Geotechnical Evaluation

2035 North Pacific Avenue
Santa Cruz, California

Slatter Construction

126 Fern Street | Santa Cruz, California 95060

May 21, 2018 | Project No. 403215001





Geotechnical | Environmental | Construction Inspection & Testing | Forensic Engineering & Expert Witness

Geophysics | Engineering Geology | Laboratory Testing | Industrial Hygiene | Occupational Safety | Air Quality | GIS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

May 21, 2018
Project No. 403215001

Mr. Sid Slatter
Construction
126 Fern Street
Santa Cruz, California 95060

Subject: Geotechnical Evaluation
2035 North Pacific Avenue
Santa Cruz, California

Dear Mr. Slatter:

In accordance with your authorization, we have performed a geotechnical evaluation for the construction of a three-story building with a below ground parking garage located at 2035 North Pacific Avenue in Santa Cruz, California. This report presents the findings and conclusions from our geotechnical evaluation and our geotechnical recommendations regarding the proposed project.

As an integral part of our role as the geotechnical engineer-of-record, we request the opportunity to review the construction plans before they go to bid and to provide follow-up construction observation and testing services.

Ninyo & Moore appreciates the opportunity to be of service to you on this project.

Sincerely,
NINYO & MOORE

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1 INTRODUCTION

In accordance with your request, we have prepared this geotechnical evaluation for the proposed three-story building with a below ground parking garage to be constructed at 2035 North Pacific Avenue in Santa Cruz, California (Figure 1). This report presents our findings and conclusions regarding the geotechnical conditions at the subject site, and our recommendations for the design and construction of this project.

2 SCOPE OF SERVICES

Our scope of services included the following:

- Reviewed readily available geologic and seismic literature pertinent to the project area including geologic maps and reports, regional fault maps, aerial photographs, environmental assessment reports, and seismic hazard maps.
- Performed site reconnaissance to observe the general site conditions and to mark the proposed locations for subsurface exploration.
- Coordinated with Underground Service Alert to locate the underground utilities in the vicinity of the proposed exploratory locations.
- Coordinated with the California Department of Toxic Substances Control (DTSC) regarding our proposed subsurface exploration.
- Prepared a site-specific health and safety plan for the subsurface exploration.
- Performed subsurface exploration consisting of three (3) exploratory borings, to depths of up to 20 feet below grade, and two (2) Cone Penetration Test (CPT) soundings to depths of up to 37 feet below the existing grade to evaluate the subsurface conditions. A representative of Ninyo & Moore logged the subsurface conditions exposed in the borings and collected bulk and relatively undisturbed soil samples for laboratory tests. The borings and soundings were backfilled with cement grout.
- Soil cuttings were collected and sealed in 55-gallon drums. We submitted samples for laboratory testing of hazardous contaminants and coordinated disposal of drums and cuttings accordingly based on the test results.
- Performed laboratory tests on selected soil samples to evaluate in-situ soil moisture content and density, grain size distribution, Atterberg limits, unconfined compressive strength, direct shear strength, and corrosivity.
- Performed engineering geologic mapping of the surficial geologic conditions at the site.
- Conducted data compilation and engineering analysis of the information obtained from our background review, subsurface evaluation, geologic mapping, and laboratory testing.
- Prepared this geotechnical report presenting our findings, conclusions, and geotechnical recommendations for the project.

3 SITE DESCRIPTION

The project site is located at 2035 North Pacific Avenue, assessor's parcel number 006-361-24, in Santa Cruz, California (Figure 1). The site is located at approximately 36.9782 degrees north latitude and 122.0273 degrees west longitude.

The site is currently occupied by a one-story office building in the center portion of the site with an asphaltic concrete (AC) parking lot on either side of the building. The site is bounded to the north by a commercial property at 201 River Street, to the south by a mixed-use commercial and residential property at 2027 North Pacific Avenue, to the west by a steeply inclined slope, and to the east by North Pacific Avenue. The ground surface in the vicinity of the project ranges from an elevation of about 20 to 22 feet above mean sea level [MSL] in the parking lot and building areas (Google Earth, 2018). The elevation of the top of the slope to the west of the site is about 85 feet MSL (Google Earth, 2018). The San Lorenzo River is located approximately 700 feet to the northeast of the project site. Elevation gradients from the site toward the San Lorenzo River are relatively flat, generally less than 1 percent (Google Earth, 2018).

4 BACKGROUND DOCUMENT REVIEW

Based on a review of documents, the site was formerly occupied by manufactured gas plant (MGP) with associated structures that operated from about 1867 through 1930 (Terra Pacific Group, 2016). By the 1960's, most of the above ground MGP structures had been removed and by 1988, the office building currently located at the site had been constructed. Based on a review of aerial photographs, the building at 2027 North Pacific Avenue was constructed around 2006 and the structure across the street at 2030 North Pacific Avenue was constructed around 2007. The building at 2027 North Pacific Avenue was constructed to replace a building that was damaged during the 1989 Loma Prieta earthquake (Terra Pacific Group, 2016).

Based on the site usage history, numerous environmental assessments have been performed to evaluate potential soil and groundwater contamination at the site. Remediation activities were performed from September 4, 2012 through February 12, 2013, which included removal of soil in select locations to varying depths of up to 13 feet (Terra Pacific Group, 2016). During the excavations, a buried concrete gas holder foundation was encountered which was approximately 50 feet in diameter and extended to a depth of about 15 feet below the ground surface. The concrete was left in place as part of the remediation construction (Terra Pacific Group, 2016).

5 PROPOSED CONSTRUCTION

We understand that the proposed improvements will consist of a three-story building with an underground parking garage. Based on a review of the conceptual design plans provided (William S. Bagnall Architects Inc., 2017), the structure will have a building footprint of about 9,900 square feet and consist of an underground parking garage with 30 parking spaces, office space on the first floor, and residential units on the second and third floors. Building loads were not provided, but we assume foundation loads will be light to moderate.

6 SUBSURFACE EVALUATION AND LABORATORY TESTING

Our field exploration for this study included a site reconnaissance and a subsurface exploration conducted on March 10 and 13, 2018. The subsurface exploration consisted of two (2) Cone Penetrometer Test (CPT) soundings and three (3) small-diameter auger borings. The approximate locations of the exploration are presented on Figure 2.

The borings were advanced to depths of up to 20 feet below existing grade. A representative of Ninyo & Moore logged the subsurface conditions exposed in the borings and collected bulk and relatively undisturbed soil samples from the borings. The samples were then transported to our geotechnical laboratory for testing. The borings were backfilled with cement grout shortly after excavation. Descriptions of the subsurface materials encountered are presented in the following sections. Detailed logs of the borings are presented in Appendix A.

The CPT soundings were performed on March 10, 2018 using a truck-mounted rig with a 20-ton reaction capacity. The soundings were advanced until refusal was encountered at depths of about 18 feet (CPT-2) and 38 feet (CPT-1) below the existing grade. Cone tip resistance, sleeve friction, and pore pressure were electronically measured and recorded at vertical intervals of approximately 2 inches while the cone was advanced. The soil behavior type index (I_c) and corresponding soil behavior for the subsurface materials encountered was assessed using correlations (Robertson & Campanella, 1986) based on the cone penetration data and sleeve friction. The CPT sounding logs are presented in Appendix B.

Laboratory testing of soil samples recovered from the borings included tests to evaluate in-situ soil moisture content and density, grain size distribution, Atterberg limits, unconfined compressive strength, direct shear strength, and corrosivity. The results of the in-place soil moisture and density are shown at the corresponding sample depths on the boring logs in Appendix A. The results of the other laboratory tests are presented in Appendix C.

7 GEOLOGIC AND SUBSURFACE CONDITIONS

7.1 Regional Geologic Setting

The site is located north of Monterey Bay in the Coast Ranges geomorphic province of California. The Coast Ranges are comprised of several mountain ranges and structural valleys formed by tectonic processes commonly found around the Circum-Pacific belt. Basement rocks have been sheared, faulted, metamorphosed, and uplifted, and are separated by thick blankets of Cretaceous and Cenozoic sediments that fill structural valleys and line continental margins. The San Francisco Bay area has several ranges that trend northwest, parallel to major strike-slip faults such as the San Andreas, Hayward, and Calaveras (Figure 3). Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement.

7.2 Site Geology

Published geologic maps indicate that the site vicinity is generally underlain by Miocene age Santa Cruz Mudstone (Tsc) described as medium to thick bedded and faintly laminated, weathered, pale yellowish brown siliceous organic mudstone (Brabb et. al., 1997). Areas to the east of the site are underlain by Holocene age alluvium described as unconsolidated, heterogeneous, moderately sorted silt and sand containing discontinuous lenses of clay and silty clay (Brabb et. al., 1997). A map of the regional geology is presented as Figure 4. The results of our subsurface exploration indicate that the project site is underlain by alluvium and Santa Cruz Mudstone.

Based on our geologic mapping of the slope to the west of the project site, the slope is comprised of Santa Cruz Mudstone bedrock. The Santa Cruz Mudstone exposed on the slope consists of light brown, weathered, intensely to moderately fractured siltstone and mudstone. The bedrock is thickly bedded to massive, and where observed, bedding is near horizontal. Fractures within the rock mass are typically steeply dipping and intersect to form wedges and blocks. There are several locations on the slope where wedge and block failures have occurred. Overhanging conditions are also present on the slope in localized areas.

7.3 Subsurface Conditions

The following sections provide a generalized description of the geologic units encountered during our subsurface evaluation. More detailed descriptions are presented on the boring logs in Appendix A. A cross section depicting our interpretation of the subsurface conditions is presented as Figure 5.

7.3.1 Pavement Section

A pavement section consisting of 2 to 8 inches of asphaltic concrete (AC) over 2 to 8 inches of aggregate base (AB) was encountered in the five (5) subsurface explorations.

7.3.2 Fill

Fill was observed in the borings and CPT soundings from below the pavement section to depths that ranged from about 1½ feet to 5 feet below the ground surface. Where encountered, the fill consisted of brown to light brown, moist, firm lean clay; and moist, dense, poorly-graded sand and clayey sand.

7.3.3 Alluvium

Alluvium was encountered in Boring B-3 from below the fill to a depth of about 8 feet below the ground surface. Where encountered in our boring, the alluvium consisted of brown, moist, firm, silt. Alluvium was encountered in the CPT soundings from below the fill to depths of about 15 feet (CPT-2) and 24 feet (CPT-1) and was generally classified as silty sand to sandy silt.

7.3.4 Bedrock

Santa Cruz Mudstone bedrock was encountered in Boring B-1 at a depth of 3 feet, Boring B-2 at a depth of 2 feet, Boring B-3 at a depth of 8 feet, CPT-1 at a depth of 24 feet, and CPT-2 at a depth of 15 feet. As encountered in the borings, the bedrock generally consisted of brown to gray, moist, weathered mudstone. The mudstone varied from relatively weak to strong rock. Standard penetration test (SPT) sampling refusal was encountered in the bedrock at depths of 9 and 9½ feet in Borings B-1 and B-2, respectively.

7.4 Groundwater

Groundwater was measured at a depth of about 16 feet in both CPT sounding locations. Groundwater was not encountered in the other borings. Based on a review of available subsurface data, groundwater is generally about 10 to 16 feet below the ground surface and flows parallel to the contour of the relatively impermeable bedrock (Terra Pacific Group, 2016). Fluctuations in the groundwater level may occur due to seasonal precipitation, variations in topography or subsurface hydrogeologic conditions, creek flow, or as a result of changes to nearby irrigation practices or groundwater pumping. In addition, seeps may be encountered at elevations above the groundwater levels encountered due to perched groundwater conditions, leaking pipes, preferential drainage, or other factors not evident at the time of our exploration. Piezometers can be installed to further evaluate the depth to groundwater in the study area and fluctuation in groundwater levels if needed.

8 SEISMIC HAZARDS AND GEOTECHNICAL CONSIDERATIONS

8.1 Seismic Hazards

The seismic hazards considered in this study include the potential for ground rupture due to faulting, seismic ground shaking, liquefaction, dynamic settlement, seismic slope stability, and tsunamis. These potential hazards are discussed in the following subsections.

8.1.1 Faulting and Ground Surface Rupture

There are numerous recognized faults in northern California. Selected characteristics, as evaluated by the 2007 Working Group on California Earthquake Probabilities (WGCEP, 2008), for recognized and postulated faults (Caltrans, 2018) near the site are presented in Table 1. The fault characteristics in the table are presented in order of decreasing peak ground acceleration (PGA) based on a deterministic seismic hazard analysis utilizing the Chiou & Youngs (2008) and Campbell & Bozorgnia (2008) attenuation relationships.

Table 1 – Parameters for Nearby Faults

Fault	ID	Type	Max Moment Magnitude	Distance to Site (kilometers)
Zayante-Vergeles Upper	162	Strike Slip	7.0	9.9
San Andreas (Santa Cruz Mtns)	158	Strike Slip	8.0	17.7
Monterey Bay-Tularcitos (Monterey Bay section)	174	Strike Slip	7.2	10.9
Zayante-Vergeles Upper 2011 CFM	162	Strike Slip	7.0	11.3
San Gregorio (San Gregorio)	127	Strike Slip	7.4	16.3
San Gregorio fault zone (Sur Region section-Sur fault)	178	Strike Slip	7.4	17.6
Sargent fault (northwestern section)	164	Strike Slip	7.0	16.4
San Andreas (Peninsula)	134	Strike Slip	8.0	33.5
Monterey Bay-Tularcitos (Seaside-Monterey section)	191	Strike Slip	7.2	34.0
San Gregorio fault zone (Sur Region section-Palo Colorado fault	190	Strike Slip	7.4	41.3

The site is not located within an Alquist-Priolo Earthquake Fault Zone established by the state geologist (CGS, 2007) to delineate regions of potential ground surface rupture adjacent to active faults. As defined by the California Geological Survey (CGS), active faults are faults that have caused surface displacement within Holocene time, or within approximately the last 11,000 years (CGS, 2007). The closest fault rupture hazard zone is the one associated with the San Andreas Fault, which is located 11 miles northeast of the site.

Based on our review of the referenced geologic maps, there are no known faults at the project site, and the site is not located within a fault rupture hazard zone. Therefore, the probability of damage from surface fault rupture is considered to be low.

8.1.2 Strong Ground Motion

Based on historic activity, the potential for future strong ground motion in the project area is considered significant. Design recommendations for structures to address seismic shaking are presented in Section 10.2. The 2016 California Building Code (CBC) specifies that the potential for liquefaction and soil strength loss be evaluated, where applicable, for the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration with adjustment for site class effects in accordance with the American Society of Civil Engineers (ASCE) 7-10 Standard. The MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) was calculated as 0.500g using the USGS seismic design tool (USGS, 2018) that yielded a mapped MCE_G peak ground acceleration of 0.500g for the site and a site coefficient (F_{PGA}) of 1.00 for Site Class D.

8.1.3 Liquefaction and Strain Softening

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

The site is in an area where the California Geological Survey has not yet evaluated or established seismic hazard zones for liquefaction. City of Santa Cruz hazard maps indicate the site is located within an area considered to have a very high susceptibility for liquefaction during a major earthquake event (Figure 6). Liquefaction was documented in many areas of the downtown Santa Cruz area following the 1989 Loma Prieta earthquake and resulted in major damage to structures, sand boil settlement, buckling of pavements, and damage to underground utilities (USGS, 1998).

We encountered deposits of sand and fine-grained soil of low plasticity below the groundwater level during our subsurface exploration. We evaluated the potential for liquefaction using in-house developed spreadsheets developed in accordance with the methods presented by Idriss and Boulanger (2008) using the CPT data collected during our subsurface exploration, a design groundwater level of 10 feet below the ground surface, and considering a seismic event producing a PGA of 0.500g resulting from a Magnitude 7.0 earthquake. The results of our analysis, presented in Appendix D, indicate that sand and fine-grained soil of low plasticity below the assumed design groundwater level will liquefy under the considered ground motion based on a factor of safety against liquefaction of less than one. Selection of foundation bearing levels or ground improvement to mitigate the potential liquefaction-induced reduction in the bearing capacity of shallow foundations is a design consideration for the project. Other consequences of liquefaction, including dynamic settlement, sand-boil induced ground subsidence, and lateral spreading, are addressed in the following sections.

We did not encounter cohesive soil during our subsurface exploration. As such, we do not regard seismically induced strain-softening behavior as a design consideration.

8.1.4 Dynamic Settlement

The strong vibratory motion associated with earthquakes can also dynamically compact loose granular soil leading to surficial settlements. Dynamic settlement is not limited to the near-surface environment and may occur in both dry and saturated sand and silt. Cohesive soil is not typically susceptible to dynamic settlement.

During our subsurface exploration, we encountered granular soil in our CPT soundings. We evaluated the potential for dynamic settlement based on the procedure described by Zhang et al (2002) for saturated soil and by Robertson and Shao (2010) for dry soil. Our analysis considered a Magnitude 7.0 earthquake producing a PGA of 0.500g and groundwater level 10 feet below the ground surface. The results of our analyses, presented in Appendix D, indicate that the total free-field volumetric dynamic settlement following the considered seismic event will be up to approximately 2¾ inches following the considered seismic event. Differential dynamic settlement is estimated to be on the order of about 1⅜ inches over a horizontal distance of 30 feet. Recommendations for remedial grading with a mat slab, deep foundations, or ground improvement are provided to mitigate the dynamic settlement for significant structures. Repairing damage to pavements, flatwork, utilities, and minor structures such as equipment pads and minor retaining walls is typically the preferred approach to addressing dynamic settlement given the low risk to public safety. Ground improvement can be performed across the site to reduce the dynamic settlement and improve the seismic performance of the appurtenant hardscape, minor structures, and utilities.

8.1.5 Ground Subsidence

Sand boils that occur when liquefied, near-surface soil escapes to the ground surface, can result in ground subsidence due to loss of material that is in addition to dynamic settlement. Based on the design ground motion, relative thickness and depth of the saturated, loose granular soil encountered during our subsurface exploration, and case study data presented by Ishihara (1985), sand boils and resulting ground subsidence is a design consideration. Recommendations for remedial grading with a mat slab, deep foundations, or ground improvement are provided for significant structures. Repairing damage to pavements, flatwork, utilities, and minor structures such as equipment pads and minor retaining walls is typically the preferred approach to addressing ground subsidence given the low risk to public safety. Ground improvement can be performed across the site to reduce the ground subsidence and improve the seismic performance of the appurtenant hardscape, minor structures, and utilities.

8.1.6 Lateral Spreading

In addition to vertical displacements, seismic ground shaking can induce horizontal displacements as surficial soil deposits spread laterally by floating atop liquefied subsurface layers. For lateral spreading to occur the layer of liquefied soil must have lateral continuity. Lateral spread can occur on sloping ground or on flat ground adjacent to an exposed face. Based on empirical predictive relationships developed by Youd, et al (2002) and derived from case study records for lateral spreading, lateral spreading tends to occur where the soil susceptible to liquefaction has an overburden-corrected, equivalent SPT penetration resistance of less than 15 with a cumulative thickness of 1 meter or more. The topography of the project site is relatively flat and a free-face condition does not exist near the proposed improvements. The San Lorenzo River is located approximately 700 feet to the northeast of the project site. Elevation gradients from the site toward the San Lorenzo River are relatively flat (City of Santa Cruz, 2018 and Google Earth, 2018). Consequently, we do not regard lateral spreading as a design consideration.

8.2 Landsliding and Slope Stability

The site is bounded by a slope to the west which is up to approximately 60 feet in height and sloped at an inclination of about 50 to 60 degrees from horizontal (City of Santa Cruz, 2018). This slope extends for several hundred feet north and south of the property and is covered with various types of shrubs and trees. Portions of the slope north and south of the subject property are also covered with rock netting, which is used to mitigate surficial slope failures. On the subject property, a small portion of the slope located near the southern property line is covered with rock netting, while the rest of the slope does not have rock netting or other slope stabilization devices. The rock netting at the southern end of the slope extends approximately 30 feet to the north of the southern property boundary.

Based on our review of available maps, on the subject property the western property line is located near the toe of the slope. A retaining wall is located near the toe of the slope that is up to about 3 feet in height. Talus deposits consisting of soil, rock, and vegetation lie along the toe of the slope above the retaining walls. These deposits were generated by erosion and surficial slope failures including wedge and rectangular block type failures. Evidence of previous wedge and block type failures are present on portions of the slope where there is no rock netting and measure up to 15 feet across in greatest dimension. Based on our observations, the wedges and blocks failed along well developed continuous fractures and joints within the rock mass. Material observed at the toe of the slope included blocks up to several feet in size. These types of failures will continue to occur over time and should be considered during design of the project.

Since much of the slope lies outside the property limits, catchment structures along the western property are considered a feasible solution to mitigate the potential hazard. If easements were obtained on the neighboring properties, a rockfall-netting system could be installed to mitigate the potential hazard. Consideration should be given to providing a building setback from the toe of slope to allow for future maintenance on the catchment fences.

8.3 Static Settlement

The results of our subsurface exploration indicate that the alluvium deposits encountered below the proposed building location included layers of loose to medium dense silt and sand. Static settlement due to sustained loads is a design consideration for moderate to heavy structures. Differential static settlement is a design consideration due to the variable depth of bedrock beneath the site. Recommendations for remedial grading with a mat slab, deep foundations, or ground improvement are provided to reduce the potential static settlement for moderate to heavily loaded structures. Static settlements due to pad fills or embankments are not design considerations as no embankments or other large surcharges are proposed for the project.

