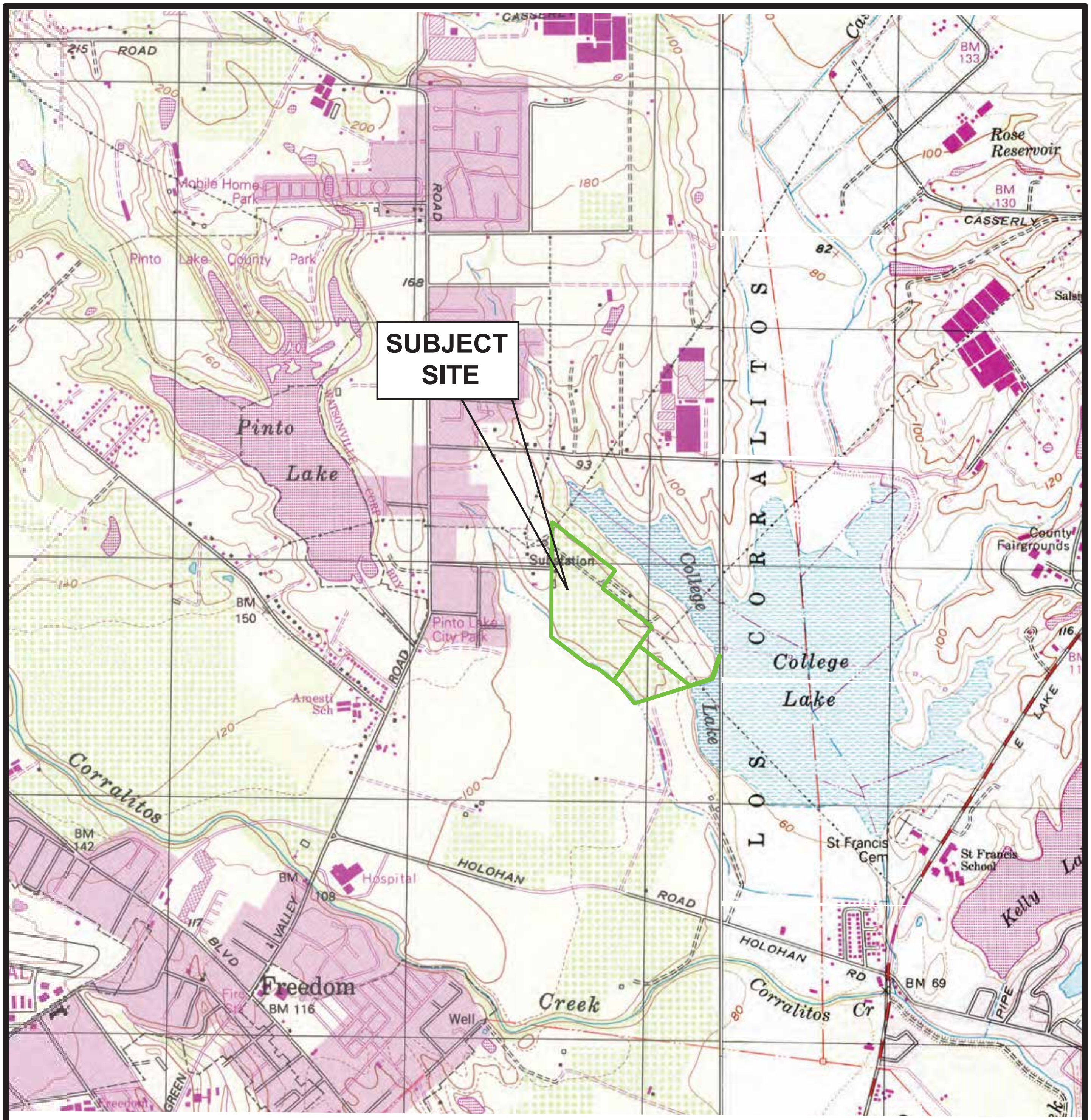
SCALE OF ACCEPTABLE RISKS FROM NON-SEISMIC GEOLOGIC HAZARDS[6]

Risk Level	Structure Type	Risk Characteristics
Extremely low risk	Structures whose continued functioning is critical, or whose failure might be catastrophic: nuclear reactors, large dams, power intake systems, plants manufacturing or storing explosives or toxic materials.	1. Failure affects substantial populations, risk nearly equals nearly zero.
Very low risk	Structures whose use is critically needed after a disaster: important utility centers; hospitals; fire, police and emergency communication facilities; fire station; and critical transportation elements such as bridges and overpasses; also dams.	1. Failure affects substantial populations. Risk slightly higher than 1 above.
Low risk	Structures of high occupancy, or whose use after a disaster would be particularly convenient: schools, churches, theaters, large hotels, and other high rise buildings housing large numbers of people, other places normally attracting large concentrations of people, civic buildings such as fire stations, secondary utility structures, extremely large commercial enterprises, most roads, alternative or non-critical bridges and overpasses.	1. Failure of a single structure would affect primarily only the occupants.
"Ordinary" risk	The vast majority of structures: most commercial and industrial buildings, small hotels and apartment buildings, and single family residences.	1. Failure only affects owners /occupants of a structure rather than a substantial population. 2. No significant potential for loss of life or serious physical injury. 3. Risk level is similar or comparable to other ordinary risks (including seismic risks) to citizens of coastal California. 4. No collapse of structures; structural damage limited to repairable damage in most cases. This degree of damage is unlikely as a result of storms with a repeat time of 50 years or less.
Moderate risk	Fences, driveways, non-habitable structures, detached retaining walls, sanitary landfills, recreation areas and open space.	1. Structure is not occupied or occupied infrequently. 2. Low probability of physical injury. 3. Moderate probability of collapse.