8.4 Unsuitable Materials

Fill materials that were not placed and compacted under the observation of a geotechnical engineer, or fill materials lacking documentation of such observation, are considered undocumented fill. Undocumented fill is unsuitable as a bearing material below foundations due to the potential for differential settlement resulting from variable support characteristics or the potential inclusion of deleterious materials. Undocumented fill was encountered up to depths of about 5 feet below the ground surface during our subsurface exploration. Based on the historical site usage, undocumented fill, contaminated soil, and buried concrete structures should be anticipated to depths of 15 feet, or deeper, at the project site. Fill was placed as part of the environmental remediation work performed at the site. Based on the Terra Pacific Group report (2016), the fill was observed and tested by a geotechnical engineer. Recommendations for subgrade preparation and foundation embedment are provided to mitigate the undocumented fill concerns.

Soil containing roots or other organic matter are not suitable as fill or subgrade material below foundations, pavements, or engineered fill. Recommendations for clearing and grubbing to remove vegetative matter in soil during site preparation are provided.

8.5 Excavation Characteristics

We anticipate that the project will involve excavations of depths up to about 5 feet for grading and utility installation, 15 feet for the building underground parking level, and up to about 40 feet for deep foundations or ground improvement. The geologic materials encountered during our subsurface evaluation include fill, alluvium, and bedrock. The alluvium materials generally consisted of firm silt and loose to medium dense silty sand and sandy silt. The bedrock generally consisted of weathered mudstone that varies from relatively weak to strong rock. We anticipate that heavy earthmoving equipment in good working condition should be able to make the proposed excavations. Difficult drilling conditions may be encountered in the bedrock materials.

Based on the historical site usage, undocumented fill, contaminated soil, and buried concrete structures should be anticipated to depths of 15 feet, or deeper, at the project site. Excavations in the fill may encounter obstructions consisting of debris, rubble, abandoned structures, or over-sized materials that may require special handling or demolition equipment for removal.

Near-vertical temporary cuts in the fill or alluvial deposits should not be considered stable. Near-vertical temporary cuts in the bedrock deposits up to 4 feet in depth should remain stable for a limited period of time. Sloughing of the materials exposed on the excavation sidewall may occur, particularly if the excavation extends near the groundwater level, encounters granular soil, is exposed to water, or if the sidewall is disturbed during construction operations. Excavation subgrade may become unstable if exposed to wet conditions. Recommendations for excavation stabilization are presented. Excavated materials may also be wet and need to be dried out before reuse as fill.

8.6 Expansive Soil

Some clay minerals undergo volume changes upon wetting or drying. Unsaturated soils containing those minerals will shrink/swell with the removal/addition of water. The heaving pressures associated with this expansion can damage structures and flatwork. We did not encounter cohesive clay soils near the elevation of the proposed underground parking structure, consequently, expansive soils are not a design consideration for the structure. Based on previous environmental remediation work performed at the site, variation in near-surface soils should be anticipated and expansive clay could be present in areas of proposed hardscape or pavement.

8.7 Corrosive/Deleterious Soil

An evaluation of the corrosivity of the on-site material was conducted to assess the impact to concrete and metals. The corrosion impact was evaluated using the results of limited laboratory testing on samples obtained during our subsurface study. Laboratory testing to quantify pH, resistivity, chloride, and soluble sulfate contents was performed on a sample of the near-surface soil. The results of the corrosivity tests are presented in Appendix C. Based on the Caltrans (2018) corrosion criteria, a project site is classified as corrosive if one or more of the following conditions exist for the representative soil samples retrieved from the site: chloride concentration of 500 ppm or greater, soluble sulfate concentration of 1,500 ppm or greater, electrical resistivity of 1,100 ohm-centimeters or less, a pH of 5.5 or less, and an area within 1,000 feet of brackish water. Based on these criteria, the site does not meet the definition of a corrosive environment. Ferrous metal will still undergo corrosion on site, but special mitigation measures are not needed. The criteria used to evaluate the deleterious nature of soil on concrete and recommendations from the American Concrete Institute (ACI) for sulfate exposure classes are presented in Table 2. Based on these criteria, the soil on site is defined as Exposure Class S0.

Table 2 – Criteria for Deleterious Soil on Concrete

Sulfate Content Percent by Weight	Exposure Class	Maximum Water to Cement Ratio	Minimum 28-day Compressive Strength
0.0 to 0.1	S0	N/A	2,500
0.1 to 0.2	S1	0.50	4,000
0.2 to 2.0	S2	0.45	4,500
> 2.0	S3	0.45	4,500

Reference: American Concrete Institute (ACI) Committee 318 Table 19.3.1.1 and Table 19.3.2.1 (ACI, 2014)

8.8 Flood Hazards

Our review of Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FEMA, 2009) found that the eastern portion of the site lies within an area designated as A99 which is described as an area to be protected from 1 percent annual chance flood event by a Federal flood protection system under construction.

9 CONCLUSIONS

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

- Our subsurface evaluation indicated that the project site is underlain by fill, alluvium and bedrock. The fill encountered consisted of brown to light brown, moist, firm lean clay; and moist, dense, poorly-graded sand and clayey sand. The alluvium under the fill generally consisted of brown, moist, firm, silt, sandy silt, and silty sand. The bedrock encountered generally consisted of brown to gray, moist, weathered mudstone which varied from relatively weak to strong rock.
- Groundwater was measured at a depth of about 16 feet in both CPT sounding locations. Groundwater was not encountered in the other borings. Based on a review of available subsurface data, groundwater is generally about 10 to 16 feet below the ground surface. Fluctuations in the groundwater levels may occur as discussed in Section 7.4.
- The project site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zone). Based on our review of published geologic maps, no surface traces of known active or potentially active faults are present along the site and the potential for surface fault rupture is considered to be low.
- The site could experience relatively strong ground shaking during a significant earthquake on a nearby fault. Recommendations for seismic design criteria are provided.
- City of Santa Cruz hazard maps indicate the site is located within an area considered to have a very high susceptibility for liquefaction. Our analysis indicates that the site is underlain by saturated sand and fine-grained soil of low plasticity below the assumed design groundwater level that will liquefy under the considered ground motion.
- The results of our analyses indicate that the total dynamic settlement following the considered seismic event will be up to approximately 2¾ inches following the considered seismic event. Differential dynamic settlement is estimated to be on the order of about 1⅜ inches over a horizontal distance of 30 feet. Recommendations for remedial grading with a mat slab, deep foundations, or ground improvement are provided to mitigate the dynamic settlement for significant structures.
- Based on the design ground motion, relative thickness and depth of the saturated, loose granular soil encountered during our subsurface exploration, and case study data presented by Ishihara (1985), sand boils and resulting ground subsidence is a design consideration. Recommendations for remedial grading with a mat slab, deep foundations, or ground improvement are provided for significant structures.
- The results of our analyses indicate that lateral spreading is not a design consideration.
- Based on our analysis and geologic mapping, surficial slope stability is a design consideration. Recommendations have been provided for a rock catchment fence along the

toe of the slope and for shoring of excavations for the underground parking garage construction.

- Static settlement due to sustained loads is a design consideration for moderate to heavy structures. Differential static settlement is a design consideration due to the variable depth of bedrock beneath the site.
- Undocumented fill is unsuitable as a bearing material below foundations. Undocumented fill was encountered up to depths of about 5 feet below the ground surface during our subsurface exploration. Based on the historical site usage, undocumented fill, contaminated soil, and buried concrete structures should be anticipated to depths of 10 feet, or deeper, at the project site.
- The earth materials underlying the site should be excavatable with conventional earth moving equipment in good working condition. Difficult drilling conditions may be encountered in the bedrock materials. Excavations may encounter debris, rubble, or other obstructions in on-site fill materials. Near-vertical excavations in granular materials should be considered unstable. Recommendations for excavation stabilization are presented in the following sections of the report.
- Oversized material, debris, abandoned foundations, or the other obstructions may be encountered in the fill materials.
- Based on the materials encountered in our subsurface exploration, expansive soils are not a design consideration for the structure.
- Based on the results of our limited soil corrosivity tests during this study and Caltrans corrosion guidelines (2018), the site does not meet the definition of a corrosive environment. We do not consider corrosive soil to be a design consideration for the project
- Flood Insurance Rate Maps (FEMA, 2009) indicate that the eastern portion of the site lies within an area designated as A99 which is described as an area to be protected from 1 percent annual chance flood event by a Federal flood protection system under construction.

10 RECOMMENDATIONS

The following sections include our geotechnical recommendations for the design and construction of the proposed improvements. These recommendations are based on our evaluation of the site geotechnical conditions and our understanding of the planned construction. The proposed improvements should be designed and constructed in accordance with these recommendations, applicable codes, and appropriate construction practices.

10.1 Earthwork

Earthwork at the site is anticipated to generally consist of cuts and fills related to construction for the proposed improvements. Earthwork operations should be performed in accordance with the requirements of applicable governing agencies and the recommendations presented in the following sections of this report.

10.1.1 Site Preparation

Site preparation activities should include demolition of existing pavement and structures within the limits of work, and removal of the subsequent rubble/grindings, underlying aggregate base and subgrade to the planned subgrade elevation. Rubble and excavated materials that do not meet criteria for use as fill should be disposed of in an appropriate landfill. Existing utilities to be abandoned should be removed, crushed in place, or backfilled with grout.

Excavations resulting from removal of foundations, buried utilities, tree stumps, or obstructions should be backfilled with compacted fill in accordance with the recommendations in the following sections.

10.1.2 Excavation Stabilization

We anticipate that the project will involve excavations of depths up to about 5 feet for grading and utility installation, 15 feet for the building underground parking level, and up to about 40 feet for deep foundations or ground improvement. Excavations, including foundation and utility excavations, should be stabilized by shoring sidewalls or laying slopes back in accordance with the Excavation Rules and Regulations (29 Code of Federal Regulations [CFR], Part 1926) stipulated by the Occupational Safety and Health Administration (OSHA). The on-site soils should be considered as soil Type C in accordance with OSHA requirements. Temporary shoring design considerations and parameters are provided in Section 10.4.

Our recommendations for lateral earth pressures and allowable slope gradients are based upon the limited subsurface data provided by our exploratory borings, and reflect the influence of the environmental conditions that existed at the time of our exploration. Excavation stability, material classifications, allowable slopes, and shoring pressures should be re-evaluated and revised, as-needed, during construction. Excavations, shoring systems and the surrounding areas should be evaluated daily by a competent person for indications of possible instability or collapse.

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

Excavations made in close proximity to existing structures may undermine the foundation of those structures and/or cause soil movement related distress to the existing structures. Stabilization techniques for excavations in close proximity to existing structures will need to account for the additional loads imposed on the shoring system and appropriate setback distances for temporary slopes. The geotechnical engineer should be consulted for additional recommendations if the proposed excavations cross below a plane extending down and away from the foundation bearing surfaces of the adjacent structure at an angle of 2:1 (horizontal to vertical) from the bottom edge of the footing or if the proposed excavation is less than 18 inches from the face of the footing.

The excavation bottoms may become unstable and subject to pumping under heavy equipment loads if the excavation subgrade is exposed to water or if excavations are close to or below the groundwater (before or after dewatering.) The contractor should be prepared to stabilize the bottom of the excavations. In general, unstable bottom conditions may be mitigated by dewatering to depress groundwater levels below the bottom of the excavation, overexcavating to a suitable depth and replacing the wet material with suitable fill, compacting a layer of crushed rock fill into the subgrade, or using geogrid to stabilize additional fill. Specific recommendations for excavation stabilization will be influenced by the nature of the excavation and the conditions encountered during construction.

We anticipate that some of the bottoms of the trenches will be near or below the groundwater and will be unstable. In general, unstable bottom conditions may be mitigated by overexcavating the excavation bottom to suitable depths and replacing with gravel wrapped in geofabric. Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by the geotechnical consultant at the time of construction.

10.1.3 Observations and Removals

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

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

10.1.4 Material Recommendations

Materials used during earthwork operations should comply with the requirements listed in Table 3. On-site soils used for fill may need moisture conditioning to achieve appropriate moisture content for compaction. Materials should be evaluated by the geotechnical engineer for suitability prior to use. The contractor should notify the geotechnical consultant 72 hours prior to import of materials or use of on-site materials to permit time for sampling, testing, and evaluation of the proposed materials. On-site materials may need to be dried out before re-use as fill. The contractor should be responsible for the uniformity of import material brought to the site.

Table 3 – Recommended Material Requirements

Material and Use	Source	Requirements[1,2]
Select Fill	Import	Close-graded with 35 percent or more passing No. 4 sieve and either: Expansion Index of 50 or less, Plasticity Index of 12 or less, or less than 10 percent, by dry weight, passing No. 200 sieve
General Fill -For uses not otherwise specified	Import or On-site Borrow	Import: As per Select Fill On-Site Borrow: No additional requirements[1]
Pipe/Conduit Bedding and Pipe Zone Material -material below conduit invert to 12 inches above conduit	Import	90 to 100 percent (by mass) should pass No. 4 sieve, and 5 percent or less should pass No. 200 sieve
Trench Backfill - above bedding material	Import or On-site Borrow	As per general fill and excluding rock/lumps retained on 4-inch sieve or 2-inch sieve in top 12 inches
Aggregate Base	Import	Class II; CSS[4] Section 26-1.02
Asphalt Concrete	Import	Type A; CSS[4] Section 39-2
Controlled Low Strength Material (CLSM)	Import	CSS[4] Section 19-3.02F

Notes:

[1] In general, fill should be free of rocks or lumps in excess of 6 inches in diameter, trash, debris, roots, vegetation or other deleterious material.

[2] In general, import fill should be tested or documented to be non-corrosive[3] and free from hazardous materials in concentrations above levels of concern.

[3] Non-corrosive as defined by the Corrosion Guidelines (Caltrans, 2018).

[4] CSS is California Standard Specifications (Caltrans, 2015).

10.1.5 Subgrade Preparation

Subgrade in trenches and below footings, slabs, pavement, flatwork, or fill, should be prepared as per the recommendations in Table 4. Prepared subgrade should be maintained in a moist (but not saturated) condition by the periodic sprinkling of water prior to placement of additional overlying fill or construction of footings and slabs. Subgrade that has been permitted to dry out and loosen or develop desiccation cracking, should be scarified, moisture conditioned, and recompacted as per the requirements above.

Table 4 – Subgrade Preparation Recommendations

Subgrade Location	Preparation Recommendations
Utility Trenches	• Check for unsuitable materials as per 10.1.3. • Do not scarify. Remove or compact loose/soft material.
Below Slabs, Pavement, Flatwork, and General Fill,	• Check for unsuitable materials as per Section 10.1.3. • Scarify top 8 inches then moisture condition and compact as per Section 10.1.6. • Keep in moist condition by sprinkling water.
Below Footings	• Check for unsuitable materials as per Sections 10.1.3. • Scarify and moisture condition exposed subgrade as needed to achieve a moisture content near or above the optimum as evaluated by ASTM D1557. Compact exposed subgrade per Section 10.1.6. • Keep in moist condition by sprinkling water.

10.1.6 Fill Placement and Compaction

Fill and backfill should be compacted in horizontal lifts in conformance with the recommendations presented in Table 5. The allowable uncompacted thickness of each lift of fill depends on the type of compaction equipment utilized, but generally should not exceed 8 inches in loose thickness.

Table 5 – Fill Placement and Compaction Recommendations			
Fill Type	**Location**	**Compacted Density[1]**	**Moisture Content[2]**
Subgrade	Below pavement and areas subject to vehicular loading (top 18 inches below finish subgrade)	95 percent	+ 2 percent or above
	In locations not already specified	90 percent	+ 2 percent or above
Bedding and Pipe Zone Fill	Material below invert to 12 inches above pipe or conduit	90 percent	Near Optimum
Trench Backfill	Top 18 inches below finish subgrade for areas subject to vehicular loading	95 percent	+ 2 percent or above
	In locations not already specified	90 percent	+ 2 percent or above
Select or General Fill	Top 18 inches below finish subgrade for areas subject to vehicular loading	95 percent	+ 2 percent or above
	In locations not already specified	90 percent	+ 2 percent or above
Asphalt Concrete	Pavement section	91 to 97 percent	Not Applicable
Aggregate Base	Below areas subject to vehicular loading and hardscape	95 percent	Near Optimum

Notes:

[1] Expressed as percent relative compaction or ratio of field density to reference density (typically on a dry density basis for soil and aggregate and on a wet density basis for asphalt concrete). The reference density of soil and aggregate should be evaluated by ASTM D 1557. The reference density of asphalt concrete should be evaluated by ASTM D 2041.

[2] Target moisture content at compaction relative to the optimum as evaluated by ASTM D 1557

10.1.7 Utility Trenches

We anticipate that the project will involve excavations of depths up to about 5 feet for utility installation. Excavations for utility excavations should be stabilized by shoring sidewalls or laying slopes back in accordance with the Excavation Rules and Regulations (29 Code of Federal Regulations [CFR], Part 1926) stipulated by the Occupational Safety and Health Administration (OSHA). The on-site soils should be considered as soil Type C in accordance with OSHA requirements. Excavation stability, material classifications, allowable slopes, and shoring pressures should be re-evaluated and revised, as-needed, during construction. Excavations, shoring systems and the surrounding areas should be evaluated daily by a competent person for indications of possible instability or collapse. Dewatering pits or sumps should be used to depress the groundwater level (if encountered) below the bottom of the excavation.

Utility trenches should be backfilled with materials that conform to our recommendations in Section 10.1.4. Trench backfill, bedding, and pipe zone fill should be compacted in accordance with Section 10.1.6 of this report. Bedding and pipe zone fill should be shoveled under pipe haunches and compacted by manual or mechanical, hand-held tampers. Trench backfill should be compacted by mechanical means. Densification of trench backfill by flooding or jetting should not be permitted.

To reduce potential for moisture intrusion into the building envelope, we recommend plugging utility trenches at locations where the trench excavations cross under the building perimeter. The trench plug should be constructed of a compacted, fine-grained, cohesive soil that fills the cross-sectional area of the trench for a distance equivalent to the depth of the excavation. Alternatively, the plug may be constructed of concrete or CLSM.

10.1.8 Rainy Weather Considerations

We recommend that the construction be performed during the period between approximately April 15 and October 15 to avoid the rainy season. In the event that grading is performed during the rainy season, the plans for the project should be supplemented to include a stormwater management plan prepared in accordance with the requirements of the relevant agency having jurisdiction. The plan should include details of measures to protect the subject property and adjoining off-site properties from damage by erosion, flooding or the deposition of mud, debris, or construction-related pollutants, which may originate from the site or result from the grading operation. The protective measures should be installed by the commencement of grading, or prior to the start of the rainy season. The protective measures should be maintained in good working order unless the project drainage system is installed by that date and approval has been granted by the building official to remove the temporary devices.

In addition, construction activities performed during rainy weather may impact the stability of excavation subgrade and exposed ground. Temporary swales should be constructed to divert surface runoff away from excavations and slopes. Steep temporary slopes should be covered with plastic sheeting during significant rains. The geotechnical consultant should be consulted for recommendations to stabilize the site as-needed. A thin layer (approximately 3 inches) of lean concrete or CLSM may be poured over prepared subgrade for footings or slabs to maintain the appropriate moisture condition during erections of forms and placement of reinforcing steel.

10.2 Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 6 presents the seismic design parameters for the site in accordance with the CBC (2016) guidelines and adjusted MCE_R spectral response acceleration parameters (USGS, 2018).

Table 6 – 2016 California Building Code Seismic Design Criteria	
Seismic Design Parameter **Evaluated for 36.9782° North Latitude, 122.0273°West Longitude**	**Value**
Site Class	D[1]
Site Coefficient, Fa	1.0
Site Coefficient, Fv	1.5
Mapped Spectral Acceleration at 0.2-second Period, S_S	1.500g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.600g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.500g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.900g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.000g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.600g

[1]For structures with a fundamental period of ½ second or less.

10.3 Rock Catchment Fence

Based on our evaluation, the slope located west of the subject property is considered surficially unstable and remedial measures are needed to mitigate the impact that future surficial failures may have on the project. Since much of the slope lies outside the property limits, catchment structures along the western property are considered a feasible solution to mitigate the potential hazard. A debris catchment fence specifically designed for such applications should be considered. The catchment structure should be designed by a contractor that specializes in catchment structure design and construction, such as Geobrugg or Maccaferri. Additional design criteria can be provided, if needed, depending on the catchment structure selected. Consideration should be given to providing a building setback from the toe of slope to allow for future maintenance on the catchment fences.

10.4 Temporary Shoring

Temporary shoring can consist of a soldier-pile-and-lagging wall, soil nail wall, sheet piling, or other similar type of construction. Temporary shoring details should be designed by a structural engineer. The geotechnical consultant should be provided an opportunity to review the plans and calculations prepared by the structural engineer to check for consistency with these recommendations.

The shoring system should be designed using the lateral earth pressures presented on Figure 7 for cantilever excavations or Figure 8 for braced or tied-back excavations. The recommended design pressures are based on the assumptions that the shoring system is constructed without raising the ground surface elevation behind the shoring, that there are no surcharge loads, such as soil stockpiles and construction materials, and that no loads act above a 1:1 (horizontal to vertical) plane extending up and back from the base of the shoring system. For shoring systems subjected to the above-mentioned surcharge loads, the contractor should include the effect of these loads on the lateral earth pressures against the shoring wall. Where tiebacks are used for design, the bond zone of tiebacks should be located beyond an imaginary line that slopes upward from the base of the wall at an angle of 60 degrees from horizontal.