[6] Non-seismic geologic hazards include flooding, landslides, erosion, wave runup and sinkhole collapse

APPENDIX B

Figures 1 through 5

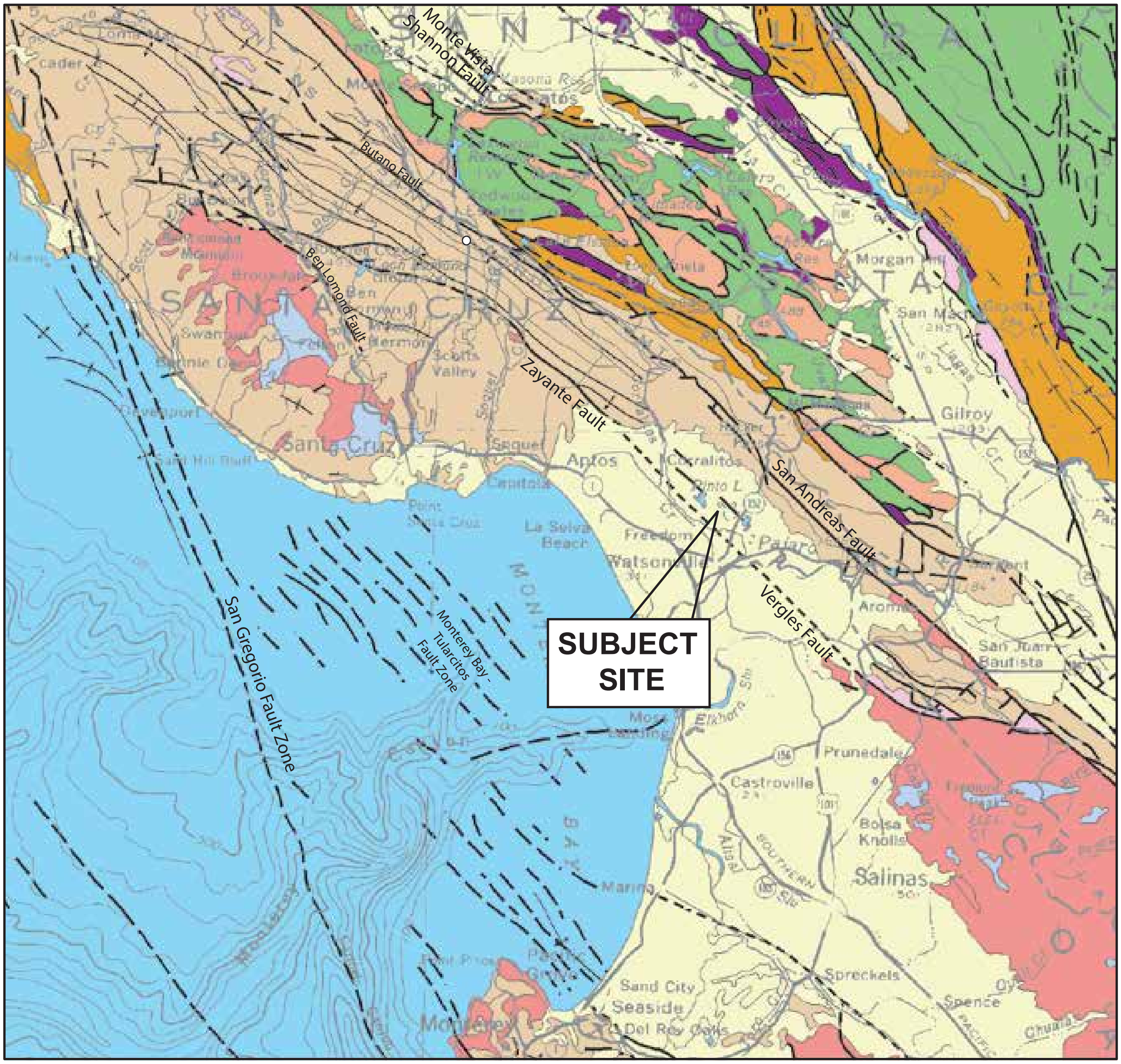


BASE MAP: WATSONVILLE WEST QUADRANGLE, California, 7.5 Minute Series, United States Geological Survey, 1995, scale 1:24,000 and WATSONVILLE EAST QUADRANGLE, California, 7.5 Minute Series, United States Geological Survey, 1995, scale 1:24,000.

BAYSIDE GEOLOGY

SITE LOCATION MAP
90 Minto Road, Watsonville, California
Santa Cruz County APN 051-101-77 & 78

FIGURE
1



EXPLANATION

Geologic Units

- Quaternary Deposits
- Quaternary Volcanics
- Tertiary Sedimentary Rocks
- Tertiary Volcanic Rocks
- Pre-Tertiary Sedimentary Rocks
- Pre-Tertiary Volcanic Rocks
- Granitic Intrusive Rocks
- Franciscan Complex
- Ultramafic Rocks
- Pre-Tertiary Metamorphic Rock
- Pre-Cambrian Metamorphic and Igneous Rocks

Symbols

- contact
- fault, certain
- fault,approx. located
- fault, concealed or inferred
- anticline
- monocline
- syncline

Reference: Jennings, C.W., 1977, Geologic Map of California: California Department of Conservation, Division of Mines and Geology, scale 1:750,000.