Settlement of the ground surface behind the shoring wall during excavation is a design concern. The amount of settlement depends heavily on the type of shoring system, the contractor's workmanship, and soil conditions. Based on our experience, we anticipate that some shoring systems may cause settlement and possible impact to structures within distances of up to approximately 50 feet from the shoring operation. We recommend that structures/improvements in the vicinity of the planned shoring installation be reviewed with regard to foundation support and tolerance to settlement. To reduce the potential for distress to adjacent structures, we recommend that the shoring system be designed to limit the ground settlement behind the shoring system to ½ inch or less. Possible causes of settlement that should be addressed include settlement during installation of the shoring, excavation for the underground parking garage construction, construction vibrations, dewatering, and removal of the support system. Vibrations from the driving of sheet piles may result in some dynamic settlement and may affect the adjacent structures. We recommend that shoring installation be evaluated carefully by the contractor prior to construction and that ground vibration and settlement monitoring be performed during construction. To reduce the potential for settlement of the retained soil, voids behind the shoring should be backfilled with compacted fill during installation and voids resulting from the removal of the shoring should be filled with CLSM or compacted fill. To reduce the potential for settlement associated with shoring removal, the benefit of leaving the shoring system buried in-place may be considered.

The contractor should retain a qualified and experienced engineer to design the shoring system. The shoring parameters presented in this report are minimum requirements, and the contractor should evaluate the adequacy of these parameters and make the required modifications for their design. We recommend that the contractor take appropriate measures to protect workers.

OSHA requirements pertaining to worker safety should be observed. The on-site soils should be considered as soil Type C in accordance with OSHA requirements. The geotechnical consultant should also observe the shoring installation.

Drilled holes for soldier pile installation may need to be stabilized by use of temporary casing or drilling slurry. Drilled holes above the planned bottom of the adjacent excavation may need to be backfilled with CLSM to stabilize the hole while the excavation proceeds and lagging is installed. Standing water should be removed from the drilled hole before placement of CLSM or lean concrete for piles embedded in lean concrete piers. Alternatively, a tremie pipe could be used to delivered the CLSM or lean concrete to the bottom of the excavation below standing water. Casing should be removed from the excavation as the concrete or CLSM is placed and the concrete or CLSM should be placed in a manner that reduces the potential for segregation of the components or impacting the side of the excavation.

Drilled holes for tieback installation should include temporary casing to prevent loss of sand and resulting settlement. Hollow-stem or flight augers should not be permitted. The holes should be grouted prior to removal of the casing and additional grout added as casing is removed.

10.5 Construction Dewatering

Groundwater was measured at a depth of about 16 feet in both CPT sounding locations. Based on a review of available subsurface data, groundwater is generally about 10 to 16 feet below the ground surface and flows parallel to the contour of the relatively impermeable bedrock. Water intrusion into the excavations may occur as a result of groundwater seepage or surface runoff. The contractor should be prepared to take appropriate dewatering measures in the event that water intrudes into the excavations. Sump pits, trenches, or similar measures should be used to depress the water level below the bottom of the excavation. Considerations for construction dewatering should include anticipated drawdown, volume of pumping, potential for settlement, and groundwater discharge. Drawing down of the water level within the excavation may affect the water level outside of the excavation. This will result in an increase in effective stresses and may induce settlement of the soils underlying adjacent structures. Additional measures which the contractor could implement to reduce groundwater inflow and/or resulting settlement include chemical grouting, shotcreting side walls, utilizing slurry walls, and using groundwater recharge wells. Disposal of groundwater should be performed in accordance with the guidelines of the Regional Water Quality Control Board

10.6 Foundations

The following foundation design parameters and recommendations are provided based on our findings and geotechnical analysis. The foundation design parameters are not intended to preclude differential movement of foundations. Minor cracking (considered tolerable) of foundations may occur. Foundations should be designed in accordance with structural considerations and our geotechnical recommendations. In addition, requirements of the governing jurisdictions, practices of the Structural Engineers Association of California, and applicable building codes should be considered in the design of the structures.

Due to the potential for dynamic settlement, ground subsidence, and differential settlement, we anticipate foundations will consist of one of, or a combination of, the following: shallow footings in bedrock, a mat slab with remedial grading, shallow footings or mat slab over ground improvement, or deep foundations. Building loads were not provided; therefore, our recommendations included herein should be considered preliminary for use in selecting the preferred alternative. Further evaluation may be required once the preferred foundation type is selected.

Recommendations are also provided for footings for other lightly-loaded ancillary improvements.

10.6.1 Shallow Foundations in Bedrock

Foundations may be supported on shallow footings that derive support in undisturbed bedrock materials. Based on the depths to bedrock encountered during our evaluation, we anticipate that shallow foundations in bedrock will be feasible for the western portion of the site. Where the depth to bedrock is deeper than the planned excavations, ground improvement should be performed beneath shallow foundations in the alluvium materials per Section 10.6.4.

Footings 18- to 36-inches wide on level ground embedded 24 inches, or more, below the adjacent grade and bearing on prepared bedrock subgrade may be designed for an allowable bearing capacity of 4,000 pounds per square foot. The allowable bearing capacity is the net allowable bearing capacity and includes a factor of safety of 3 or more. The allowable bearing capacity may be increased by one-third when considering wind or seismic load combinations.

Mat slabs should be designed based on the anticipated loading and intended usage using an allowable bearing capacity of 4,000 psf for a foundation width of 10 feet or more. This allowable bearing capacity includes a factor of safety of more than 3 and may be increased by one-third when considering wind or seismic loading combinations. For preliminary foundation analysis, the deflection of an approximately 60 feet by 150 feet mat due to applied loads may be modeled using a modulus of subgrade reaction of 50 pounds per cubic inch. The subgrade modulus should be revised once the final building configuration and loading are known.

Preliminary estimates indicate structures supported on footings or mat slabs consistent with these recommendations should be designed for a total static settlement of ½ inch with a differential of ¼ inch over a lateral span of 20 feet for sustained loads of up to 300 kips for columns and 7 kips per foot for walls in bedrock materials. The actual settlement across the building will be dependent on the foundation system selected and loading conditions. Additional settlement analysis should be conducted once the final building configuration and loading are known. Dynamic settlement in bedrock materials due to seismic ground shaking is anticipated to be negligible.

A friction coefficient of 0.35 may be assumed for evaluating frictional resistance to lateral loads. A lateral bearing pressure of 300 psf per foot of depth up to 4,500 psf may be used to evaluate the resistance of footings to lateral loads for level ground conditions. The lateral bearing pressure should be neglected to a depth of 1 foot where the ground adjacent to the foundation is not covered by a slab or pavement. The lateral resistance can be taken as the sum of the frictional resistance and passive resistance, provided the passive resistance does not exceed one-half of the total allowable resistance. The friction coefficient and passive lateral bearing pressure should be considered ultimate values. The lateral bearing pressure may be increased by one-third when considering loads of short duration such as wind or seismic forces.

The mat slab should be reinforced with deformed steel bars that have a nominal diameter of ½ inch or more. The mat slab and slab reinforcement should be designed and detailed by the structural engineer. Masonry briquettes or plastic chairs should be used to aid in the correct placement of slab reinforcement. Recommendations for concrete and concrete cover over reinforcing steel are presented in Section 10.10.

10.6.2 Mat Foundation

A mat foundation is a suitable foundation system, provided remedial grading is performed beneath the foundation.

Remedial grading should consists of removal and replacement of material to a depth of 5 feet below the bottom of the mat foundation, or to a depth of 15 feet below the existing ground surface, whichever is deeper. Additional overexcavation of loose, soft, and/or wet areas may be appropriate, depending on our observations during construction. Prior to placing the new engineered compacted fill, the exposed relatively dense or stiff subgrade material should be scarified, moisture-conditioned, and recompacted to a depth of approximately 8 inches. The new fill should consist of material consistent with Select Fill per Section 10.1.4, on subgrade prepared in accordance with Section 10.1.5, and compacted in accordance with Section 10.1.6.

Mat slabs should be designed based on the anticipated loading and intended usage using an allowable bearing capacity of 3,000 psf for a foundation width of 10 feet or more. This allowable bearing capacity includes a factor of safety of more than 3 and may be increased by one-third when considering wind or seismic loading combinations. For preliminary foundation analysis, the deflection of an approximately 60 feet by 150 feet mat due to applied loads may be modeled using a modulus of subgrade reaction of 17 pounds per cubic inch. The subgrade modulus should be revised once the final building configuration and loading are known.

Preliminary estimates indicate structures supported on mat slabs consistent with these recommendations should be designed for a total static settlement of 1 inch with a differential of ½ inch over a lateral span of 20 feet for sustained loads. The actual settlement across the building will be dependent on the foundation system selected and loading conditions. Additional settlement analysis should be conducted once the final building configuration and loading are known. Dynamic settlement following the considered seismic event after remedial grading is performed is anticipated to be up to approximately 1¼ inch of total settlement with a differential settlement of about ⅔ inch over a horizontal distance of 30 feet.

A friction coefficient of 0.35 may be assumed for evaluating frictional resistance to lateral loads. A lateral bearing pressure of 300 psf per foot of depth up to 4,500 psf may be used to evaluate the resistance of footings to lateral loads for level ground conditions. The lateral bearing pressure should be neglected to a depth of 1 foot where the ground adjacent to the foundation is not covered by a slab or pavement. The lateral resistance can be taken as the sum of the frictional resistance and passive resistance, provided the passive resistance does not exceed one-half of the total allowable resistance. The friction coefficient and passive lateral bearing pressure should be considered ultimate values. The lateral bearing pressure may be increased by one-third when considering loads of short duration such as wind or seismic forces.

The mat slab should be reinforced with deformed steel bars that have a nominal diameter of ½ inch or more. The mat slab and slab reinforcement should be designed and detailed by the structural engineer. Masonry briquettes or plastic chairs should be used to aid in the correct placement of slab reinforcement. Recommendations for concrete and concrete cover over reinforcing steel are presented in Section 10.10.

10.6.3 Deep Foundations

Based on the depth to bedrock on the eastern portion of the site, we anticipate deep foundations will be needed to derive support in bedrock, provided ground improvement is not performed. Due to the nearby neighboring structures, we anticipate that deep foundation construction methods which generate high vibrations and noise, such as driven piles, will not be acceptable. We anticipate that drilled displacement auger cast pile foundations would be the most suitable deep foundation system, given the subsurface materials and site constraints.

Auger cast piles are cast-in-place foundations that are generally constructed by drilling a shaft in one pass with a hollow-stem auger, injecting cement grout through the hollow stem to fill the shaft as the auger is withdrawn from the excavation, then lowering a cage of reinforcing steel into the grout-filled shaft. Drilled displacement (DD) piles are constructed utilizing an auger with a shaft diameter that increases with distance above the cutting head. The increasing shaft diameter displaces the excavated soil laterally as the auger is advanced to increase the density of the soil around the excavation and reduce the quantity of drill cuttings produced. DD piles that utilize an auger with a shaft diameter that increases to meet the flighting diameter, can be considered "full displacement" piles. DD piles may be constructed as full or partial displacement piles with continuous or limited flighting. Augers with limited flighting generally include a section with reversed flights above the displacement body to gather and displace sloughed soil as the auger is rotated out of the hole.

A pre-production indicator pile program should be performed to evaluate achievable bearing depths and resistance to axial loads. The indicator pile program should consist of constructing six or more piles with the proposed equipment to refusal or a target bearing depth at locations distributed around the building footprint. High strain dynamic testing should be performed on the indicator piles in general conformance with ASTM D4945 to evaluate resistance to axial loads. The indicator piles should be instrumented to evaluate tip and shaft resistance. The proposed locations for the indicator piles and the results of the dynamic testing should be reviewed by the geotechnical engineer. The design allowable axial resistance should not exceed 50 percent of the nominal resistance achieved during the testing for downward loading conditions or 33 percent of the nominal resistance achieved due to side friction for upward loading conditions. The design allowable axial resistance may be increased by one third for seismic or wind load combinations.

For preliminary design, an ultimate nominal axial resistance of 200 kips for downward loading and 100 kips for upward loading may be assumed for 16-inch diameter auger cast piles bearing in bedrock. The total estimated pile cap settlement due to the applied loads, and downdrag from the dynamic settlement is approximately ½ inch with a differential settlement of approximately ¼ inch over a lateral distance of about 30 feet. To mitigate reduction in axial resistance due to pile group effects, the center-to-center spacing between adjacent piles should not be less than three pile diameters.

The parameters listed in Table 7 may be used to evaluate the lateral load resistance of pile foundations for non-seismic conditions. The parameters listed in Table 8 may be used to evaluate the lateral load resistance of the pile foundations for seismic conditions with consideration for select soil layers at residual strength due to liquefaction.

Table 7 – Soil Parameters for Lateral Pile Resistance under Static Conditions

Layer Depth Top-Bottom (feet)[1]	γ' (lb/ft^3)	ϕ (degrees)	Su (kip/ft^2)	K (lb/in^3)	e_{50} (%)
Alluvium 0-10	115	33	--	50	--
Alluvium 10-18	52	33	--	50	--
Alluvium 18-40	52	37	--	135	--
Bedrock (all depths)	110	33	1.00	500	0.50

[1]Note – depths are relative to the existing ground surface.

Table 8 – Soil Parameters for Lateral Pile Resistance under Seismic Conditions

Layer Depth Top-Bottom (feet)[1]	γ' (lb/ft^3)	ϕ (degrees)	Su (kip/ft^2)	K (lb/in^3)	e_{50} (%)
Alluvium 0-10	115	33	--	50	--
Alluvium 10-18	52	--	0.05	50	2.00
Alluvium 18-40	52	37	--	135	--
Bedrock (all depths)	110	33	1.00	500	0.50

[1]Note – depths are relative to the existing ground surface.

The potential for a reduction in the lateral resistance of piles due to the influence of adjacent piles should be considered in design. Piles in a row perpendicular to the direction of lateral loading should be spaced (center to center) at a distance equivalent to three pile diameters (or more) to avoid a reduction in the lateral load resistance due to group effects. A reduction in the lateral resistance due to group effects should be considered for piles in a column parallel to the direction of loading where the center-to-center spacing between adjacent piles in the column is less than eight pile diameters. The reduction in lateral resistance due to group effects for piles in a column parallel to the direction of loading is influenced by the number of piles in the column and the spacing between piles. The efficiency or available lateral resistance per pile are presented in Table 9 for piles in a column parallel to the direction of loading at various spacing. The designer may interpolate between the values in the table for an intermediate spacing or number of piles.

Table 9 – Group Efficiency for Lateral Loading of Pile Groups			
Piles in Column [1]	3B Pile Spacing [2]	6B Pile Spacing [2]	8B Pile Spacing [2]
2	60 percent	93 percent	100 percent
3	50 percent	85 percent	100 percent
4	45 percent	81 percent	100 percent
6	40 percent	78 percent	100 percent
10	36 percent	75 percent	100 percent
15	34 percent	73 percent	100 percent
20	33 percent	72 percent	100 percent

1 Number of piles in column parallel to the direction of the anticipated lateral load.
2 Center to center pile spacing in direction of the anticipated load where 'B' is the pile diameter.

A lateral earth pressure on embedded grade beams or pile caps equivalent to 300 pounds per square foot (psf) per foot of depth, up to 3,000 psf, may be considered when evaluating the resistance to lateral loads. The lateral earth pressure within one foot of finish grade should be neglected where the ground adjacent to the foundation is not covered by pavement or a concrete slab. The lateral earth pressure may be increased by one-third for wind or seismic loading conditions.

The lateral deflection needed to develop the recommended earth pressure for resistance to lateral loading on pile caps and grade beams is equivalent to 0.7 percent of the embedment depth for the pile cap or grade beam. This lateral earth pressure should be reduced proportionally where the design lateral deflection, consistent with the assumed head deflection of the pile foundation, is less than 0.7 percent of the embedment depth for the cap or beam. No reduction is needed where the embedment depths are less than 4.5 feet or 12 feet for ⅜-inch or 1-inch of design lateral deflection, respectively.

Over-rotation of the continuous flight auger during drilling can mine soil adjacent to the excavation, resulting in future settlement near the completed pile. Interruptions or variations in the rate of auger withdrawal or grout injection can incorporate defects into the pile. To address these concerns, key parameters should be monitored during the drilling and grouting operations. The contractor should furnish equipment to automatically measure auger rotation, auger depth, penetration rate, torque delivered to the auger, crowd force, lifting rate, volume of grout placed, and pressure of the grout near the auger tip. These parameters should be automatically recorded as a function of auger depth at vertical intervals of 2 feet or less and submitted to the geotechnical engineer for review. To reduce the potential for soil mining due to over-rotation, the auger penetration rate should generally exceed the auger pitch in 1½ to 2 rotations for cohesionless soil and in 2 to 3 rotations for clay. The potential for soil mining and an appropriate penetration rate for the site conditions can be evaluated by pre-production indicator piles. The target penetration rate should be selected by the foundation contractor based on the proposed equipment and experience on sites with similar ground conditions, or based on a pre-production indicator pile program. To reduce the potential for defects in the pile, the applied grouting pressure and the withdrawal rate should be maintained so that the grout pressure at the discharge point exceeds the overburden pressure. The volume of grout placed should exceed the theoretical volume of the pile, typically by about 15 to 20 percent. The contractor should select a target grout volume factor based on the proposed equipment and experience on sites with similar ground conditions, or based on a pre-production indicator pile program. The observed grout volume factor should be within 7½ percent of the target.

Auger cast piles should be installed within 3 inches of the planned location and within 2 percent of plumb. Where the lateral distance between adjacent piles is less than 6 pile diameters, the second pile should not be drilled until the grout in the first pile has set. Ninyo & Moore should observe the drilling and grouting of the auger cast piles.

10.6.4 Ground Improvement With Shallow Foundations

Ground improvement can be performed to reduce the dynamic and static settlement and increase the bearing capacity of the subsurface soils. Based on the nearby neighboring structures, ground improvement methods which generate high vibrations should not be used. Detailed design of the soil improvement, including construction procedures, equipment, and the size and spacing of the improvement should be prepared by a specialty contractor to meet the project objectives. In general, we anticipate that ground improvement methods could include compaction grouting, vibro stone columns, rammed aggregate piers, or drilled displacement columns. Based on our liquefaction and dynamic settlement analysis, we anticipate the ground improvement will extend to the depth of bedrock, which is anticipated to be up to about 40 feet below the ground surface. The ground improvement should be designed to reduce the calculated dynamic settlement at the site to ½ inch, or less, and the total static settlement to ½ inch or less. In-situ verification testing of the improved ground should be performed with Cone Penetration Test soundings to confirm the design assumptions were achieved. We recommend a minimum of four CPT soundings should be performed as part of verification testing.

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

Vibro stone columns construction involves the insertion of crushed stone in a grid pattern with a vibratory probe. The strength of the soil mass is increased due to the reinforcement of crushed stone and densification of surrounding soils. In addition, the potential for liquefaction of the subsurface soils is reduced with the improved drainage provided by these stone columns. We anticipate the allowable design bearing pressures of a Vibro stone columns system will be on the order of 4,000 psf.

Rammed aggregate piers consist of compacted gravel columns that extend through soft or liquefiable soil layers. Like stone columns, the installation of aggregate piers provides for an increase in soil strength as a result of the compacted gravel columns and increased densification of surrounding soils. In addition, the potential for liquefaction is reduced by the improved drainage of the gravel columns. The difference between aggregate piers and stone columns is in their installation. Aggregate piers are installed by pushing a probe down to the desired depth and then ramming the hole with 12-inch-thick lifts of mechanically compacted gravel. Since the added compaction increases the shear strength between the soils and aggregate piers, a higher bearing capacity can be realized for design of shallow foundations. We anticipate the allowable design bearing pressures of a RAP system will be on the order of 6,000 psf.

Drilled displacement columns consist of a grid of a grout columns installed beneath the building footprint. They are constructed with similar methods as drilled displacement auger-cast piles, but typically do not include steel reinforcement and are not structurally connected to the building foundation. An aggregate cushion is typically constructed between the top of the grout columns and the foundation techniques. We anticipate the allowable design bearing pressures of a drilled displacement columns system will be on the order of 4,000 psf.

10.6.5 Floor Slabs

Slab-on-grade floors for pile-supported buildings will settle differentially relative to the pile-supported walls and columns following a significant earthquake due to dynamic settlement. We anticipate that the differential dynamic settlement, following the design earthquake, between the slab-on-grade floor and the pile supported columns may be about 2¾ inches and liquefaction and sand ejecta may occur beneath the underground parking garage slab. Consequently, we recommend the floor slab for the underground parking garage be designed as a structural slab, where the support provided by the subgrade is neglected, to reduce the potential for differential settlement between the floor slab and the pile-supported walls and columns. Where ground improvement is performed to reduce liquefaction and dynamic settlement effects, floor slabs may be designed as slabs-on-grade.

Floor slabs should be designed by the project structural engineer based on the anticipated loading and support conditions. Slabs should be reinforced with deformed steel bars with a nominal diameter of ⅜-inch or more. Masonry briquettes or plastic chairs should be used to maintain the position of the reinforcement in the upper half of the slab during concrete placement. Refer to Section 10.10 for the recommended concrete cover over reinforcing steel. A vapor retarder is recommended in areas where moisture-sensitive floor coverings or conditioned environments are anticipated. See Section 10.11 for vapor retarding system recommendations. Slabs exposed to vehicular traffic should be underlain by crushed rock with a vapor retarding membrane or aggregate base. Joints consistent with ACI guidelines (ACI, 2015) should be constructed at periodic intervals to reduce the potential for random cracking of the slab.

10.6.6 Minor Structures

Minor structures may be supported on shallow footings bearing on alluvium or bedrock. Footings 12- to 36-inches wide on level ground embedded 18 inches below the adjacent grade and bearing on prepared subgrade may be designed for an allowable bearing capacity of 2,500 pounds per square foot. The allowable bearing capacity may be increased by one-third when considering wind or seismic load combinations.

The lateral load resistance of shallow footings may be evaluated using a coefficient of friction of 0.30 and a passive equivalent fluid pressure of 300 pcf. One foot of embedment depth should be neglected when evaluating the passive lateral earth pressure where the ground surface is not covered by a slab or pavement.