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N
W E
S

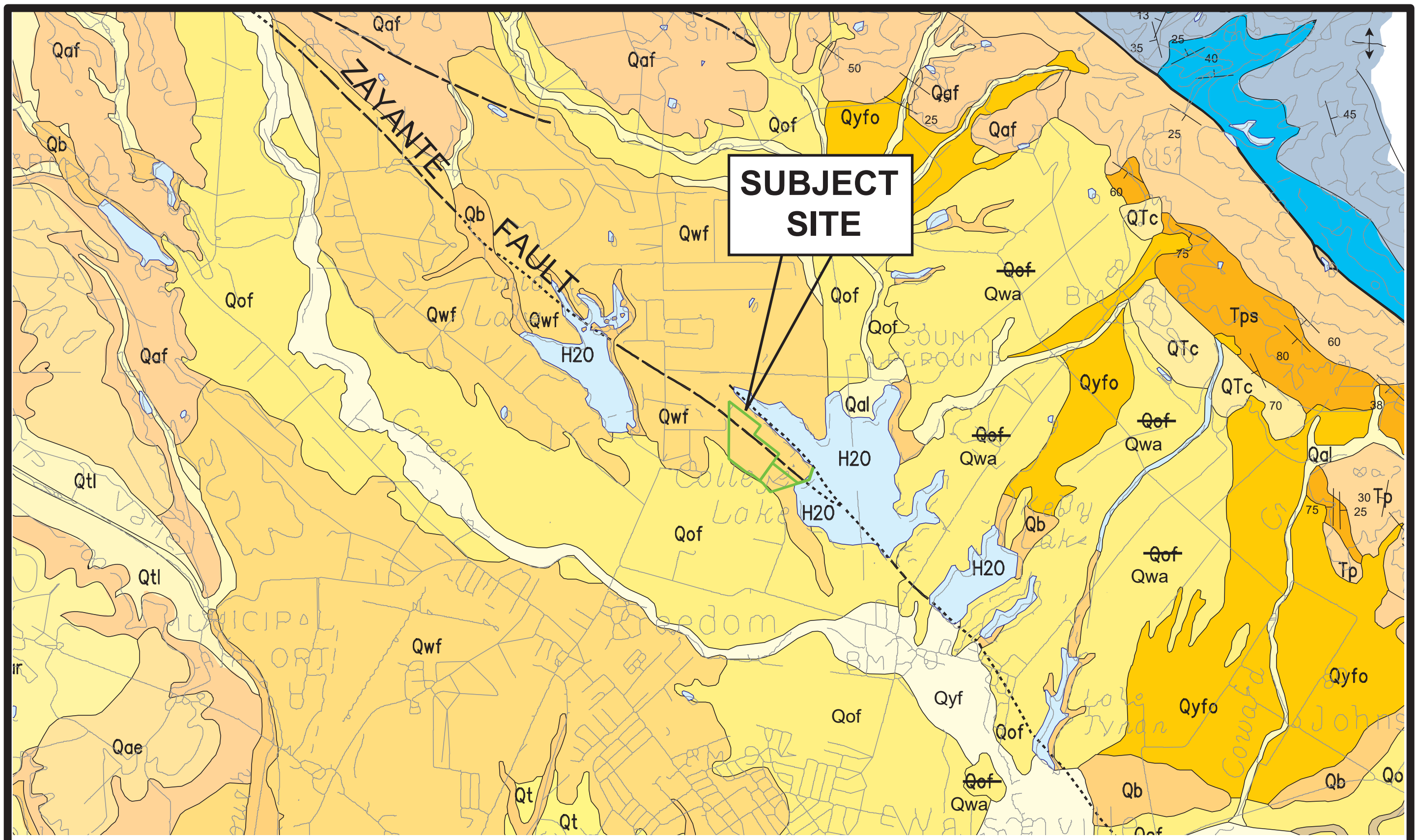
SCALE 1:500,000

10 Miles 0

BAYSIDE GEOLOGY

REGIONAL GEOLOGIC MAP
90 Minto Road, Watsonville, California
Santa Cruz County APN 051-101-77 & 78

FIGURE
2



EXPLANATION

Qtl	Colluvium (Holocene)
Qal	Alluvial deposits, undifferentiated (Holocene)
Qyf	Younger flood-plain deposits (Holocene)
Qof	Older flood-plain deposits (Holocene)
Qyfo	Alluvial fan deposits (Holocene)
Qb	Basin deposits (Holocene)
Qds	Dune sand (Holocene)
Qbs	Beach sand (Holocene)
Qt	Terrace deposits, undifferentiated (Pleistocene)
Qes	Eolian deposits of Sunset Beach (Pleistocene)
Qem	Eolian deposits of Manresa Beach (Pleistocene)
	Terrace deposits of Watsonville
Qwf	Fluvial facies (Pleistocene)
Qwa	Alluvial fan facies (Pleistocene)
Qcu	Coastal terrace deposits, undifferentiated (Pleistocene)
Qcl	Lowest emergent coastal terrace deposits
Qar	Aromas Sand, undivided (Pleistocene)
Qae	Eolian lithofacies
Qaf	Fluvial lithofacies
QTc	Continental deposits, undifferentiated (Pliocene?)
Tp	Purisima Formation (Pliocene and upper Miocene)
Tps	Predominantly massive sandstone
Tsc	Santa Cruz Mudstone (upper Miocene)
Tsm	Santa Margarita Sandstone (upper Miocene)
Tm	Monterey Formation (middle Miocene)

Contact
Fault
Anticline
Syncline
Strike and dip of beds
80 Inclined
50 Approximate dip of bedding
Vertical
Horizontal
60 Overturned
20 Strike and dip of foliation

BASE MAP: Brabb, E.E., 1997, Geologic map of Santa Cruz County, California: Digital Database, U. S. Geological Survey Open-File Report 97-489, scale 1:62,500.