Minor structures such as light poles and fences may be supported on drilled pier foundations. Drilled piers for minor structures, up to 5 feet deep, may be designed for an allowable skin friction of 200 pounds per square foot to evaluate resistance to axial loads with a one-third increase for wind or seismic loading conditions. An allowable lateral bearing pressure of 150 pounds per square foot (psf) per foot depth up to 2,250 psf may be used to evaluate resistance to lateral loads and overturning moments. The allowable lateral bearing pressure may be increased by a factor of two for structures that can accommodate ½ inch of lateral deflection. Drilled pier excavations should be cleaned of loose material prior to pouring concrete. Drilled pier excavations that encounter groundwater or cohesionless soil may be unstable and may need to be stabilized by temporary casing or use of drilling mud. Standing water should be removed from the pier excavation or the concrete should be delivered to the bottom of the excavation, below the water surface, by tremie pipe. Casing should be removed from the excavation as the concrete is placed. Concrete should be placed in the piers in a manner that reduces the potential for segregation of the components.

10.7 Below-Grade Walls

Below grade walls, such as for the underground parking garage, that are restrained by framing, floor diaphragms, or shear walls should be designed to resist at-rest earth pressures. Restrained walls subjected to lateral earth pressures should be designed using the parameters presented on Figure 9. Walls with inclined backfill should be designed for an additional equivalent fluid earth pressure of 1 pcf per one degrees of backfill inclination. Below grade walls should be supported on the same foundation system as the main building structure.

To reduce potential for moisture intrusion into the underground parking garage, a subdrain, as described in Section 10.11, should be constructed behind the wall and connected to a sump. Geocomposite drain panels (Miradrain 6000XL, or similar) placed against the back of the wall may be used to supplement a smaller subdrain located near the base of the wall. Measures to reduce the rate of moisture or vapor intrusion through the wall may be advisable for walls where the discoloration resulting from moisture intrusion would be undesirable. Such measures might include use of concrete with a low water-to-cementitious-materials ratio, and/or the placement of an asphalt emulsion or 15-mil thick plastic membrane to the back surface of the wall

10.8 Uplift Considerations

For structures that will extend below the water table, uplift forces will need to be considered. Hydrostatic uplift forces should be evaluated for a potential shallow groundwater condition of approximately 10 feet below the ground surface. The resistance to uplift may then be taken as the sum of the weight of the structure and the uplift resistance of the sidewalls.

We recommend that the structure be designed to resist hydrostatic uplift. Alternatives for resisting the anticipated uplift pressures include constructing a thick concrete mat foundation or extending the foundation a selected distance outside the exterior walls of the structure (flanges). The resistance to uplift may then be taken as the sum of the weight of the structure and the weight of the wedge of soil within the zone of influence (Figure 10). Alternatively, tie-down anchors can be installed to resist hydrostatic uplift forces.

10.9 Flatwork

Concrete walkways and other exterior flatwork not subject to vehicular loading should be 4 inches thick (or more) over 6 inches of aggregate base. Appropriate jointing of concrete flatwork can encourage cracks to form at joints, reducing the potential for crack development between joints. Joints should be laid out in a square pattern at consistent intervals. Contraction and construction should be detailed and constructed in accordance with the guidelines of ACI Committee 302 (ACI, 2016). The lateral spacing between contraction joints should be 8 feet or less for a 4-inch thick slab.

Distributed reinforcing steel may be utilized to reduce the potential for differential slab movement, should cracking occur between joints. The distributed reinforcing steel should be terminated about 6 inches from contraction joints and should consist of No. 3 deformed bars at 18 inches on center, both ways. Slabs reinforced with distributed steel should be 5 inches thick (or more). To reduce the potential for differential slab movement across joints, the distributed steel may be extended through the joints. This improvement will be balanced by a reduction in the functionality of the contraction joint to encourage crack formation at joints. Masonry briquettes or plastic chairs should be used to maintain the position of the reinforcement in the upper half of the slab with 1½ inches of cover over the steel.

10.10 Concrete Placement

Laboratory testing indicated that the concentration of sulfate and corresponding potential for sulfate attack on concrete is negligible for the soil tested. However, due to the variability in the on-site soil and the potential future use of reclaimed water at the site, we recommend that Type II/V or Type V cement be used for concrete structures in contact with soil. In addition, we recommend a water-to-cement ratio of no more than 0.45. A 3-inch thick, or thicker, concrete cover should be maintained over reinforcing steel where concrete is in contact with soil in accordance with recommendations of ACI Committee 318 (ACI, 2014).

10.11 Moisture Vapor Retarder

The migration of moisture through slabs underlying enclosed spaces or overlain by moisture sensitive floor coverings should be discouraged by providing a moisture vapor retarding system between the subgrade soil and the bottom of slabs. We recommend that the moisture vapor retarding system consist of a 4-inch-thick capillary break, overlain by a 15-mil-thick plastic membrane. The capillary break should be constructed of clean, compacted, open-graded crushed rock or angular gravel of ¾-inch nominal size. To reduce the potential for slab curling and cracking, an appropriate concrete mix with low shrinkage characteristics and a low water-to-cementitious-materials ratio should be specified. In addition, the concrete should be delivered and placed in accordance with ASTM C94 with attention to concrete temperature and elapsed time from batching to placement, and the slab should be cured in accordance with the ACI Manual of Concrete Practice (ACI, 2016), as appropriate. The plastic membrane should conform to the requirements in the latest version of ASTM Standard E 1745 for a Class A membrane. The bottom of the moisture barrier system should be higher in elevation than the exterior grade, if possible. Positive drainage should be established and maintained adjacent to foundations and flatwork.

Where the exterior grade is at a higher elevation than the moisture vapor retarding system (including the capillary break layer), consideration should be given to constructing a subdrain around the foundation perimeter. The subdrain should consist of ¾-inch crushed rock wrapped in filter fabric (Mirafi 140N, or equivalent). The subdrain should be capped by a pavement or 12 inches of native soil and drained by a perforated pipe (Schedule 40 polyvinyl chloride pipe, or similar). The pipe should be sloped at 1 percent or more to discharge at an appropriate outlet away from the foundation. The pipe should be located below the bottom elevation of the moisture vapor retarding system but above a plane extending down and away from the bottom edge of the foundation at a 2:1 (horizontal to vertical) gradient.

10.12 Drainage

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

Landscaping adjacent to foundations should include vegetation with low-water demands and irrigation should limited to that which is needed to sustain the plants. Trees should be restricted from the areas adjacent to foundations a distance equivalent to the canopy radius of the mature tree. Bioretention areas should not be located within a distance of 20 feet from structure foundations.

Care should be taken by the contractor during grading to preserve any berms, drainage terraces, interceptor swales or other drainage devices on or adjacent to the project area. Drainage patterns established at the time of grading should be maintained for the life of the project. The property owner and maintenance personnel should be made aware that altering drainage patterns might be detrimental to wall performance.

11 CONSTRUCTION MONITORING AND INSTRUMENTATION

An instrumentation program should be implemented to evaluate design assumptions, and monitor vibrations at adjacent structures, groundwater levels, deformations of the excavations, and ground surface settlement. The monitoring program should include seismographs, groundwater observation wells, and an array of surface control points. The data obtained should be distributed to appropriate parties during the course of construction.

11.1 Documentation of Existing Conditions

We recommend a pre-construction existing conditions survey be performed on structures within approximately 50 feet of proposed construction activities. The pre-construction survey should consist of photographic documentation of the exterior portions of the buildings and hardscape features, including distress features, such as cracks and/or separations that may be present. Consideration may be given to videotaping the survey.

11.2 Construction Vibrations

Human experience has shown that vibrations at very low levels can be perceived and judged as being much higher than they actually are. Hendron and Oriard (1972) stated that transient vibrations from construction activities, such as pile driving, are noticeable at peak particle velocities as low as 0.02 to 0.06 inches per second (ips). At peak particle velocities as low as 0.2 to 0.4 ips, the vibrations are disturbing and may result in complaints and damage claims. However, these vibration levels are below the peak particle velocity threshold considered to cause cosmetic damage to commercial/residential construction.

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

We recommend that vibration caused by construction activities be monitored in terms of peak particle velocity during construction. To monitor the peak particle velocity, seismographs could be positioned near the adjacent structures and monitored at selected intervals during construction to check that the peak particle velocity does not exceed 0.2 inches per second. If peak particle velocities exceed this threshold, construction activity should stop and construction procedures should be re-evaluated to reduce the potential for excessive vibration. Additional seismographs should be located at various structures farther from the construction activities to monitor vibrations as a function of distance from the site. After review of the data obtained, the number of seismographs may be reduced at the discretion of the client and the geotechnical consultant.

11.3 Groundwater Monitoring

As previously noted, settlement of the ground surface and adjacent structures may also be caused by drawdown of the water table. We recommend, therefore, that the contractor monitor water levels outside of the excavation so that the groundwater will not be lowered more than approximately 3 feet below the bottom of the excavation. To monitor the groundwater levels outside of the excavations, we recommend that the existing groundwater monitoring wells in the site vicinity be used and additional groundwater monitoring wells be installed as needed. The monitoring wells should be installed at locations that will likely be accessible during construction. The groundwater levels should be monitored daily or several times a day during dewatering as appropriate.

11.4 Ground Survey Monitoring

We recommend that arrays of ground survey targets be installed around the proposed excavations and on the slope to the west of the project. The survey targets should be installed near the excavations at approximately 20-foot spacings. We recommend that the contractor be responsible for maintaining total settlement or horizontal displacement at any survey point to less than ½ inch. If the settlements reach this limit, we recommend that a further review of construction methodologies be performed and appropriate changes be made.

Consideration should be given to placing survey monitoring points on nearby structures to monitor the performance of the structures. In this way, a record of the performance of the structure will be maintained and available. This information, in conjunction with pre-construction surveys, is helpful in reducing potential claims and expediting and limiting settlement of legitimate claims.

11.5 Inclinometer Monitoring

We recommend that inclinometers be installed behind the temporary excavations for the underground parking garage. The inclinometers should extend to depths of 15 feet, or more, below the bottom of the proposed excavation. The inclinometer casings should be Durham Geo Slope Indicator 2.75-inch QC, or approved equivalent, installed in accordance with the manufacturers' recommendations. Baseline monitoring should be performed after the inclinometer grout has set and prior to excavations being performed. Monitoring should be performed before and after each excavation sequence and at weekly intervals.

12 CONSTRUCTION OBSERVATION

The recommendations provided in this report are based on our understanding of the proposed project and on our evaluation of the data collected based on subsurface conditions disclosed by widely spaced subsurface exploration. It is imperative that the interpolated subsurface conditions be checked by a qualified person during construction. Observation of foundation excavations and observation and testing of compacted fill and backfill should be performed by a qualified person during construction. In addition, the project plans and specifications should be reviewed to check for conformance with the recommendations of this report prior to construction. It should be noted that, upon review of these documents, some recommendations presented in this report might be revised or modified.

During construction we recommend that the duties of the geotechnical consultant include, but not be limited to:

- Observing preparation and compaction of subgrade.
- Observing excavation bottoms and the placement and compaction of fill.
- Evaluating imported materials prior to their use as fill, if used.
- Performing field density tests to evaluate fill and subgrade compaction.
- Observing foundation excavations for bearing materials and cleaning prior to placement of reinforcing steel or concrete.

13 LIMITATIONS

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

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

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

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

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

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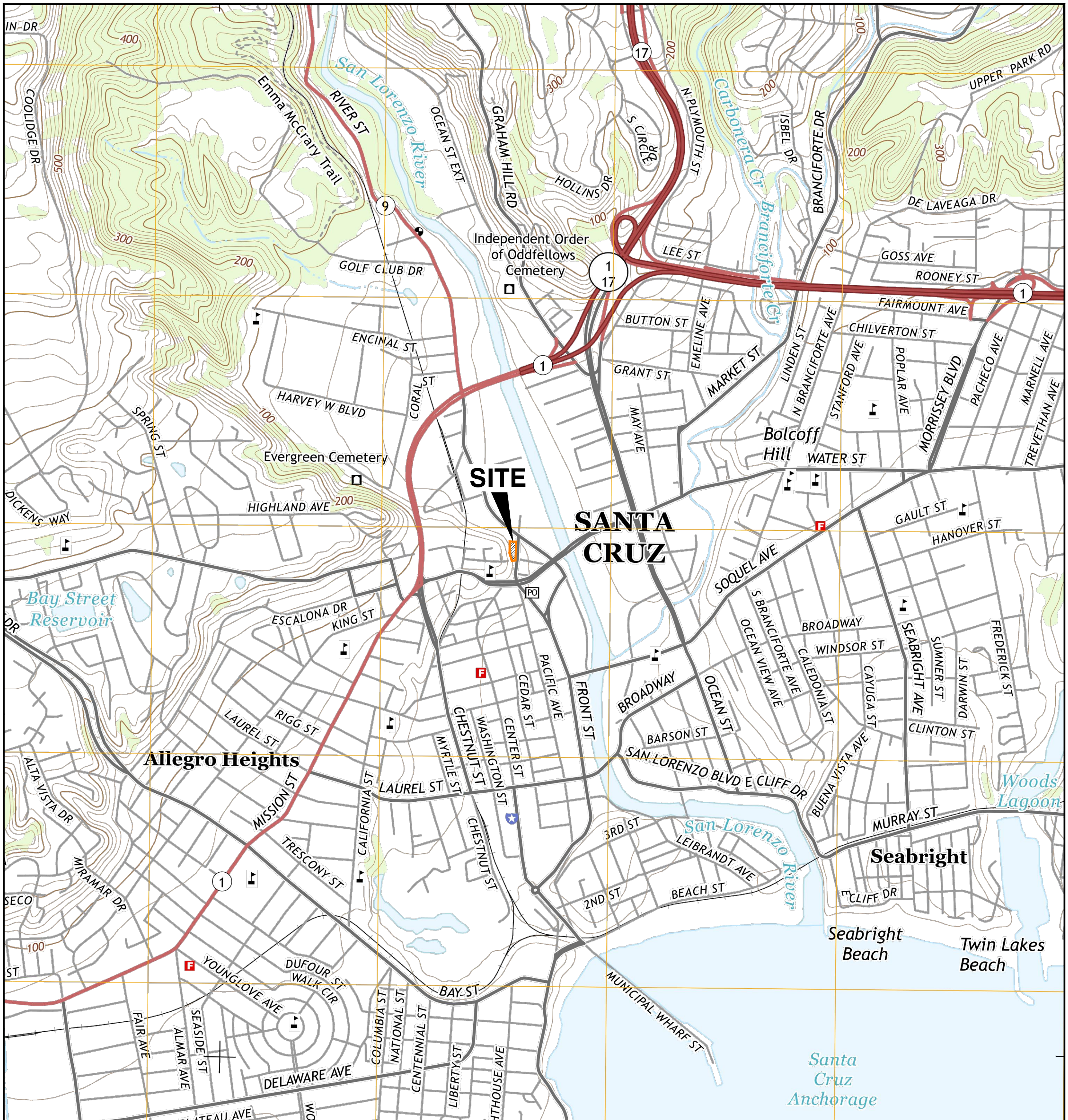
FIGURES



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE | REFERENCE: USGS, 2015

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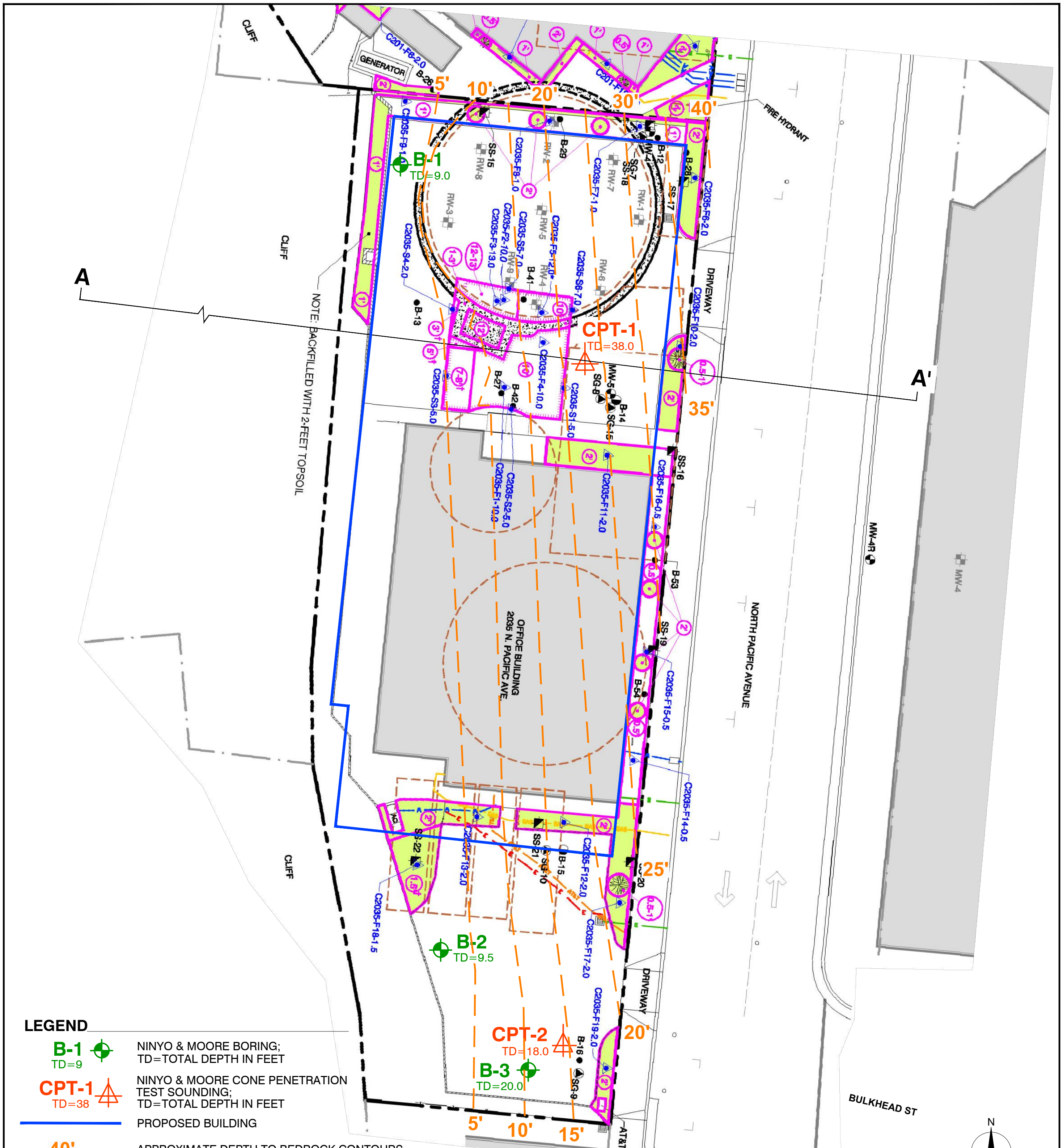
FIGURE 1

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SITE LOCATION

2035 NORTH PACIFIC AVENUE
SANTA CRUZ, CALIFORNIA
403215001 | 5/18



LEGEND

B-1 TD=9	NINYO & MOORE BORING; TD=TOTAL DEPTH IN FEET
CPT-1 TD=38	NINYO & MOORE CONE PENETRATION TEST SOUNDING; TD=TOTAL DEPTH IN FEET
(blue line)	PROPOSED BUILDING
—40'— —	APPROXIMATE DEPTH TO BEDROCK CONTOURS

FEET 0 30 60

NOTE: FOR INFORMATION ON BORINGS BY OTHERS, REMEDIATION WORK, AND BURIED STRUCTURES, SEE REFERENCED TERRA PACIFIC GROUP REPORT (2016)

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE | REFERENCE: TERRA PACIFIC GROUP, 2016.