N
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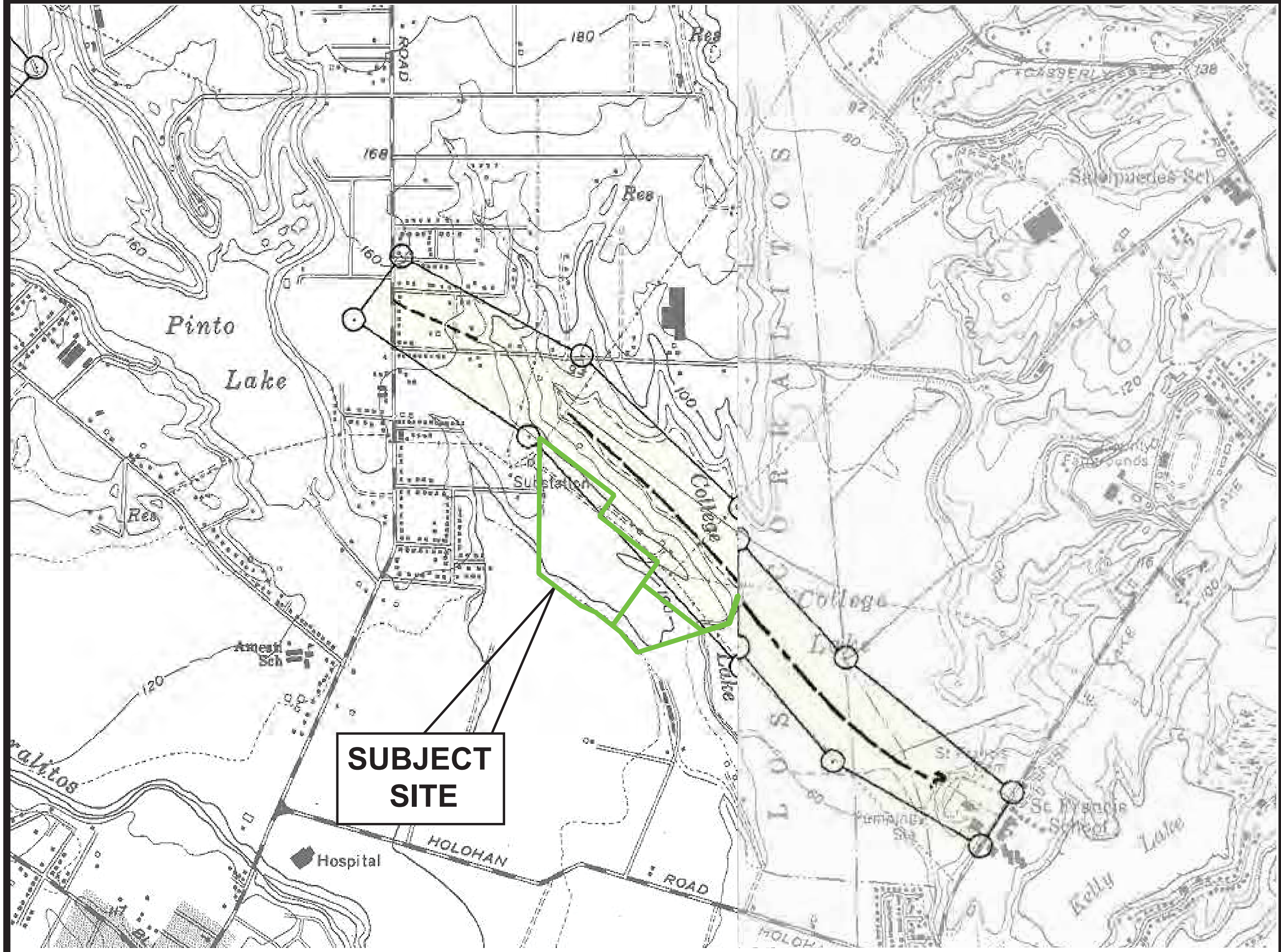
S

SCALE 1:62,500
0.5 0 0.5 1 mile

BAYSIDE GEOLOGY

USGS GEOLOGIC MAP
90 Minto Road, Watsonville, California
Santa Cruz County APN 051-101-77 & 78

FIGURE
3



EXPLANATION

1906 C ?	**ACTIVE FAULTS** Faults considered to have been active during Holocene time and to have a relatively high potential for surface rupture; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by creep or possible creep.
⊙———⊙	**FAULT ZONE BOUNDARIES** These are delineated as straight-line segments that connect encircled turning points so as to define fault zone segments.

BASE MAP: Gay, T.E., Jr., 1976, State of California Special Studies Zones Map, Watsonville West Quadrangle, California Division of Mines and Geology, scale 1:24,000. and Davis, J.F., 1982, State of California Special Studies Zones, Watsonville East Quadrangle, Revised Official Map, California Division of Mines and Geology, scale 1:24,000.

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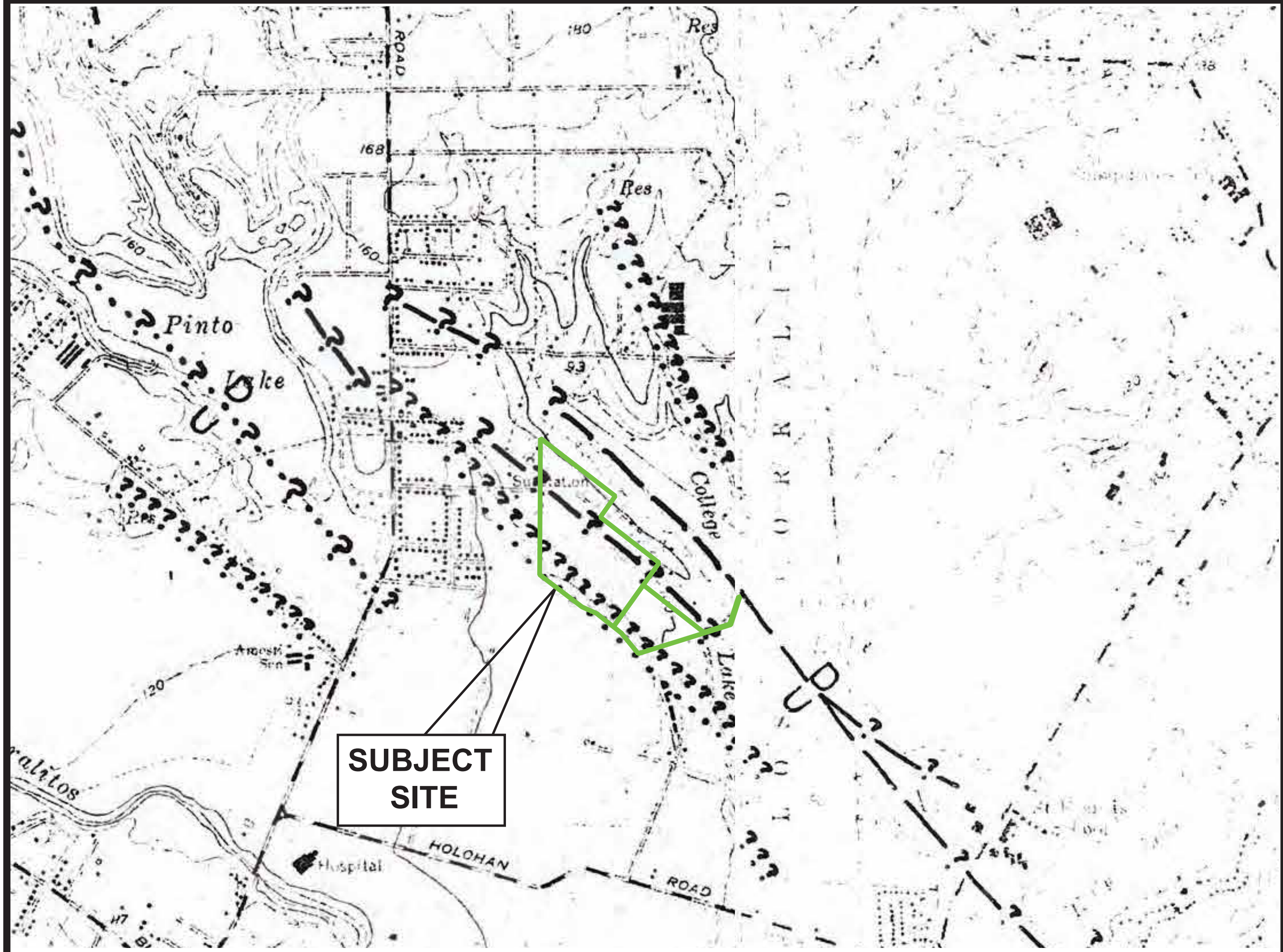
SCALE 1:24,000

1000 0 1000 2000 feet

BAYSIDE GEOLOGY

STATE FAULT MAP (Alquist-Priolo)
90 Minto Road, Watsonville, California
Santa Cruz County APN 051-101-77 & 78

FIGURE
4



Classification Of Faults

— — U/D — - ?··?	Probable fault - Dashed where exposed at earth's surface, dotted where concealed beneath younger deposits, vertical movement shown by U (up) and D (down).
?—?—?—?··?	Possible fault - Dashed where exposed at earth's surface, dotted where concealed beneath younger deposits.
? ? ? ? ? ? ? ? ? ?	Photolineament of unknown origin.

BASE MAP: WATSONVILLE EAST and WEST QUADRANGLES:
Hall, N.T., Sarna-Wojcicki, A.M., and Dupré, W.R., 1974,
Faults and Their Potential Hazards in Santa Cruz County,California,
U.S. Geological Survey Miscellaneous Field Studies Map MF-626,
3 sheets, scale 1:62,500.

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W E
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SCALE 1:24,000

1000 0 1000 2000 feet

BAYSIDE GEOLOGY	**COUNTY FAULT MAP** 90 Minto Road, Watsonville, California Santa Cruz County APN 051-101-77 & 78	FIGURE 5