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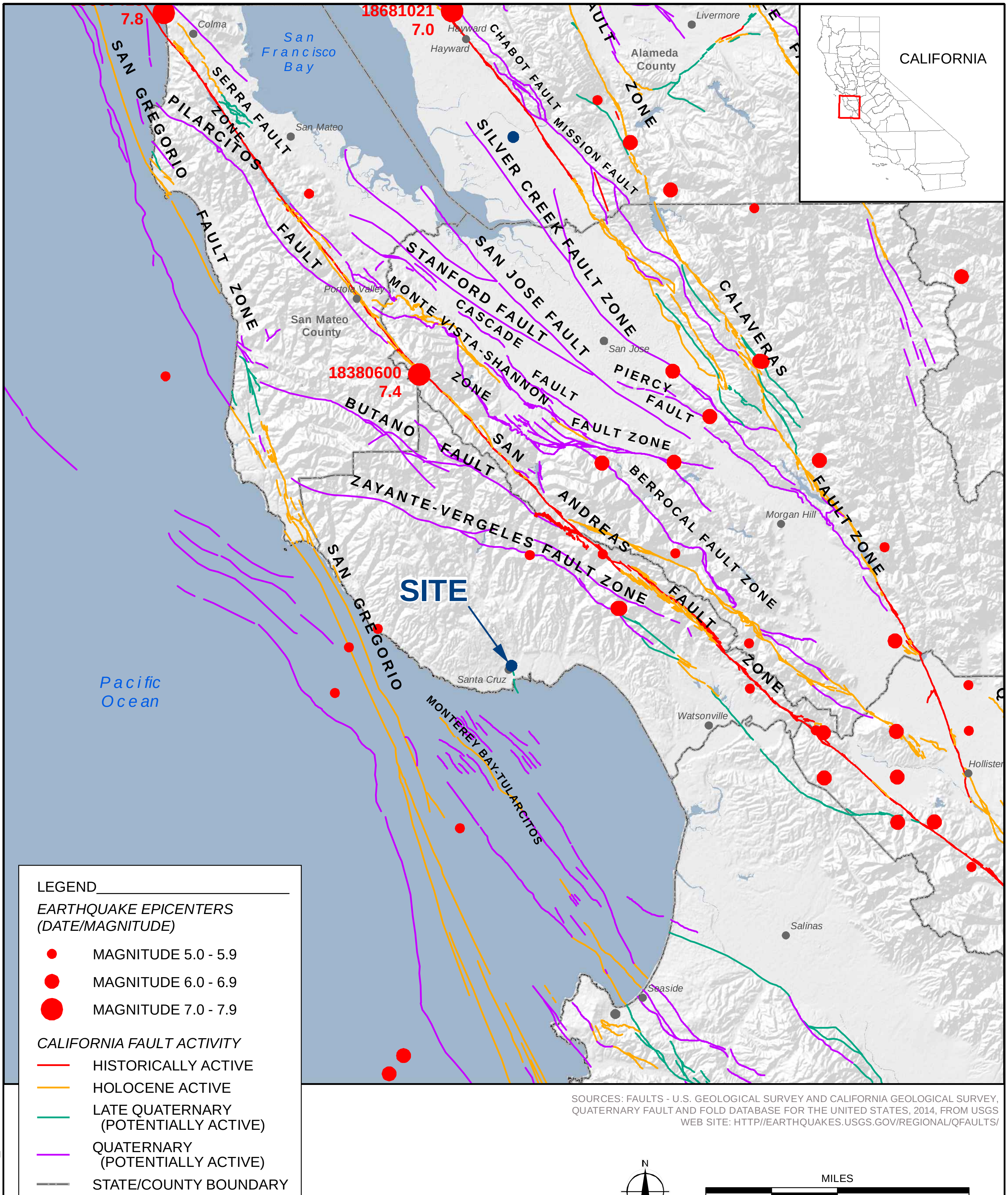
FIGURE 2

EXPLORATION LOCATIONS

2035 NORTH PACIFIC AVENUE
SANTA CRUZ, CALIFORNIA

403215001 | 5/18

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



SOURCES: FAULTS - U.S. GEOLOGICAL SURVEY AND CALIFORNIA GEOLOGICAL SURVEY, QUATERNARY FAULT AND FOLD DATABASE FOR THE UNITED STATES, 2014, FROM USGS WEB SITE: HTTP//EARTHQUAKES.USGS.GOV/REGIONAL/QFAULTS/

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

FIGURE 3

FAULT LOCATIONS AND EARTHQUAKE EPICENTERS

2035 NORTH PACIFIC AVENUE
SANTA CRUZ, CALIFORNIA

403215001 | 5/18

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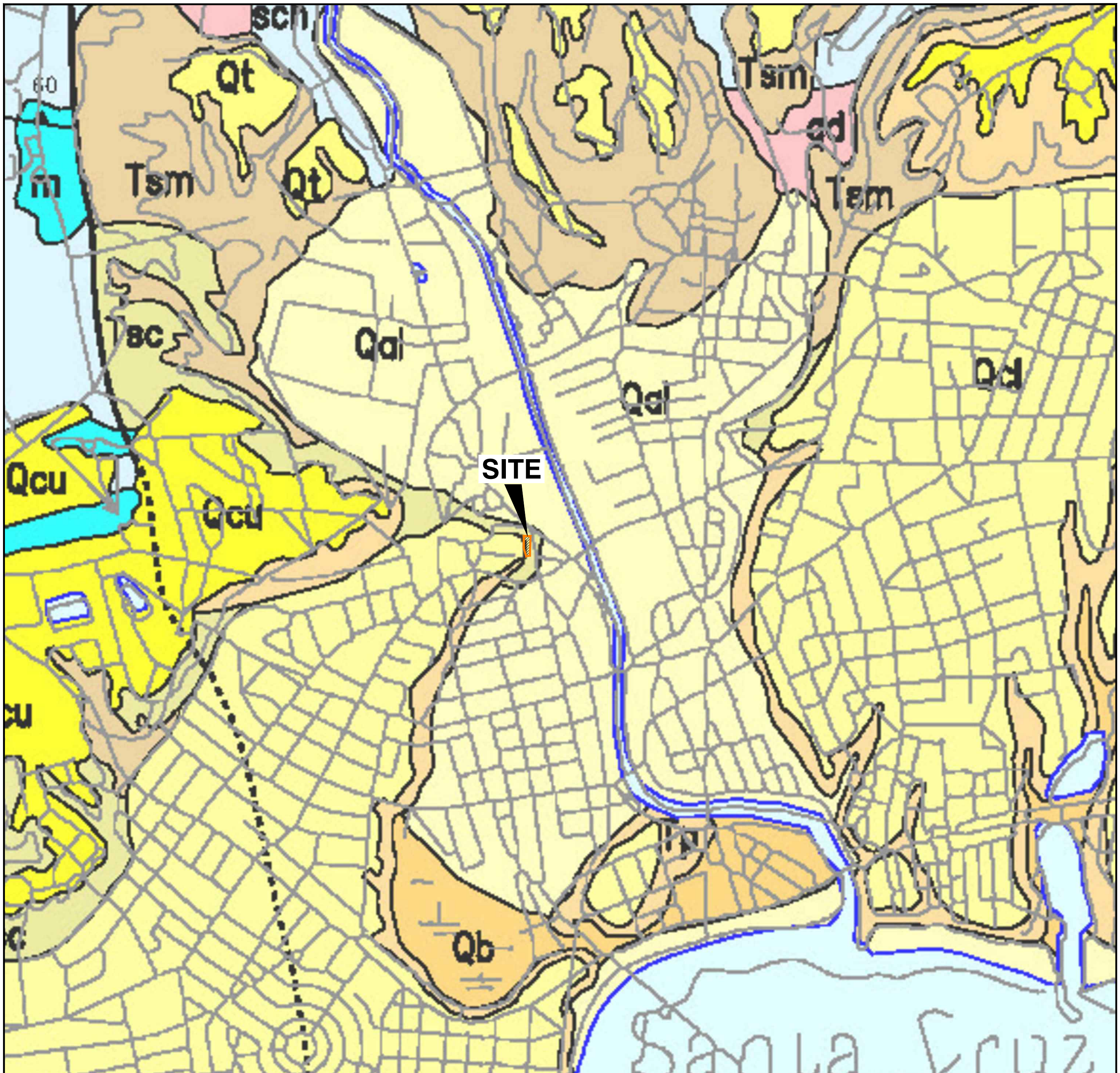

LEGEND

Qal	ALLUVIAL DEPOSITS (HOLOCENE)	Qt	TERRACE DEPOSITS, UNDIFFERENTIATED (PLEISTOCENE)	qd	QUARTZ DIORITE (CRETACEOUS)
Qb	BASIN DEPOSITS (HOLOCENE)	Tp	PURISIMA FORMATION (PLEISTOCENE & UPPER MIOCENE)	m	MARBLE (MESOZOIC OR PALEOZOIC)
Qcl	COLLUVIUM (HOLOCENE)	Tsc	SANTA CRUZ MUDSTONE (UPPER MIOCENE)	sch	METASEDIMENTARY ROCKS (MESOZOIC OR PALEOZOIC)
Qcu	COASTAL TERRACE DEPOSITS, UNDIFFERENTIATED (PLEISTOCENE)	Tsm	SANTA MARGARITA SANDSTONE (UPPER MIOCENE)		

GEOLOGIC CONTACT

N

FEET

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE | REFERENCE: USGS, BRABB, 1989

403215001_RG.dwg 03/22/2018 AEK

FIGURE 4

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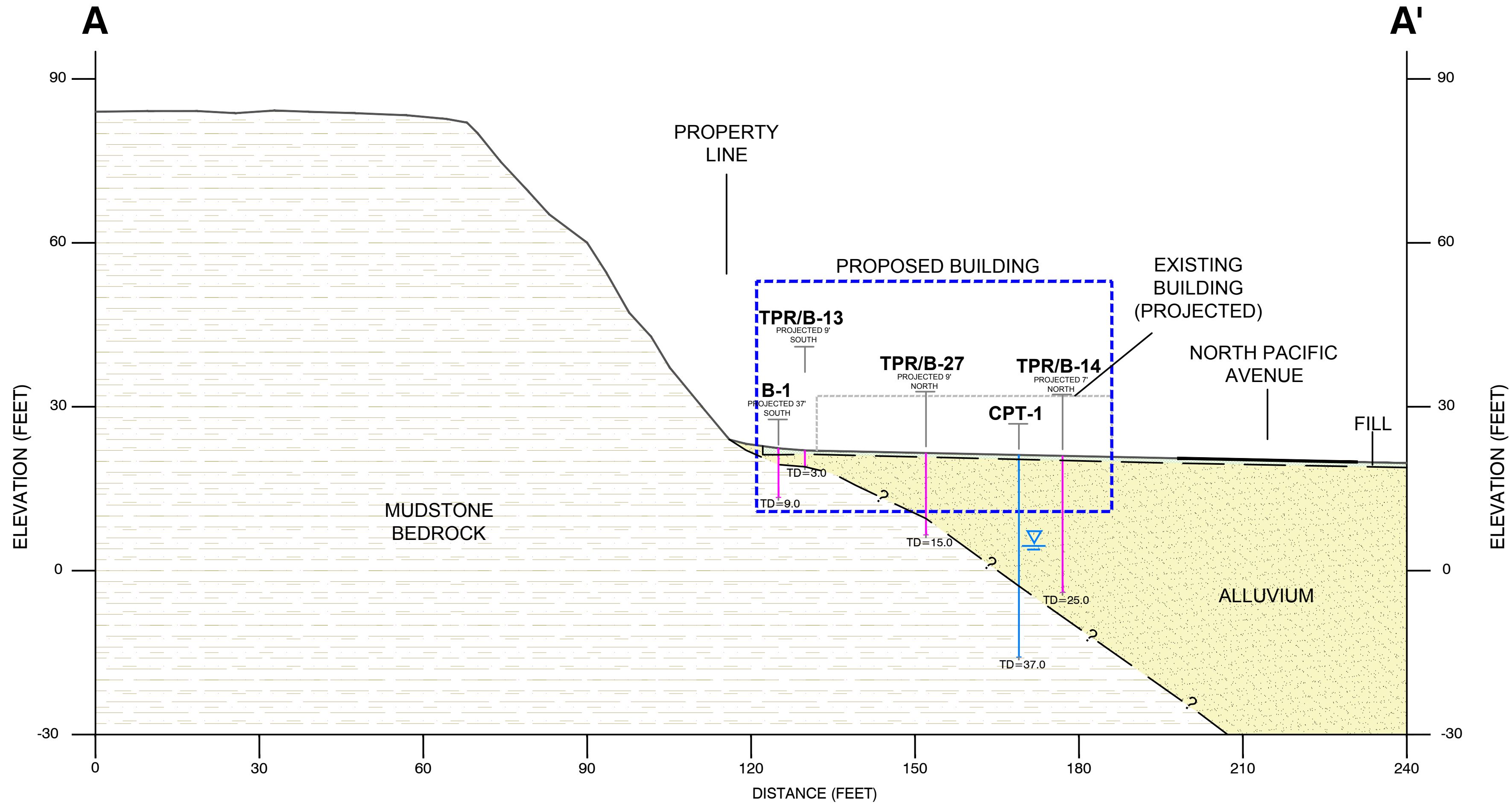
REGIONAL GEOLOGY

2035 NORTH PACIFIC AVENUE
SANTA CRUZ, CALIFORNIA
403215001 I 5/18



LEGEND

ALLUVIUM (HOLOCENE)	CPT-1: CORE PENETRATION TEST; TD=TOTAL DEPTH IN FEET
MUDSTONE BEDROCK	B-1: BORING; TD=TOTAL DEPTH IN FEET
GEOLOGIC CONTACT; QUERIED WHERE UNCERTAIN	GROUNDWATER LEVEL (AT TIME OF EXPLORATION)

FEET

HORIZ: 0 30 60

VERT: 0 30 60

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. TOPOGRAPHY BASED ON CITY OF SANTA CRUZ GIS DATA

FIGURE 5

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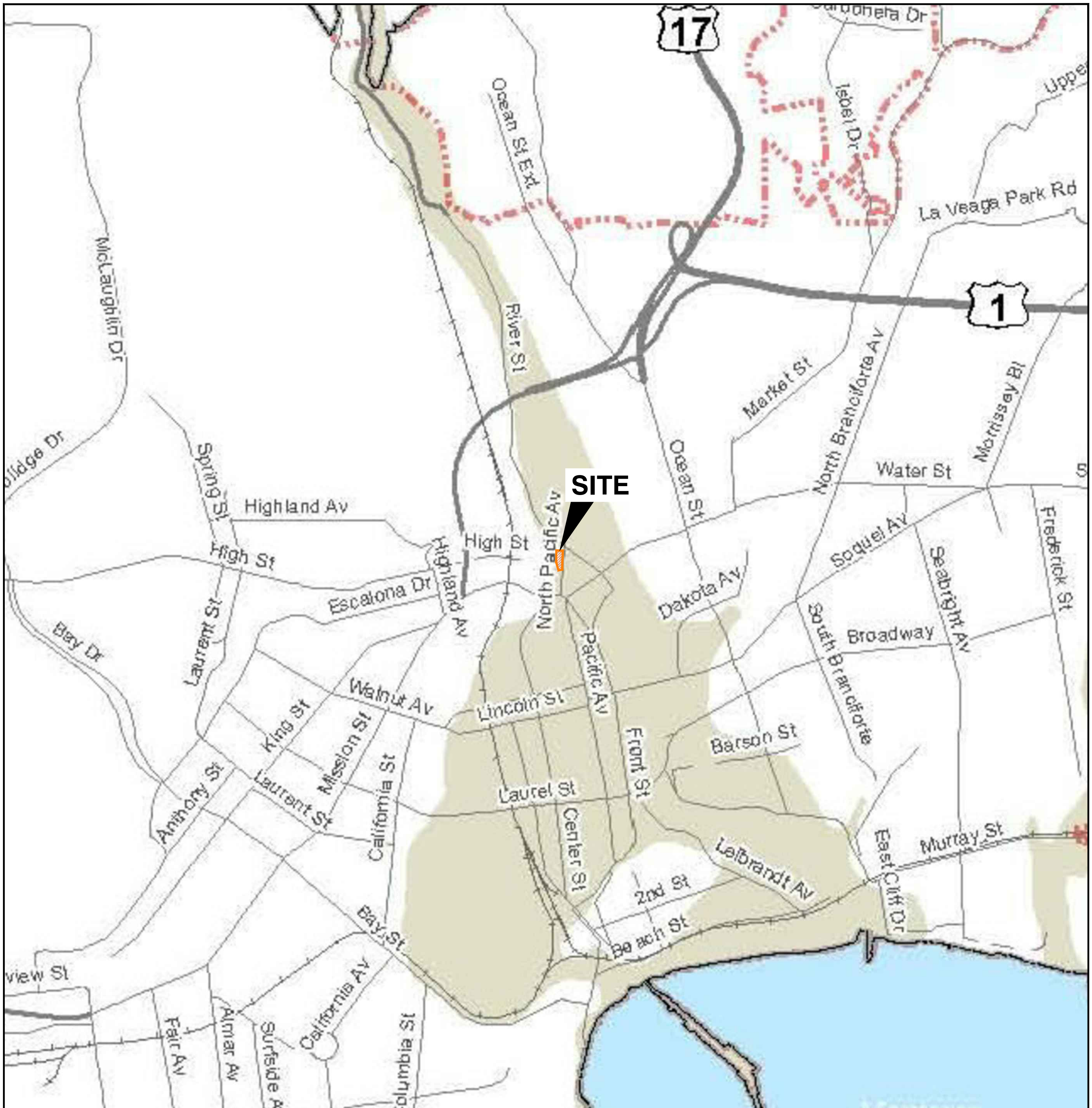
CROSS SECTION A-A'

2035 NORTH PACIFIC AVENUE
SANTA CRUZ, CALIFORNIA

403215001 | 5/18



LEGEND

City Limits

Liquefaction

Very High

REFERENCE: CITY OF SANTA CRUZ, 2017.

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

N

FEET

0 2,000 4,000

403215001_LHM.dwg 03/23/2018 JP

FIGURE 6

LIQUEFACTION HAZARD MAP

2035 NORTH PACIFIC AVENUE
SANTA CRUZ, CALIFORNIA
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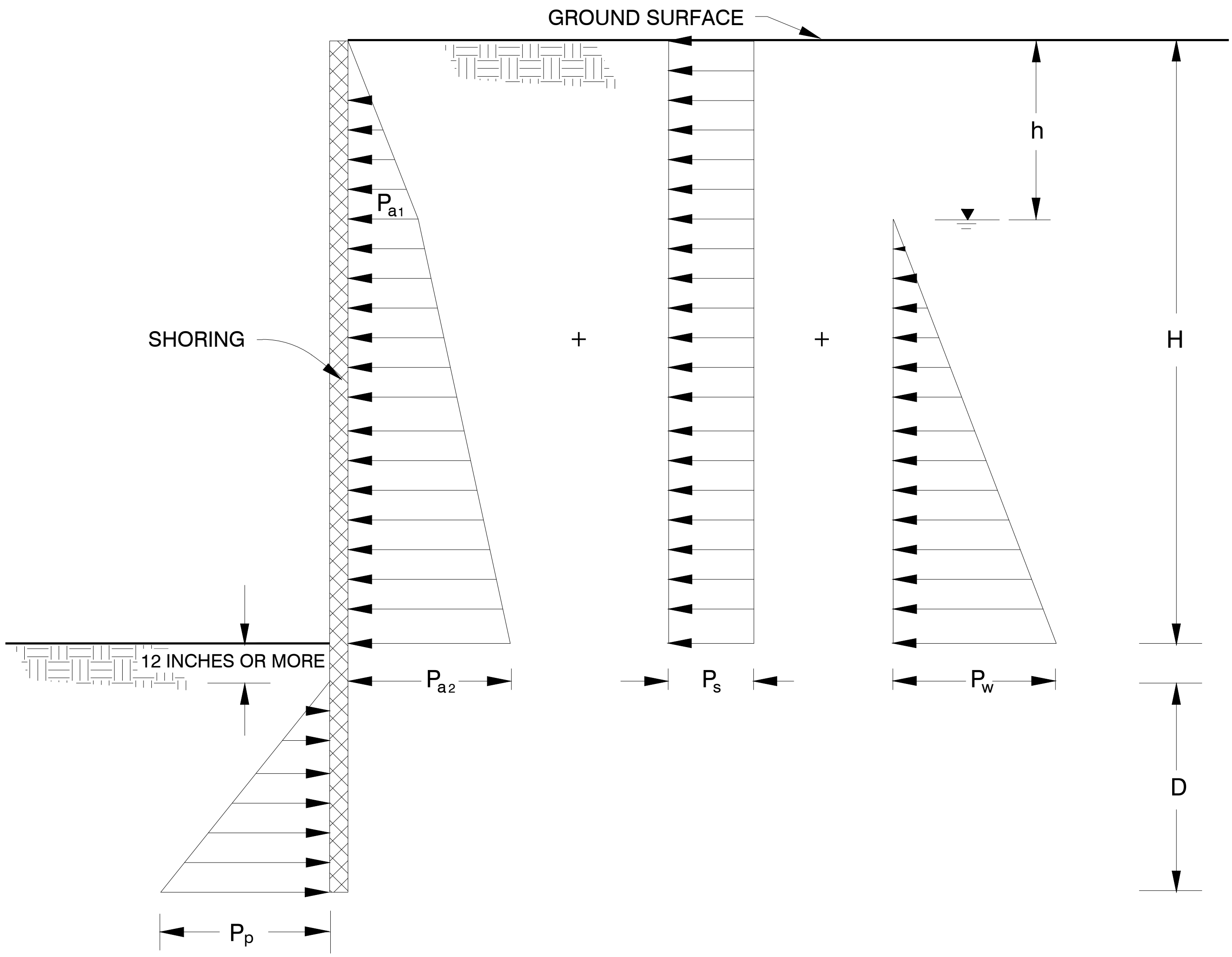

NOTES:

1. ACTIVE LATERAL EARTH PRESSURE, P_a
   P_{a1} = 35h psf; $P_{a2} = P_{a1}$ + 16 (H - h) psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   P_s = 120 psf
3. HYDROSTATIC PRESSURE, P_w
   P_w = 62.4 (H - h) psf
4. PASSIVE LATERAL EARTH PRESSURE, P_p
   P_p = 143D psf
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED.
6. H, h AND D ARE IN FEET
7. ▽ GROUNDWATER TABLE
8. ASSUMES NO SLOPES WITHIN 10 FEET OF TOP OF WALL