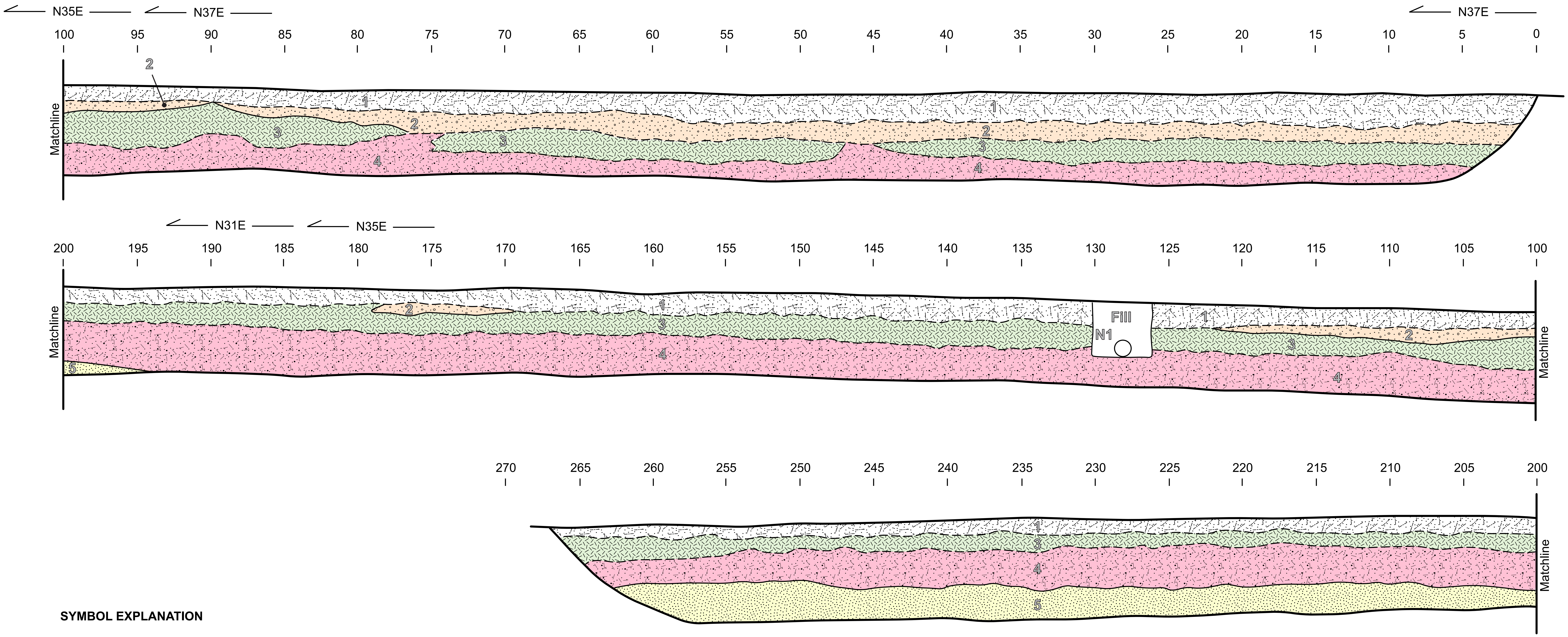
BAYSIDE GEOLOGY

SITE SUBSURFACE EXPLORATION MAP
90 Minto Road, Watsonville, California
Santa Cruz County APN 051-101-77 & 78

PLATE
1

BAYSIDE GEOLOGY	LOG of TRENCH 1 90 Minto Road, Watsonville, California Santa Cruz County APN 051-101-77 & 78	PLATE 2

TRENCH 1
Southeast wall logged
Scale 1”=5’, h=v



SYMBOL EXPLANATION

Sharp contact between units

? ?
Diffuse contact between units (between 0.1 to 2 inches); queried where uncertain

Gradational contact between units (greater than 2 inches); height of hachures approximately reflects measured transition between units

GENERAL NOTES

No evidence of faulting of any kind, no offset or truncation of units, no striated clay-lined surfaces, etc.

NOTES

N1 Unit contacts are through going and undisturbed, as observed on opposite wall.

UNIT DESCRIPTIONS

1- Sandy SILT (”A” Soil Horizon) - Very dark yellowish brown (Munsell Color: 10YR 3/2) contains few pebbles and widely scattered iron-oxide nodules, scattered micro-pores, few roots and rootlets, moist. Sand is generally fine grained. Includes 5 to 8 inch thick upper till zone.

2 - Pebbly SAND (Qwf - Terrace deposits of Watsonville - Fluvial facies, Pleistocene) - Light gray (Munsell Color: 10YR 7/2) contains some silt and clay as well as abundant pebbles and iron-oxide nodules, massive, moist. Sand is generally fine grained, rounded, and primarily composed of quartz. Pebbles are composed primarily of sandstone and mudstone with occasional chert and grantics. Iron-oxide nodules are 1/8 to 1/2 inch in diameter and concentrated at base of unit.

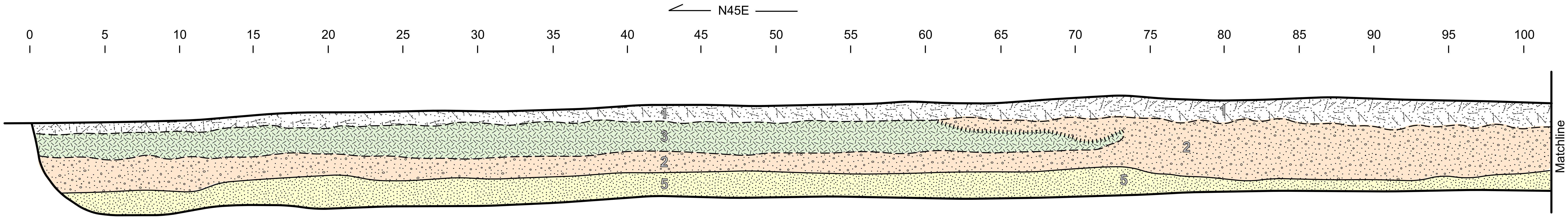
3 - Fat CLAY (Qwf - Terrace deposits of Watsonville - Fluvial facies, Pleistocene) - Olive brown (Munsell Color: 2.5Y 4/3) minor sand content varies laterally, massive, moist to wet.

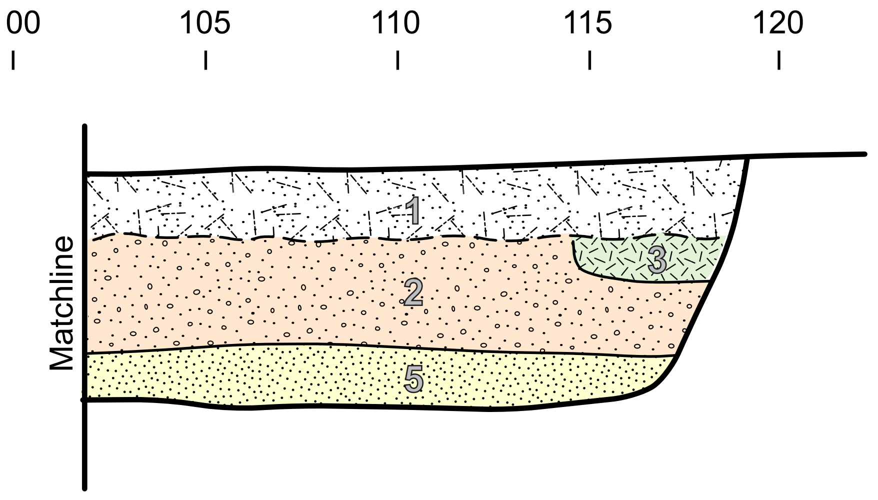
4 - Sandy CLAY / clayey SAND (Qwf - Terrace deposits of Watsonville - Fluvial facies, Pleistocene) - Brown (Munsell Color: 10YR 5/3) contains occasional pebbles, ratio of sand to clay varies vertically and laterally with structure, massive to laminated and cross bedded, well developed shrink/swell fabric, dry to wet. Sand is generally fine grained, but ranges from very fine to medium, is rounded, and primarily composed of quartz. Contains frequent paleo-liquefaction structures (sand blows, sills and dykes marked by secondary oxidation).