NOT TO SCALE

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 7

LATERAL EARTH PRESSURES FOR TEMPORARY CANTILEVERED SHORING BELOW GROUNDWATER

2035 NORTH PACIFIC AVENUE
SANTA CRUZ, CALIFORNIA

403215001 I 5/18

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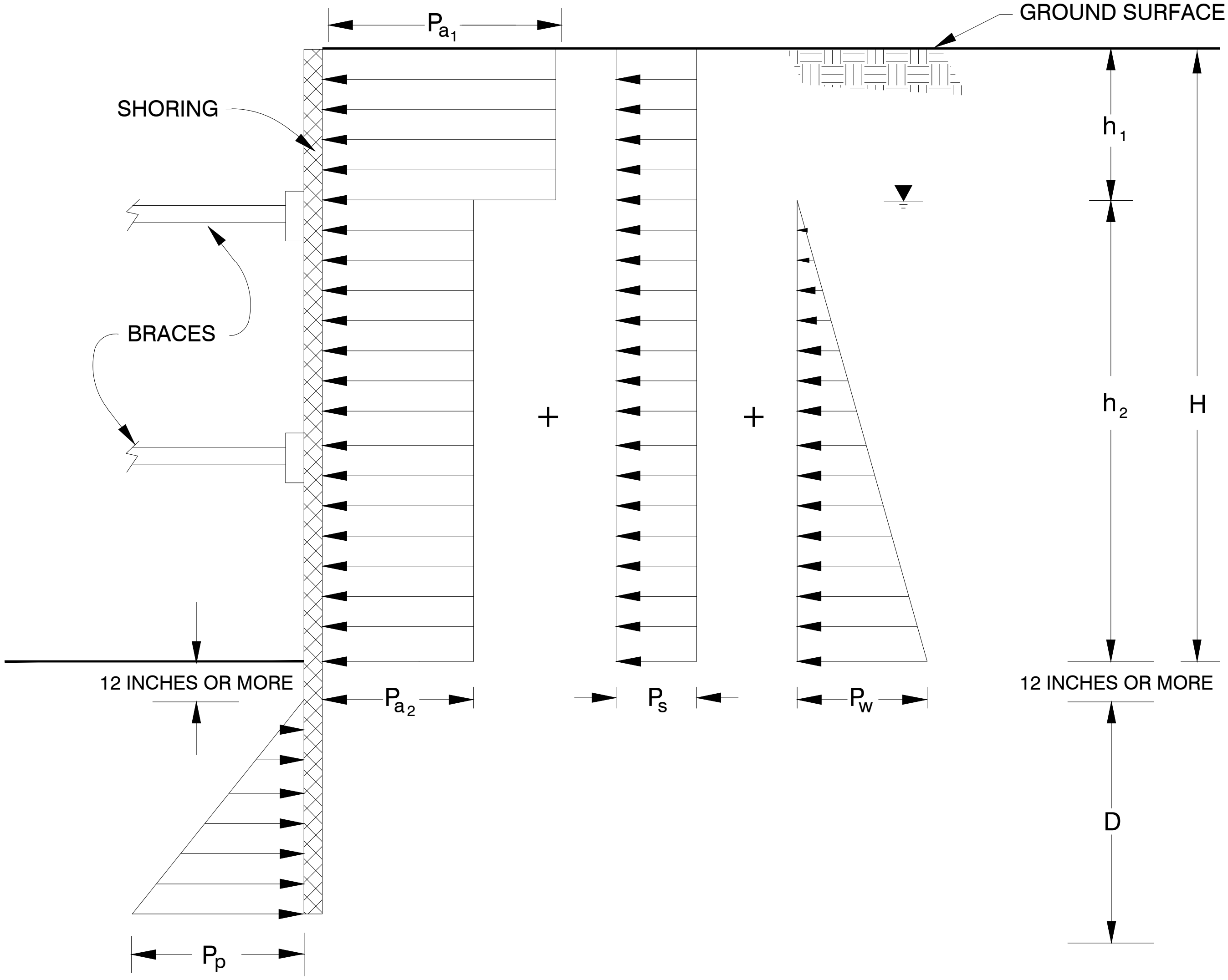

NOTES:

1. APPARENT LATERAL EARTH PRESSURES, P_{a1} AND P_{a2}
   $P_{a_1} = 23\ h_1$ psf
   $P_{a_2} = 11\ h_2$ psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   $P_s = 120$ psf
3. WATER PRESSURE, P_w
   $P_w = 62.4\ h_2$ psf
4. PASSIVE PRESSURE, P_p
   $P_p = 143$ D psf
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H, h_1, h_2 AND D ARE IN FEET
7. ▼ GROUNDWATER TABLE
8. ASSUMES NO SLOPES WITHIN 10 FEET OF TOP OF WALL.

NOT TO SCALE

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 8

LATERAL EARTH PRESSURES FOR BRACED EXCAVATION BELOW GROUNDWATER (GRANULAR SOIL)

2035 NORTH PACIFIC AVENUE
SANTA CRUZ, CALIFORNIA

403215001 I 5/18

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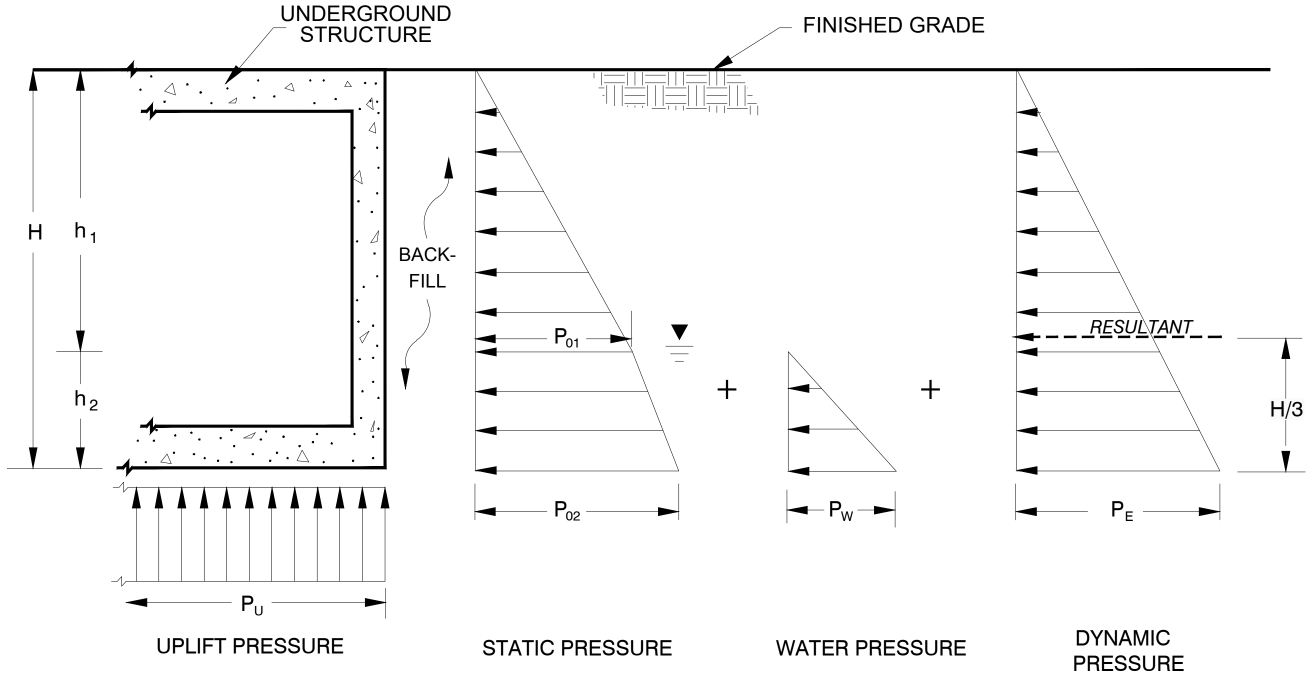

NOTES:

1. APPARENT LATERAL EARTH PRESSURES, P_{01} AND P_{02}
   $P_{01} = 54\ h_1$ psf
   $P_{02} = 54\ h_1 + 25\ h_2$ psf

2. WATER PRESSURE, P_w
   $P_w = 62.4\ h_2$ psf

3. DYNAMIC LATERAL EARTH PRESSURE, P_E
   $P_E = 12\ H$ psf

4. P_E IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010).

5. UPLIFT PRESSURE, P_u
   $P_u = 62.4\ h_2$ psf

6. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED

7. H, h_1 AND h_2 ARE IN FEET

8. ▼ GROUNDWATER TABLE

9. ASSUMES NO SLOPES WITHIN 10 FEET OF TOP OF WALL.

NOT TO SCALE

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FIGURE 9

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LATERAL EARTH PRESSURES FOR UNDERGROUND STRUCTURES

2035 NORTH PACIFIC AVENUE
SANTA CRUZ, CALIFORNIA

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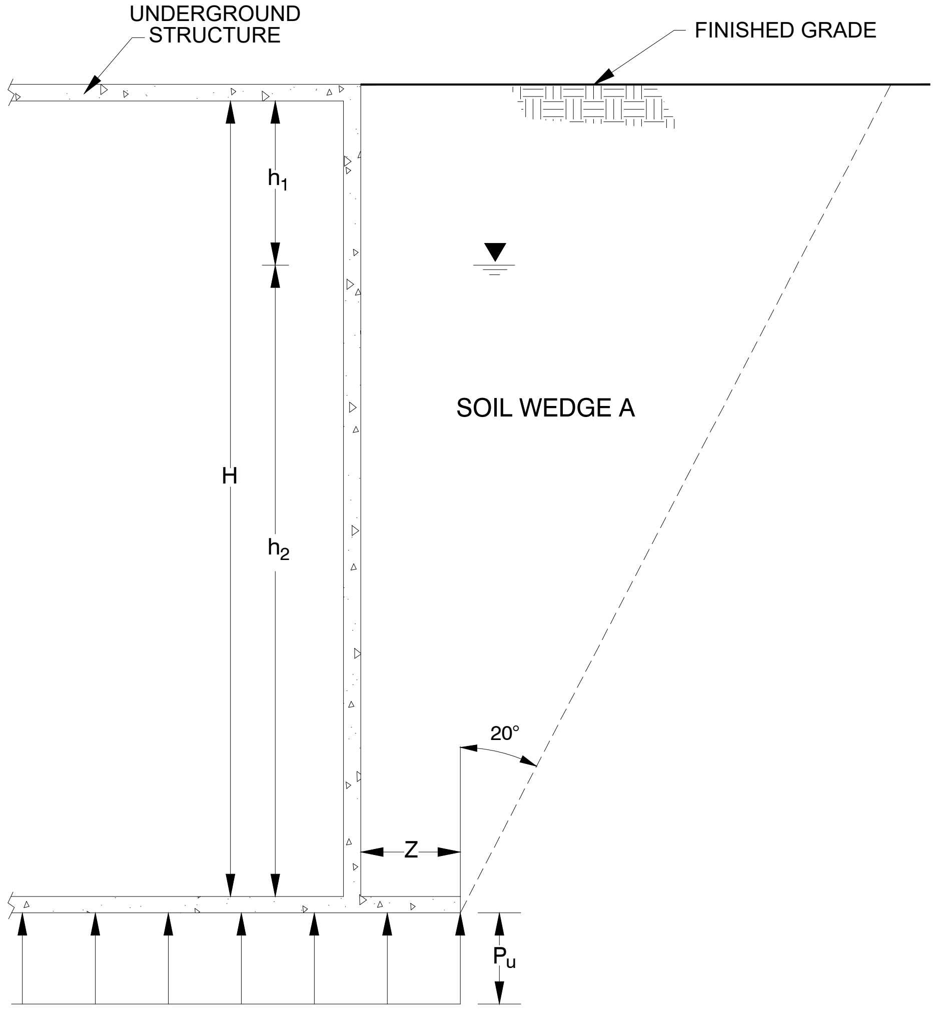

RESISTANCE TO UPLIFT = WEIGHT OF STRUCTURE + WEIGHT OF SOIL WEDGE A

NOTES:

1. UNIT WEIGHT OF SOILS, γ OR γ_b
   γ = 110 pcf ABOVE GROUNDWATER TABLE
   γ_b = 47 pcf BELOW GROUNDWATER TABLE
2. UPLIFT PRESSURE, P_u
   $P_u = 62.4\ h_2$ psf
3. H, Z, h_1 AND h_2 ARE IN FEET
4. ▼ GROUNDWATER TABLE

NOT TO SCALE

FIGURE 10

UPLIFT RESISTANCE DIAGRAM FOR UNDERGROUND STRUCTURES

2035 NORTH PACIFIC AVENUE
SANTA CRUZ, CALIFORNIA

403215001 I 5/18

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APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

Disturbed soil samples were obtained in the field using the following method.

Bulk Samples

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

The Standard Penetration Test (SPT) Sampler

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

Field Procedure for the Collection of Relatively Undisturbed Samples

Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler

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

BORING LOG EXPLANATION SHEET

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

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BORING LOG

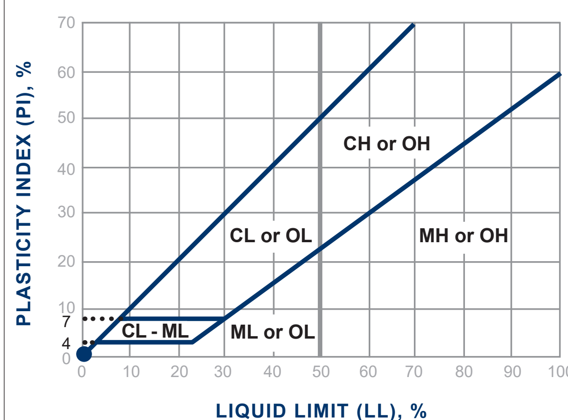
Soil Classification Chart Per ASTM D 2488

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

Grain Size

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

Plasticity Chart



Apparent Density - Coarse-Grained Soil

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

Consistency - Fine-Grained Soil

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

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USCS METHOD OF SOIL CLASSIFICATION

DATE DRILLED	3/14/18	BORING NO.	B-1
GROUND ELEVATION	24' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	8" HSA, B-53 Blue Truck Mounted (Exploration Geo), 3" HA top 5'		
DRIVE WEIGHT	140 LBS (wireline)	DROP	30 INCH
SAMPLED BY	KCC	LOGGED BY KCC	REVIEWED BY TPS

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 8 inches thick.
								AGGREGATE BASE: Approximately 6 inches thick.
							SW	FILL: Light brown, moist, dense, poorly-graded SAND.
								BEDROCK: Gray, moist, weathered MUDSTONE.
5			50/5"	25.8	79.3			
			50/6"					
10								Total Depth = Sampler refusal at 9 feet.
								Backfilled the hole with cement grout shortly after drilling.
								Notes:
								Groundwater, though not encounterd during drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
15								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
20								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 8 inches thick.
	X						SC	FILL: Light brown, moist, dense, clayey SAND.
	X							BEDROCK: Brown, moist, weathered MUDSTONE.
5		X	50/3"					
		X	50/6"					
10								Total Depth = Sampler refusal at 9.5 feet.
								Backfilled the hole with cement grout shortly after drilling.
								Notes:
								Groundwater, though not encounterd during drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
15								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
20								

DATE DRILLED 3/14/18 BORING NO. B-2

GROUND ELEVATION 24' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" HSA, B-53 Blue Truck Mounted (Exploration Geo), 3" HA top 5'

DRIVE WEIGHT 140 LBS (wireline) DROP 30 INCH

SAMPLED BY KCC LOGGED BY KCC REVIEWED BY TPS

FIGURE A- 2

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2035 NORTH PACIFIC AVENUE
SANTA CRUZ, CALIFORNIA
403215001 | 5/18

DATE DRILLED 3/14/18	BORING NO. B-3		
GROUND ELEVATION 22' ± (MSL)	SHEET 1 OF 2		
METHOD OF DRILLING 8" HSA, B-53 Blue Truck Mounted (Exploration Geo), 3" HA top 5'			
DRIVE WEIGHT 140 LBS (wireline)	DROP 30 INCH		
SAMPLED BY KCC	LOGGED BY KCC	REVIEWED BY TPS	

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 3 inches thick.
								AGGREGATE BASE: Approximately 2 inches thick.
	X						CL	FILL: Light brown, moist, firm, lean CLAY; trace sand, trace gravel. Cobble.
5		X	15				SC	ALLUVIUM: Brown, moist, medium dense, clayey SAND.
		X	33					BEDROCK: Gray, moist, weathered MUDSTONE.
10								
		X	50/6"	25.6				
15								
		X	75	34.9				
20								

FIGURE A- 3

2035 NORTH PACIFIC AVENUE
SANTA CRUZ, CALIFORNIA

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

DATE DRILLED 3/14/18 BORING NO. B-3

GROUND ELEVATION 22' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" HSA, B-53 Blue Truck Mounted (Exploration Geo), 3" HA top 5'

DRIVE WEIGHT 140 LBS (wireline) DROP 30 INCH

SAMPLED BY KCC LOGGED BY KCC REVIEWED BY TPS

DESCRIPTION/INTERPRETATION

20

Total Depth = 20 feet.

Backfilled the hole with cement grout shortly after drilling.

Notes:

Groundwater, though not encounterd during drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

25

30

35

40

APPENDIX B

Cone Penetration Testing

APPENDIX B

CONE PENETRATION TESTING

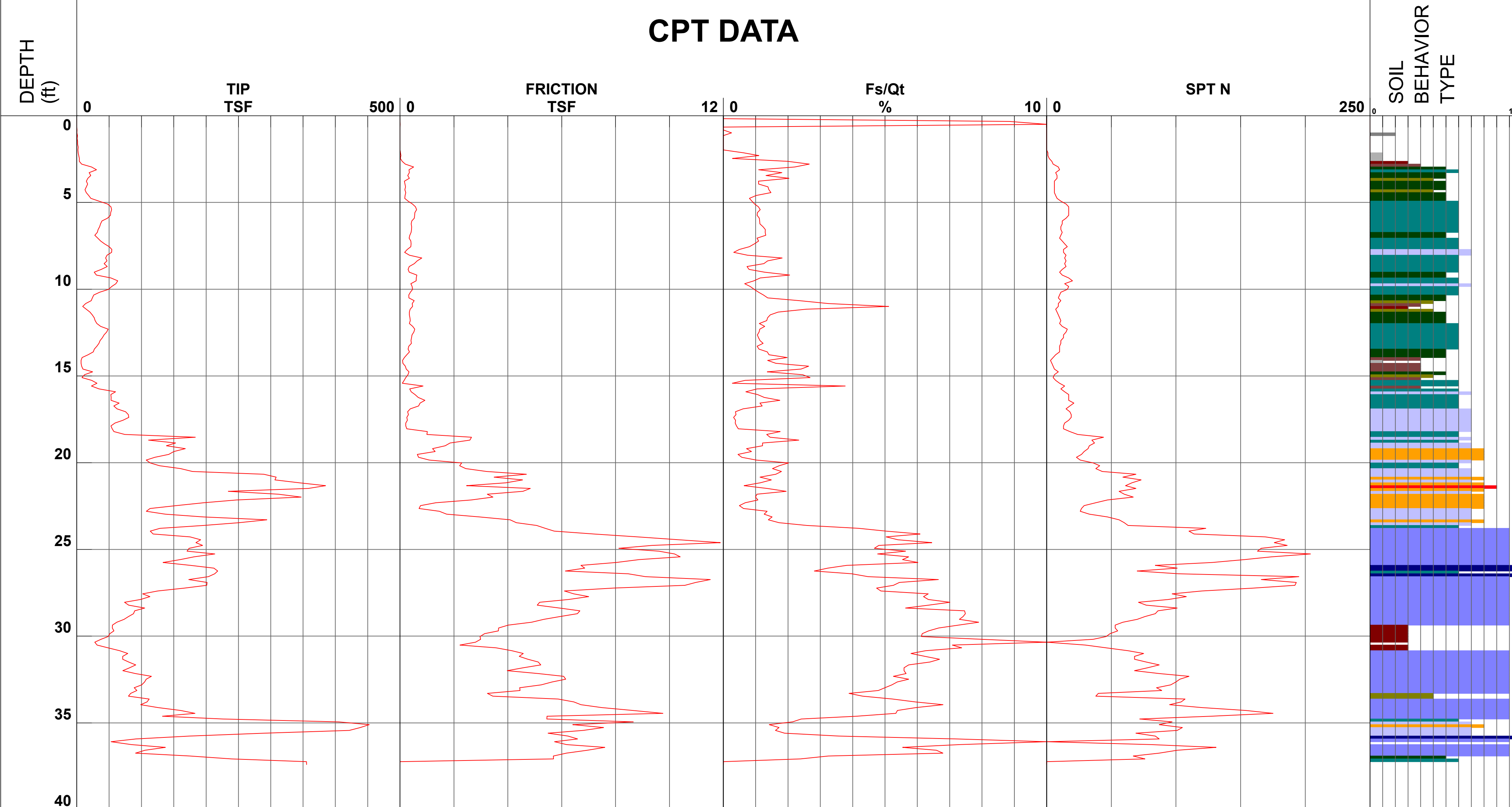
Field Procedure for Cone Penetration Testing

A penetrometer with a conical tip having an apex angle of 60 degrees and a cone base area of 10 square centimeters was hydraulically pushed through the soil using the reaction mass of a 20-ton rig at a constant rate of about 20 millimeter per second in accordance with ASTM D 5778. The penetrometer was instrumented to measure, by electronic methods, the force on the conical point required to penetrate the soil, the force on a friction sleeve behind the cone tip as the penetrometer was advanced, and the pore pressure (Pw) on a transducer behind the cone tip. Penetration data was collected and recorded electronically at intervals of about 2-inches. Cone resistance (Q_c) was calculated by dividing the measured force of penetration by the cone base area. Friction sleeve resistance (F_s) was calculated by dividing the measured force on the friction sleeve by the surface area of the sleeve. The friction ratio (Fs/Qc) was calculated as the ratio of the tip resistance to the sleeve friction. A graph of the computed values of cone resistance (tip) and friction ratio are presented on the logs in the following pages. The tip resistance and friction ratio were used to classify the soil type encountered using the method by Robertson & Campanella (1986). Equivalent SPT blowcounts at a 60 percent energy ratio (N_{60}-values) were calculated from the tip resistance and friction ratio using the method by Jeffries and Davies (1993). A graph of the equivalent N_{60} values (SPT N_{eq}) and the encountered soil types are also presented on the logs in the following pages.