5 - SAND (Qwf - Terrace deposits of Watsonville - Fluvial facies, Pleistocene) - Yellowish brown (Munsell Color: 10YR 5/4) contains some silt and clay with few pebbles, thinly bedded to laminated, varies from poorly to well sorted, moist. Sand is generally fine grained, but ranges to coarse, is rounded, and primarily composed of quartz. Pebbles are composed primarily of sandstone and mudstone with occasional chert and grantics. Contains frequent paleo-liquefaction structures (sand blows, sills and dykes marked by secondary oxidation).

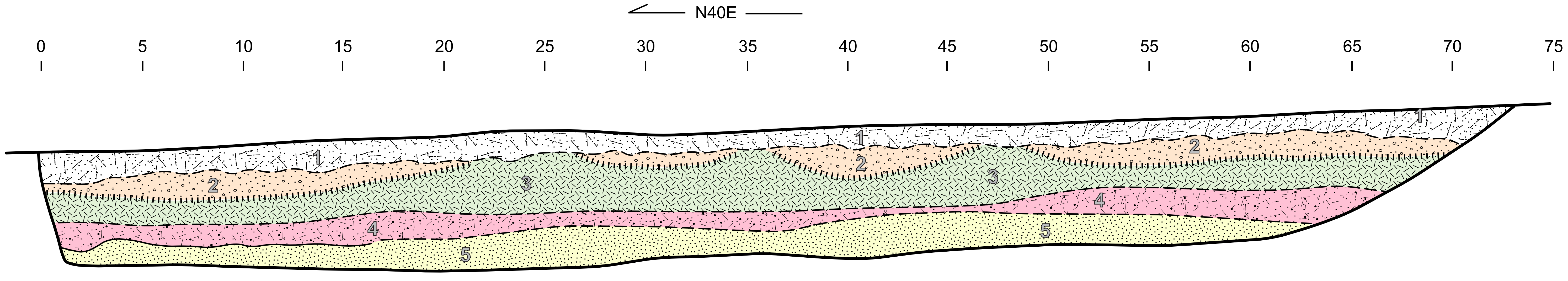
BAYSIDE GEOLOGY	**LOG of TRENCH 2 & 3** 90 Minto Road, Watsonville, California Santa Cruz County APN 051-101-77 & 78	PLATE 3

TRENCH 2
Southeast wall logged
Scale 1"=5', h=v





TRENCH 3
Southeast wall logged
Scale 1"=5', h=v



SYMBOL EXPLANATION

- ⁀ Sharp contact between units
- - - - -?-? Diffuse contact between units (between 0.1 to 2 inches); queried where uncertain
- IIIIIIIIII Gradational contact between units (greater than 2 inches); height of hachures approximately reflects measured transition between units

GENERAL NOTES

No evidence of faulting of any kind, no offset or truncation of units, no striated clay-lined surfaces, etc.

UNIT DESCRIPTIONS

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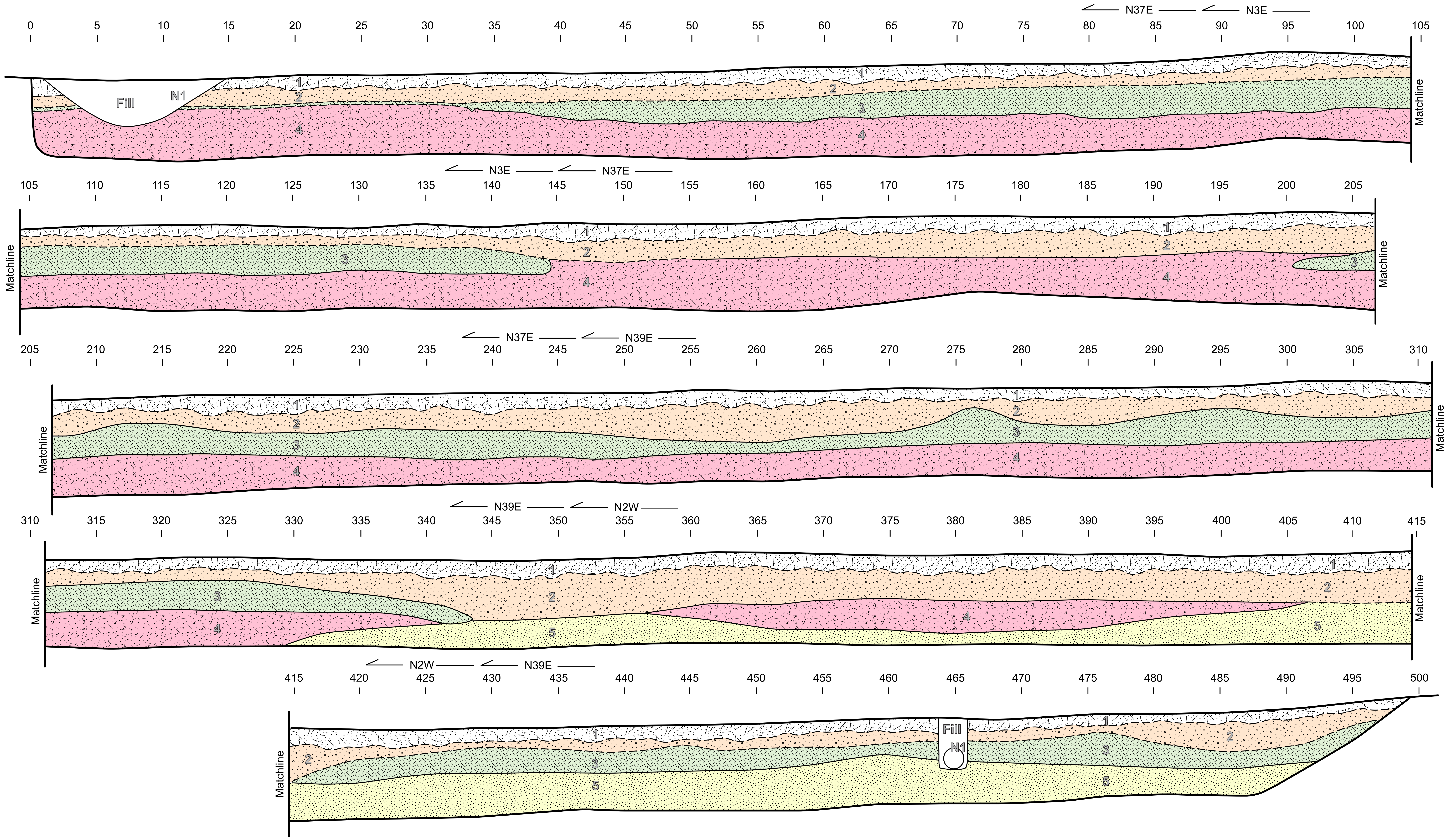
TRENCH 4
Southeast wall logged
Scale 1”=5’, h=v

BAYSIDE GEOLOGY

LOG of TRENCH 4
90 Minto Road, Watsonville, California
Santa Cruz County APN 051-101-77 & 78

PLATE
4



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¦¦¦¦¦¦¦¦¦¦	Gradational contact between units (greater than 2 inches); height of hachures approximately reflects measured transition between units

GENERAL NOTES

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NOTES

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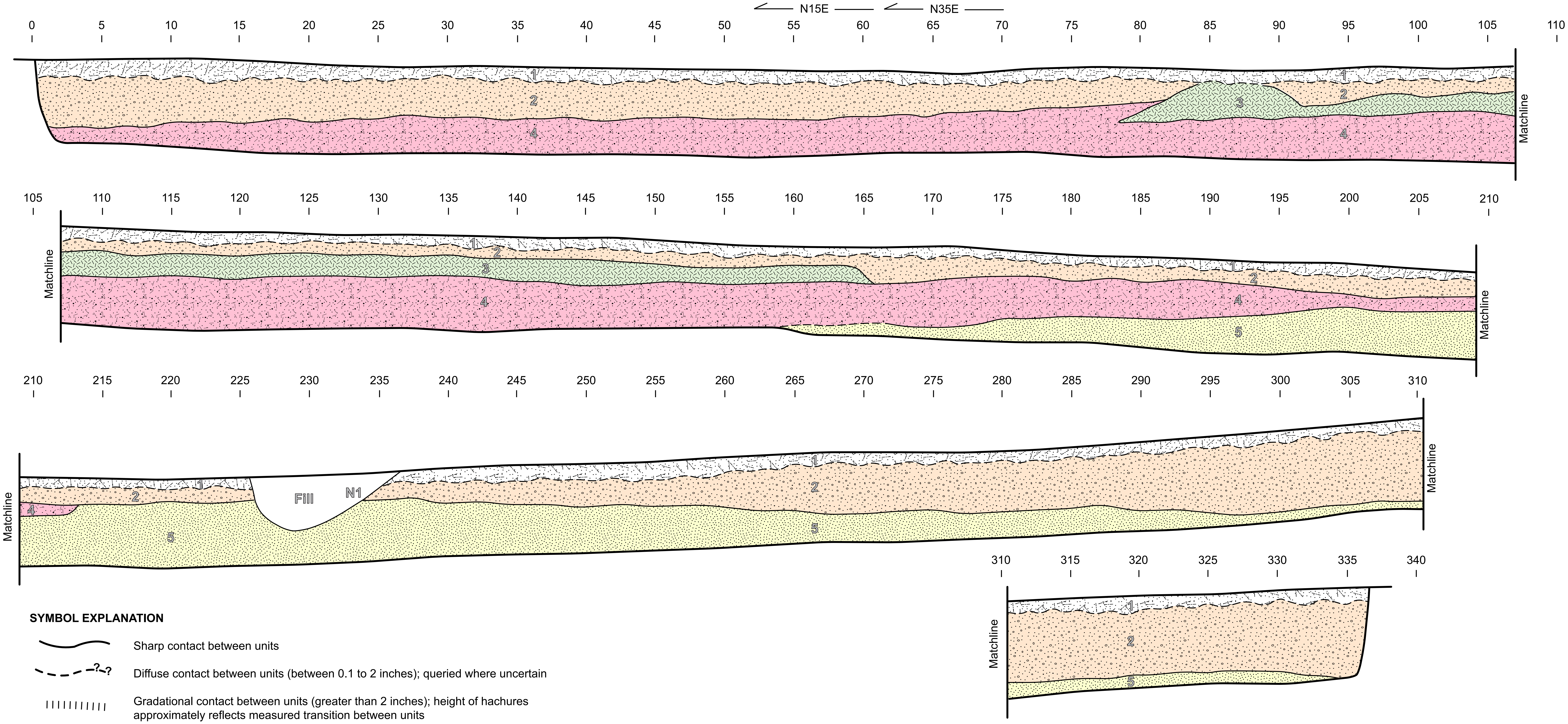
BAYSIDE GEOLOGY

LOG of TRENCH 5
90 Minto Road, Watsonville, California
Santa Cruz County APN 051-101-77 & 78

PLATE
5

TRENCH 5
Southeast wall logged
Scale 1"=5', h=v



SYMBOL EXPLANATION

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Diffuse contact between units (between 0.1 to 2 inches); queried where uncertain

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