Ninyo & Moore

Project	Slatter-2035 N Pacific Ave-Geo	Operator	BH-RB	Filename	SDF(599).cpt
Job Number	403215001	Cone Number	DDG1281	GPS	
Hole Number	CPT-01	Date and Time	3/10/2018 8:13:47 AM	Maximum Depth	37.40 ft
EST GW Depth During Test		16.10 ft			

Net Area Ratio .8



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

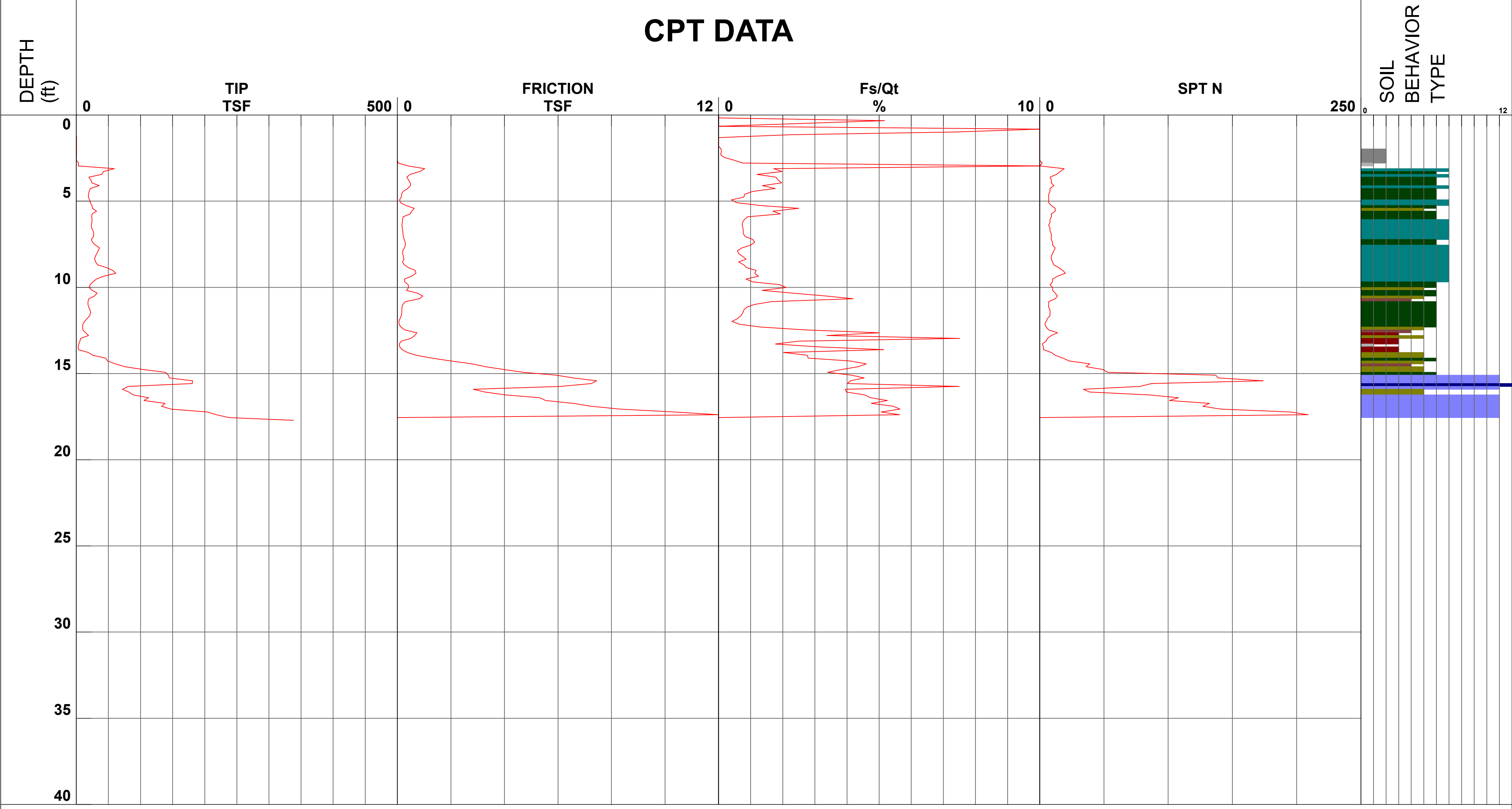
S*Soil behavior type and SPT based on data from UBC-1983

Ninyo & Moore

Project	Slatter-2035 N Pacific Ave-Geo	Operator	BH-RB	Filename	SDF(601).cpt
Job Number	403215001	Cone Number	DDG1281	GPS	
Hole Number	CPT-02	Date and Time	3/10/2018 11:14:05 AM	Maximum Depth	17.72 ft
EST GW Depth During Test		16.00 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

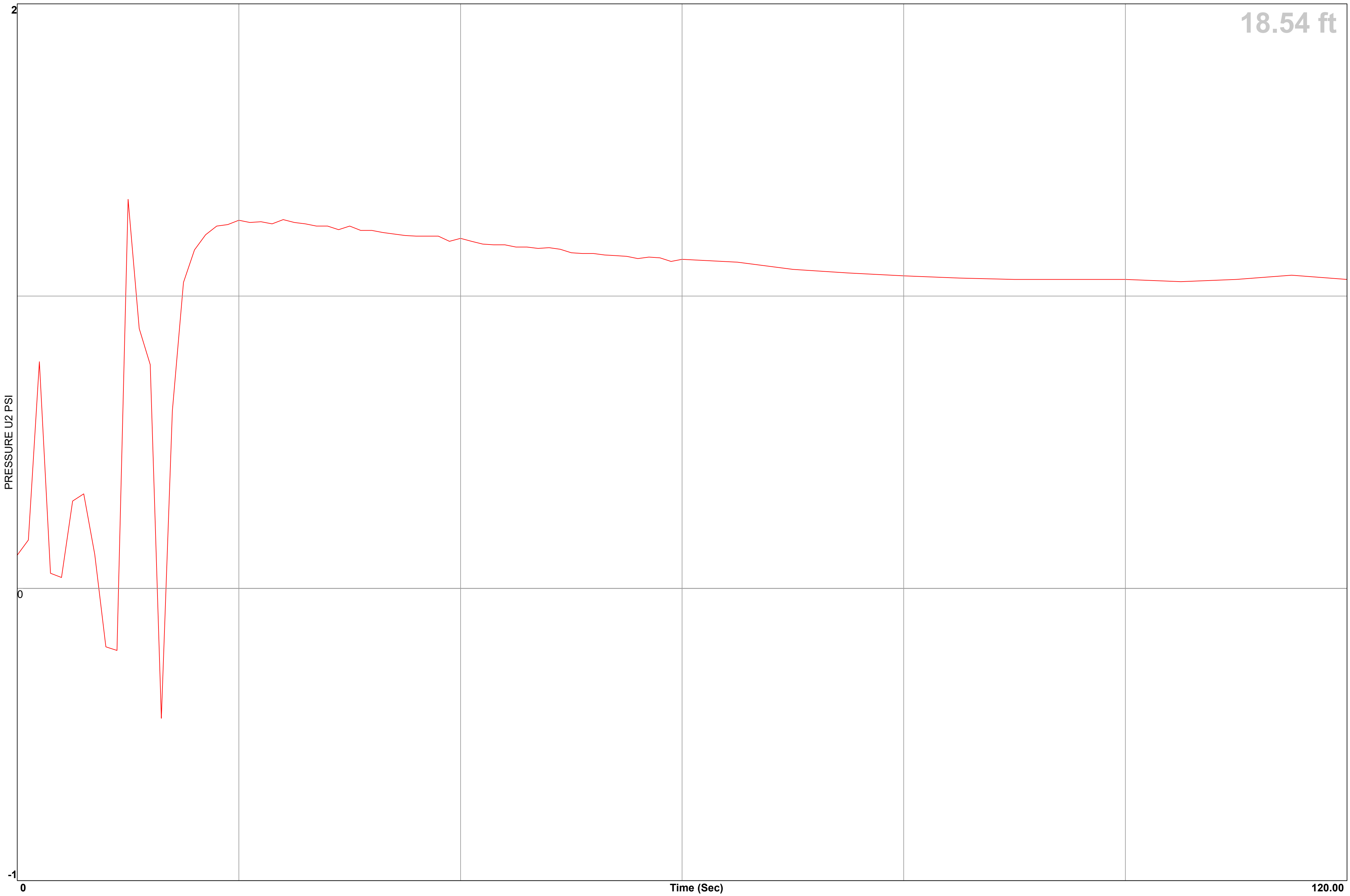
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Ninyo & Moore

Location	Slatter-2035 N Pacific Ave-Geo	Operator	BH-RB
Job Number	403215001	Cone Number	DDG1281
Hole Number	CPT-01	Date and Time	3/10/2018 8:13:47 AM
Equilized Pressure	1.0	EST GW Depth During Test	16.1

GPS



Project ID: Ninyo & Moore
Data File: SDF(599).cpt
CPT Date: 3/10/2018 8:13:47 AM
GW During Test: 16 ft

Sounding ID: CPT-01
Project No: 403215001
Cone/Rig: DDG1281

Depth ft	qc PS tsf	* qc1n PS -	q1ncs PS -	* qt PS tsf	Slv Stss tsf	pore prss (psi)	Frct Rato %	* Material Behavior Description	Unit Wght pcf	Qc to N	* SPT R-N1 60%	SPT R-N 60%	* SPT IcN1 60%	* Rel Den %	* Ftn Ang deg	Und Shr tsf	OCR - -	* Fin Ic %	* Ic SBT Indx	* Nk -
0.33	1.1	0.0	-	1.1	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	1	1	1	-	-	0.1	9.9	95	3.48	15
0.49	1.1	0.0	-	1.1	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	0	1	0	-	-	0.1	9.9	95	3.48	15
0.66	1.1	0.0	-	1.1	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	0	1	0	-	-	0.1	9.3	95	3.48	15
0.82	1.1	0.8	-	1.1	0.0	0.1	0.1	sensitive fine SOIL	115	2.0	0	1	0	-	-	0.1	7.4	95	3.64	15
0.98	1.1	0.3	-	1.1	0.0	0.0	0.4	sensitive fine SOIL	115	2.0	0	1	0	-	-	0.1	6.1	95	4.28	15
1.15	1.0	1.6	-	1.0	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	1	1	1	-	-	0.1	4.8	81	3.29	15
1.31	1.1	1.0	-	1.1	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	0	1	0	-	-	0.1	4.5	95	3.56	15
1.48	1.3	2.0	-	1.3	0.0	0.2	0.1	sensitive fine SOIL	115	2.0	1	1	1	-	-	0.1	4.5	74	3.21	15
1.64	1.2	2.0	-	1.2	0.0	0.3	0.1	sensitive fine SOIL	115	2.0	1	1	1	-	-	0.1	4.0	74	3.22	15
1.80	2.2	3.5	-	2.2	0.0	0.3	0.1	sensitive fine SOIL	115	2.0	2	1	1	-	-	0.1	6.6	56	2.96	15
1.97	1.8	2.9	-	1.8	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	1	1	1	-	-	0.1	4.9	62	3.05	15
2.13	2.1	3.3	-	2.1	0.0	-0.1	0.7	sensitive fine SOIL	115	2.0	2	1	1	-	-	0.1	5.3	69	3.15	15
2.30	2.9	4.7	-	2.9	0.0	0.0	1.1	silty CLAY to CLAY	115	1.5	3	2	2	-	-	0.2	6.9	65	3.10	15
2.46	3.8	6.2	-	3.8	0.0	0.1	0.3	sensitive fine SOIL	115	2.0	3	2	2	-	-	0.3	8.6	45	2.78	15
2.62	4.0	6.4	-	4.0	0.1	0.1	2.1	silty CLAY to CLAY	115	1.5	4	3	2	-	-	0.3	8.4	65	3.09	15
2.79	7.4	11.9	-	7.4	0.2	0.4	2.7	silty CLAY to CLAY	115	1.5	8	5	4	-	-	0.5	9.9	53	2.92	15
2.95	22.8	36.6	96.0	22.9	0.5	1.5	2.2	silty SAND to sandy SILT	120	4.0	9	6	9	34	42	-	-	29	2.47	16
3.12	30.5	49.0	79.7	30.5	0.3	0.5	1.1	silty SAND to sandy SILT	120	4.0	12	8	10	43	43	-	-	18	2.18	16
3.28	19.0	30.5	81.6	19.0	0.3	1.0	1.8	silty SAND to sandy SILT	120	4.0	8	5	7	28	41	-	-	30	2.48	16
3.45	21.6	34.7	74.1	21.6	0.3	0.2	1.3	silty SAND to sandy SILT	120	4.0	9	5	8	32	41	-	-	25	2.35	16
3.61	17.1	27.4	83.1	17.1	0.3	0.3	2.1	clayy SILT to silty CLAY	115	2.0	14	9	7	-	-	1.2	9.9	33	2.55	15
3.77	14.8	23.8	60.2	14.9	0.2	1.2	1.1	silty SAND to sandy SILT	120	4.0	6	4	6	20	38	-	-	29	2.45	16
3.94	16.8	26.9	62.7	16.8	0.2	1.1	1.1	silty SAND to sandy SILT	120	4.0	7	4	6	24	39	-	-	27	2.41	16
4.10	14.8	23.8	66.5	14.8	0.2	1.0	1.4	silty SAND to sandy SILT	120	4.0	6	4	6	20	38	-	-	31	2.51	16
4.27	12.6	20.2	64.0	12.6	0.2	1.0	1.4	clayy SILT to silty CLAY	115	2.0	10	6	5	-	-	0.9	9.9	34	2.57	15
4.43	14.5	23.3	68.1	14.5	0.2	0.8	1.5	silty SAND to sandy SILT	120	4.0	6	4	6	19	37	-	-	32	2.53	16
4.59	18.5	29.6	63.2	18.5	0.2	1.3	1.0	silty SAND to sandy SILT	120	4.0	7	5	7	27	38	-	-	25	2.35	16
4.76	21.9	35.1	61.8	21.9	0.2	1.3	0.8	silty SAND to sandy SILT	120	4.0	9	5	8	32	39	-	-	20	2.24	16
4.92	34.1	54.8	78.9	34.2	0.3	0.5	0.9	silty SAND to sandy SILT	120	4.0	14	9	11	47	41	-	-	16	2.09	16
5.09	47.6	76.3	98.9	47.6	0.5	0.5	1.0	clean SAND to silty SAND	125	5.0	15	10	15	58	43	-	-	13	2.00	16
5.25	52.8	84.7	109.2	52.8	0.6	0.1	1.1	clean SAND to silty SAND	125	5.0	17	11	17	62	43	-	-	13	1.99	16
5.41	54.0	86.6	112.3	54.0	0.6	-0.1	1.1	clean SAND to silty SAND	125	5.0	17	11	17	62	43	-	-	13	2.00	16
5.58	52.5	84.2	108.0	52.5	0.6	0.0	1.1	clean SAND to silty SAND	125	5.0	17	10	16	61	43	-	-	13	1.99	16
5.74	52.0	83.4	107.0	52.0	0.5	0.0	1.1	clean SAND to silty SAND	125	5.0	17	10	16	61	43	-	-	13	1.99	16
5.91	47.4	76.0	102.6	47.4	0.5	-0.2	1.1	clean SAND to silty SAND	125	5.0	15	9	15	58	42	-	-	14	2.04	16
6.07	39.0	62.5	91.6	39.0	0.4	-0.5	1.1	silty SAND to sandy SILT	120	4.0	16	10	13	51	41	-	-	16	2.11	16
6.23	36.7	58.8	88.9	36.7	0.4	-0.2	1.1	silty SAND to sandy SILT	120	4.0	15	9	12	49	40	-	-	17	2.13	16
6.40	35.4	55.9	89.1	35.4	0.4	0.0	1.2	silty SAND to sandy SILT	120	4.0	14	9	12	48	40	-	-	18	2.17	16
6.56	33.0	51.5	87.5	33.0	0.4	-0.2	1.3	silty SAND to sandy SILT	120	4.0	13	8	11	45	40	-	-	19	2.21	16
6.73	31.8	49.0	85.4	31.8	0.4	0.0	1.3	silty SAND to sandy SILT	120	4.0	12	8	11	43	39	-	-	20	2.23	16
6.89	27.8	42.3	80.7	27.8	0.4	-0.1	1.3	silty SAND to sandy SILT	120	4.0	11	7	9	39	38	-	-	22	2.29	16
7.05	32.5	48.9	78.7	32.5	0.3	0.3	1.1	silty SAND to sandy SILT	120	4.0	12	8	10	43	39	-	-	18	2.18	16
7.22	36.8	54.7	84.7	36.9	0.4	1.8	1.1	silty SAND to sandy SILT	120	4.0	14	9	11	47	40	-	-	17	2.15	16
7.38	42.9	63.0	87.8	42.9	0.4	0.6	1.0	silty SAND to sandy SILT	120	4.0	16	11	13	52	40	-	-	15	2.06	16
7.55	50.1	72.8	91.8	50.2	0.4	1.9	0.8	clean SAND to silty SAND	125	5.0	15	10	14	57	41	-	-	12	1.97	16
7.71	54.2	77.8	87.8	54.2	0.3	1.8	0.5	clean SAND to silty SAND	125	5.0	16	11	14	59	41	-	-	9	1.83	16
7.87	53.8	76.4	81.7	53.8	0.2	0.9	0.3	clean SAND to silty SAND	125	5.0	15	11	14	58	41	-	-	7	1.75	16
8.04	46.9	65.9	84.5	46.9	0.4	0.5	0.8	clean SAND to silty SAND	125	5.0	13	9	13	53	40	-	-	13	1.99	16
8.20	44.6	62.1	110.4	44.6	0.8	0.5	1.8	silty SAND to sandy SILT	120	4.0	16	11	13	51	40	-	-	20	2.24	16
8.37	46.6	64.1	99.9	46.7	0.6	4.0	1.4	silty SAND to sandy SILT	120	4.0	16	12	13	52	40	-	-	17	2.15	16
8.53	42.4	57.8	91.5	42.4	0.5	1.1	1.3	silty SAND to sandy SILT	120	4.0	14	11	12	49	40	-	-	18	2.16	16
8.69	47.1	63.5	82.2	47.1	0.3	0.6	0.7	clean SAND to silty SAND	125	5.0	13	9	12	52	40	-	-	13	1.99	16
8.86	37.7	50.4	73.4	37.7	0.3	0.3	0.8	silty SAND to sandy SILT	120	4.0	13	9	10	44	39	-	-	16	2.10	16
9.02	26.9	35.6	74.6	26.9	0.3	-0.1	1.3	silty SAND to sandy SILT	120	4.0	9	7	8	33	37	-	-	24	2.34	16
9.19	30.4	39.9	96.9	30.4	0.6	-0.2	2.1	silty SAND to sandy SILT	120	4.0	10	8	9	37	37	-	-	28	2.43	16
9.35	52.8	68.7	97.5	52.8	0.6	0.3	1.2	silty SAND to sandy SILT	120	4.0	17	13	14	55	40	-	-	15	2.08	16
9.51	63.3	81.7	103.1	63.4	0.6	2.2	1.0	clean SAND to silty SAND	125	5.0	16	13	16	60	41	-	-	12	1.97	16
9.68	60.5	77.4	91.9	60.6	0.4	0.8	0.7	clean SAND to silty SAND	125	5.0	15	12	15	59	41	-	-	10	1.90	16
9.84	54.1	68.5	88.9	54.1	0.4	0.3	0.8	clean SAND to silty SAND	125	5.0	14	11	13	55	40	-	-	13	2.00	16
10.01	49.0	61.5	86.6	49.0	0.5	-0.1	1.0	silty SAND to sandy SILT	120	4.0	15	12	12	51	39	-	-	15	2.07	16
10.17	35.3	44.0	76.5	35.3	0.4	0.7	1.1	silty SAND to sandy SILT	120	4.0	11	9	9	40	37	-	-	20	2.23	16
10.34	26.2	32.3	71.4	26.2	0.3	0.4	1.3	silty SAND to sandy SILT	120	4.0	8	7	7	30	36	-	-	25	2.37	16
10.50	24.0	29.4	72.3	24.0	0.3	0.6	1.4	silty SAND to sandy SILT	120	4.0	7	6	7	27	35	-	-	28	2.43	16
10.66	22.2	29.9	94.1	22.2	0.5	0.2	2.4	clayy SILT to silty CLAY	115	2.0	15	11	8	-	-	1.5	9.9	34	2.57	15
10.83	13.9	21.5	-	13.9	0.5	1.4	3.4	clayy SILT to silty CLAY	115	2.0	11	7	6	-	-	0.9	6.8	45	2.78	15
10.99	9.1	13.9	-	9.3	0.5	7.4	5.5	silty CLAY to CLAY	115	1.5	9	6	5	-	-	0.6	4.3	63	3.07	15
11.16	14.6	21.9	-	14.8	0.4	9.0	2.7	clayy SILT to silty CLAY	115	2.0	11	7	6	-	-	1.0	6.9	41	2.71	15
11.32	20.4	24.1	74.7	20.4	0.3	-2.2	1.7	clayy SILT to silty CLAY	115	2.0	12	10	6	-	-	1.4	9.6	33	2.56	15
11.48	23.9	28.1	73.1	23.9	0.3	-0.8	1.5	silty SAND to sandy SILT	120	4.0	7	6	7	25	34	-	-	29	2.47	16
11.65	26.9	31.3	73.8	26.8	0.4	-0.3	1.4	silty SAND to sandy SILT	120	4.0	8	7	7	29	35	-	-	27	2.41	16
11.81	28.4	32.9	74.4	28.4	0.4	-0.2	1.4	silty SAND to sandy SILT	120	4.0	8	7	8	30	35	-	-	26	2.39	16
11.98	31.1	35.8	71.0	31.1	0.3	-0.2	1.1	silty SAND to sandy SILT	120	4.0	9	8	8	33	36	-	-	23	2.31	16
12.14	36.9	42.1	80.1	36.9	0.5	-0.1	1.3	silty SAND to sandy SILT	120	4.0	11	9	9	38	37	-	-	22	2.28	16
12.30	48.6	55.1	86.2	48.6	0.5	-0.8	1.1	silty SAND to sandy SILT	120	4.0	14	12	12	47	38	-	-	18	2.16	16
12.47	46.2	52.1	83.3	46.2	0.5	-1.3	1.1	silty SAND to sandy SILT	120	4.0	13	12	11	45	38	-	-	18	2.17	16
12.63	42.1	47.1	77.9	42.1	0.4	-0.6	1.1	silty SAND to sandy SILT	120	4.0	12	11	10	42	37	-	-	19	2.19	16
12.80	39.5	43.9	76.4	39.5	0.4	-0.3	1.1	silty SAND to sandy SILT	120	4.0	11	10	9	40	37	-	-	20	2.23	16
12.96	35.9	39.7	74.5	35.9	0.4	0.1	1.2	silty SAND to sandy SILT	120	4.0	10	9	9	36	36	-	-	22	2.28	16
13.12	34.1	37.5	75.3	34.1	0.4	-0.3	1.3	silty SAND to sandy SILT	120	4.0	9	9	8	35	36	-	-	23	2.32	16
13.29	31.1	33.9	68.0	31.1	0.3	-0.6	1.1	silty SAND to sandy SILT	120	4.0	8	8	8	31	35	-	-	23	2.32	16
13.45	27.3	29.5	66.2	27.3	0.3	0.1	1.1	silty SAND to sandy SILT	120	4.0	7	7	7	27	34	-	-	26	2.38	16
13.62	25.6	27.6	70.8	25.6	0.3	-0.6	1.4	silty SAND to sandy SILT	120	4.0	7	6	7	25	34	-	-	29	2.46	16
13.78	17.6	19.8	65.6	17.6	0.2	-0.6	1.5	clayy SILT to silty CLAY	115	2.0	10	9	5	-	-	1.2	6.7	35	2.60	15
13.94	8.2	9.9	-	8.3	0.2	3.7	2.2	silty CLAY to CLAY	115	1.5	7	5	3	-	-	0.5	2.9	56	2.97	15
14.11	6.7	8.0	-	7.0	0.1	13.9	1.6	silty CLAY to CLAY	115	1.5	5	4	3	-	-	0.4	2.3	57	2.98	15
14.27	6.5	7.7	-	6.9	0.1	18.4	1.9	silty CLAY to CLAY	115	1.5	5	4	3	-	-	0.4	2.2	61	3.04	15
14.44	7.5	8.7	-	7.8	0.2	19.2	3.0	silty CLAY to CLAY	115	1.5	6	5	3	-	-	0.5	2.5	65	3.09	15
14.60	9.5	10.9	-	9.9	0.2	21.4	2.6	silty CLAY to CLAY	115	1.5	7	6	3	-	-	0.6	3.3	56	2.97	15
14.76	24.3	25.2	69.0	24.4	0.3	2.0	1.4	silty SAND to sandy SILT	120	4.0	6	6	6	22	33	-	-	30	2.49	16
14.93	12.0	13.5	-	12.1	0.3	3.4	2.6	silty CLAY to CLAY	115	1.5	9	8	4	-	-	0.8	4.1	51	2.89	15
15.09	8.0	8.9	-	8.4	0.2	19.4	3.0	silty CLAY to CLAY	115	1.5	6	5	3	-	-	0.5	2.6	64	3.08	15
15.26	23.0	23.4	51.4	23.6	0.2	30.4	0.7	silty SAND to sandy SILT	120	4.0	6	6	5	19	32	-	-	25	2.37	16
15.42	31.2	31.7	45.7	31.2	0.1	-2.0	0.3	silty SAND to sandy SILT	120	4.0	8	8	6	29	34	-	-	16	2.09	16

* Indicates the parameter was calculated using the normalized point stress.
The parameters listed above were determined using empirical correlations.
A Professional Engineer must determine their suitability for analysis and design.

Middle Earth Geo Testing

Depth ft	qc PS tsf	* qc1n PS -	q1ncs PS -	* qt PS tsf	Slv Stss tsf	pore prss (psi)	Frct Ratio %	* Material Behavior Description	Unit Wght pcf	Qc to N	* SPT R-N1 60%	SPT R-N 60%	* SPT IcN1 60%	* Rel Den %	* Ftn Ang deg	Und Shr tsf	OCR -	* Fin Ic %	* Ic SBT Indx	* Nk -
15.58	22.7	24.4	-	22.6	0.9	-0.9	3.9	clayy SILT to silty CLAY	115	2.0	12	11	7	-	-	1.5	7.7	45	2.78	15
15.75	34.5	34.6	68.6	34.5	0.4	-1.2	1.1	silty SAND to sandy SILT	120	4.0	9	9	8	32	35	-	-	23	2.31	16
15.91	59.8	59.7	78.3	59.8	0.4	-0.6	0.7	clean SAND to silty SAND	125	5.0	12	12	12	50	38	-	-	13	2.01	16
16.08	52.6	52.4	82.4	52.6	0.6	0.2	1.1	silty SAND to sandy SILT	120	4.0	13	13	11	46	37	-	-	18	2.16	16
16.24	53.3	52.9	88.3	53.2	0.7	-1.3	1.3	silty SAND to sandy SILT	120	4.0	13	13	11	46	37	-	-	19	2.20	16
16.40	52.9	52.4	100.8	52.9	0.9	1.1	1.8	silty SAND to sandy SILT	120	4.0	13	13	12	46	37	-	-	22	2.29	16
16.57	65.6	64.8	94.3	65.6	0.7	-1.1	1.1	silty SAND to sandy SILT	120	4.0	16	16	13	53	38	-	-	16	2.10	16
16.73	57.2	56.4	89.4	57.2	0.7	-0.2	1.2	silty SAND to sandy SILT	120	4.0	14	14	12	48	38	-	-	18	2.17	16
16.90	62.3	61.3	77.3	62.4	0.4	0.7	0.6	clean SAND to silty SAND	125	5.0	12	12	12	51	38	-	-	12	1.97	16
17.06	75.5	74.0	81.5	75.5	0.3	0.1	0.4	clean SAND to silty SAND	125	5.0	15	15	13	57	39	-	-	8	1.79	16
17.23	79.8	78.1	85.3	79.8	0.3	0.3	0.4	clean SAND to silty SAND	125	5.0	16	16	14	59	39	-	-	8	1.78	16
17.39	80.4	78.5	83.6	80.5	0.3	1.4	0.3	clean SAND to silty SAND	125	5.0	16	16	14	59	39	-	-	7	1.74	16
17.55	71.3	69.4	77.9	71.3	0.3	2.0	0.4	clean SAND to silty SAND	125	5.0	14	14	13	55	39	-	-	9	1.82	16
17.72	59.1	57.3	67.8	59.1	0.2	0.9	0.4	clean SAND to silty SAND	125	5.0	11	12	11	49	38	-	-	10	1.89	16
17.88	52.8	51.1	63.8	52.8	0.2	1.2	0.4	clean SAND to silty SAND	125	5.0	10	11	10	45	37	-	-	12	1.96	16
18.05	54.8	52.9	66.8	54.8	0.3	1.6	0.5	clean SAND to silty SAND	125	5.0	11	11	10	46	37	-	-	12	1.97	16
18.21	57.6	55.5	103.9	57.6	1.0	0.7	1.8	silty SAND to sandy SILT	120	4.0	14	14	12	48	37	-	-	22	2.28	16
18.37	74.2	71.3	105.6	74.2	1.0	0.8	1.4	silty SAND to sandy SILT	120	4.0	18	19	15	56	39	-	-	16	2.11	16
18.54	183.4	175.9	201.2	183.4	2.7	0.2	1.5	clean SAND to silty SAND	125	5.0	35	37	33	86	43	-	-	9	1.85	16
18.70	111.0	106.2	165.5	111.0	2.6	1.8	2.4	silty SAND to sandy SILT	120	4.0	27	28	22	69	41	-	-	17	2.15	16
18.87	153.0	146.0	167.1	153.0	1.9	1.2	1.2	clean SAND to silty SAND	125	5.0	29	31	27	79	43	-	-	9	1.85	16
19.03	138.5	131.9	154.5	138.5	1.7	-1.7	1.2	clean SAND to silty SAND	125	5.0	26	28	25	76	42	-	-	10	1.88	16
19.19	167.8	159.4	161.8	167.8	1.2	-2.0	0.7	clean SAND to silty SAND	125	5.0	32	34	28	82	43	-	-	6	1.67	16
19.36	150.3	142.4	153.4	150.3	1.3	-0.3	0.9	clean SAND to silty SAND	125	5.0	28	30	26	79	42	-	-	7	1.76	16
19.52	142.3	134.5	134.5	142.3	0.6	-1.9	0.5	clean SAND to silty SAND	125	5.0	27	28	23	77	42	-	-	5	1.61	16
19.69	120.8	113.9	120.1	120.8	0.7	0.2	0.6	clean SAND to silty SAND	125	5.0	23	24	20	71	41	-	-	7	1.72	16
19.85	107.3	100.9	121.8	107.3	1.1	-1.0	1.0	clean SAND to silty SAND	125	5.0	20	21	19	67	41	-	-	11	1.92	16
20.01	113.5	106.5	156.0	113.5	2.3	-0.8	2.0	silty SAND to sandy SILT	120	4.0	27	28	22	69	41	-	-	16	2.11	16
20.18	127.9	119.7	159.1	127.9	2.2	0.9	1.7	silty SAND to sandy SILT	120	4.0	30	32	24	73	41	-	-	14	2.02	16
20.34	160.5	150.0	180.1	160.5	2.4	-2.1	1.5	clean SAND to silty SAND	125	5.0	30	32	28	80	43	-	-	11	1.91	16
20.51	179.0	166.8	204.2	179.0	3.2	-0.1	1.8	clean SAND to silty SAND	125	5.0	33	36	32	84	43	-	-	11	1.94	16
20.67	289.2	268.9	292.0	289.3	4.7	0.9	1.6	clean SAND to silty SAND	125	5.0	54	58	49	95	45	-	-	8	1.77	16
20.83	308.4	286.0	286.0	308.4	3.5	-0.2	1.1	clean SAND to silty SAND	125	5.0	57	62	49	95	46	-	-	5	1.63	16
21.00	306.5	283.6	299.3	306.5	4.5	0.1	1.5	clean SAND to silty SAND	125	5.0	57	61	50	95	46	-	-	7	1.73	16
21.16	344.1	317.7	317.7	344.1	3.9	1.8	1.1	clean SAND to silty SAND	125	5.0	64	69	54	95	46	-	-	5	1.61	16
21.33	384.8	354.5	354.5	384.8	2.5	-2.1	0.6	clean SAND to silty SAND	125	5.0	71	77	56	95	47	-	-	5	1.38	16
21.49	358.2	329.2	331.4	358.5	4.8	12.7	1.4	clean SAND to silty SAND	125	5.0	66	72	57	95	46	-	-	5	1.65	16
21.65	234.5	215.0	254.2	234.5	4.6	2.9	2.0	clean SAND to silty SAND	125	5.0	43	47	41	92	44	-	-	10	1.89	16
21.82	311.8	285.2	285.2	311.9	3.2	5.8	1.0	clean SAND to silty SAND	125	5.0	57	62	49	95	46	-	-	5	1.60	16
21.98	347.0	316.8	316.8	347.5	3.4	25.6	1.0	clean SAND to silty SAND	125	5.0	63	69	53	95	46	-	-	5	1.56	16
22.15	247.0	225.0	230.3	247.1	2.6	5.6	1.1	clean SAND to silty SAND	125	5.0	45	49	39	94	44	-	-	6	1.68	16
22.31	196.5	178.5	178.5	196.5	1.3	2.1	0.7	clean SAND to silty SAND	125	5.0	36	39	31	86	43	-	-	5	1.61	16
22.47	153.9	139.5	139.5	153.8	0.8	-1.5	0.5	clean SAND to silty SAND	125	5.0	28	31	24	78	42	-	-	5	1.61	16
22.64	113.3	102.5	112.3	113.3	0.7	-1.5	0.6	clean SAND to silty SAND	125	5.0	20	23	19	68	40	-	-	8	1.78	16
22.80	107.7	97.2	128.2	107.7	1.5	0.1	1.4	clean SAND to silty SAND	125	5.0	19	22	19	66	40	-	-	13	2.01	16
22.97	136.7	123.1	148.3	136.8	1.7	1.8	1.3	clean SAND to silty SAND	125	5.0	25	27	23	74	41	-	-	11	1.92	16
23.13	195.7	175.9	203.3	195.7	2.9	-0.3	1.5	clean SAND to silty SAND	125	5.0	35	39	33	86	43	-	-	10	1.86	16
23.30	293.9	263.6	277.6	294.0	4.1	0.6	1.4	clean SAND to silty SAND	125	5.0	53	59	47	95	45	-	-	7	1.72	16
23.46	251.1	224.6	255.2	251.1	4.3	3.2	1.7	clean SAND to silty SAND	125	5.0	45	50	42	94	44	-	-	9	1.84	16
23.62	196.1	175.1	239.1	196.0	5.1	-6.8	2.6	silty SAND to sandy SILT	120	4.0	44	49	35	85	43	-	-	14	2.05	16
23.79	128.5	114.5	233.7	128.4	5.4	-7.2	4.2	very stiff fine SOIL	120	2.0	57	64	26	71	41	-	-	24	2.33	30
23.95	113.8	101.2	244.1	115.5	5.7	89.3	5.1	very stiff fine SOIL	120	2.0	51	57	24	67	40	-	-	27	2.42	30
24.12	118.1	104.8	280.4	119.5	7.2	69.4	6.2	very stiff fine SOIL	120	2.0	52	59	25	69	40	-	-	30	2.48	30
24.28	175.1	155.1	309.3	176.2	8.8	55.9	5.1	very stiff fine SOIL	120	2.0	78	88	35	81	42	-	-	23	2.31	30
24.44	191.7	169.4	340.1	191.6	10.3	-5.4	5.4	very stiff fine SOIL	120	2.0	85	96	38	84	43	-	-	23	2.32	30
24.61	184.3	162.6	371.4	184.2	11.9	-6.1	6.5	very stiff fine SOIL	120	2.0	81	92	38	83	43	-	-	26	2.39	30
24.77	194.6	171.3	319.5	194.4	9.4	-7.4	4.8	very stiff fine SOIL	120	2.0	86	97	38	85	43	-	-	22	2.27	30
24.94	173.8	152.8	293.2	173.7	8.1	-6.0	4.7	very stiff fine SOIL	120	2.0	76	87	34	81	42	-	-	22	2.29	30
25.10	170.5	149.6	324.6	170.4	9.6	-9.2	5.7	very stiff fine SOIL	120	2.0	75	85	34	80	42	-	-	25	2.36	30
25.26	213.6	187.0	335.9	213.4	10.2	-8.1	4.8	very stiff fine SOIL	120	2.0	93	100	41	88	43	-	-	21	2.25	30
25.43	181.4	158.5	339.4	181.2	10.4	-9.0	5.8	very stiff fine SOIL	120	2.0	79	91	36	82	43	-	-	25	2.36	30
25.59	160.1	139.6	308.9	159.9	8.9	-9.9	5.6	very stiff fine SOIL	120	2.0	70	80	32	78	42	-	-	25	2.37	30
25.76	133.4	116.1	294.0	133.2	8.0	-9.9	6.1	very stiff fine SOIL	120	2.0	58	67	28	72	41	-	-	29	2.45	30
25.92	175.9	152.8	261.9	175.8	6.7	-3.8	3.9	stiff SAND to clayy SAND	115	1.0	100	100	33	-	-	11.6	9.9	20	2.22	16
26.08	212.0	183.9	271.8	211.8	6.9	-9.5	3.3	stiff SAND to clayy SAND	115	1.0	100	100	38	-	-	14.0	9.9	16	2.11	16
26.25	218.1	188.8	260.8	218.0	6.1	-7.5	2.8	silty SAND to sandy SILT	120	4.0	47	55	38	88	43	-	-	15	2.06	16
26.41	213.3	184.3	301.1	213.1	8.5	-8.0	4.0	stiff SAND to clayy SAND	115	1.0	100	100	39	-	-	14.0	9.9	19	2.19	16
26.58	203.5	175.5	311.5	203.3	9.1	-8.6	4.5	very stiff fine SOIL	120	2.0	88	100	38	86	43	-	-	20	2.24	30
26.74	173.2	149.1	360.5	173.0	11.5	-6.5	6.7	very stiff fine SOIL	120	2.0	75	87	35	80	42	-	-	27	2.43	30
26.90	201.6	173.3	346.8	201.4	11.0	-7.7	5.5	very stiff fine SOIL	120	2.0	87	100	39	85	43	-	-	23	2.32	30
27.07	200.9	172.4	338.6	200.8	10.6	-7.3	5.3	very stiff fine SOIL	120	2.0	86	100	38	85	43	-	-	23	2.30	30
27.23	166.8	142.8	284.5	166.6	7.9	-8.6	4.8	very stiff fine SOIL	120	2.0	71	83	32	79	42	-	-	23	2.31	30
27.40	125.2	107.0	246.9	125.0	6.1	-8.3	4.9	very stiff fine SOIL	120	2.0	53	63	25	69	40	-	-	26	2.40	30
27.56	101.7	86.8	261.7	101.6	6.4	-8.7	6.4	very stiff fine SOIL	120	2.0	43	51	22	62	39	-	-	33	2.55	30
27.72	113.1	96.3	271.7	112.9	7.0	-9.1	6.3	very stiff fine SOIL	120	2.0	48	57	24	66	40	-	-	31	2.51	30
27.89	98.6	83.8	257.2	98.4	6.2	-9.8	6.4	very stiff fine SOIL	120	2.0	42	49	21	61	39	-	-	33	2.56	30
28.05	73.9	56.4	-	73.8	5.2	-8.8	7.2	silty CLAY to CLAY	115	1.5	38	49	15	-	-	5.1	9.9	41	2.70	15
28.22	80.2	61.0	-	80.1	5.1	-6.0	6.5	silty CLAY to CLAY	115	1.5	41	53	16	-	-	5.6	9.9	38	2.65	15
28.38	105.0	88.8	245.8	104.9	5.9	-6.9	5.7	very stiff fine SOIL	120	2.0	44	53	22	63	39	-	-	31	2.50	30
28.54	89.5	67.6	-	89.3	6.7	-8.4	7.6	very stiff fine SOIL	120	2.0	34	45	18	54	38	-	-	39	2.67	30
28.71	87.9	66.2	-	87.7	6.6	-9.0	7.6	very stiff fine SOIL	120	2.0	33	44	18	53	38	-	-	39	2.68	30
28.87	80.0	60.0	-	79.8	5.9	-9.3	7.6	silty CLAY to CLAY	115	1.5	40	53	16	-	-	5.6	9.9	41	2.70	15
29.04	73.1	54.6	-	72.9	5.3	-10.0	7.5	silty CLAY to CLAY	115	1.5	36	49	15	-	-	5.1	9.9	42	2.72	15
29.20	61.8	46.1	-	61.6	4.9	-10.7	8.1	silty CLAY to CLAY	115	1.5	31	41	13	-	-	4.3	9.9	46	2.80	15
29.36	55.2	41.0	-	55.0	4.0	-9.5	7.5	silty CLAY to CLAY	115	1.5	27	37	12	-	-	3.8	9.9	47	2.81	15
29.53	55.0	40.7	-	54.8	3.7	-9.6	6.9	silty CLAY to CLAY	115	1.5	27	37	11	-	-	3.8	9.9	45	2.78	15
29.69	57.5	42.4	-	57.3	3.6	-9.4	6.5	silty CLAY to CLAY	115	1.5	28	38	12	-	-	4.0	9.9	43	2.76	15
29.86	51.1	37.6	-	50.9	3.1	-9.6	6.4	silty CLAY to CLAY	115	1.5	25	34	11	-	-	3.5	9.9	45	2.78	15
30.02	48.5	35.6	-	48.3	3.0	-9.4	6.4	silty CLAY to CLAY	115	1.5	24	32	10	-	-	3.3	9.9	46	2.80	15
30.19	37.4	27.3	-	37.2	3.0	-8.1	8.4	silty CLAY to CLAY	115	1.5	18	25	9	-	-	2.5	8.6	56	2.97	15
30.35	27.9	20.3	-	27.7	2.8	-7.5	9.9	silty CLAY to CLAY	115	1.5	14	19	7	-	-	1.9	6.3	67	3.12	15
30.51	31.4	22.8	-	31.3	2.2	-7.8	7.5	silty CLAY to CLAY	115	1.5	15	21	7	-	-	2.1	7.1	58	2.99	15
30.68	48.4	35.0	-	48.2	3.6	-7.2	7.7	silty CLAY to CLAY	115	1.5	23	32	10	-	-	3.3	9.9	50	2.86	15
30.84	66.3	47.9	-	66.2	4.1	-6.8	6.4	silty CLAY to CLAY	115	1.5	32	44	13	-	-	4.6	9.9	41	2.71	15

* Indicates the parameter was calculated using the normalized point stress.
The parameters listed above were determined using empirical correlations.
A Professional Engineer must determine their suitability for analysis and design.

Project ID: Ninyo & Moore
Data File: SDF(599).cpt
CPT Date: 3/10/2018 8:13:47 AM
GW During Test: 16 ft

Sounding ID: CPT-01
Project No: 403215001
Cone/Rig: DDG1281

. Depth ft	. qc PS tsf	* qc1n PS -	. q1ncs PS -	* qt PS tsf	. Slv Stss tsf	. pore prss (psi)	. Frct Rato %	* Material Behavior Description	. Unit Wght pcf	. Qc to N	* SPT R-N1 60%	. SPT R-N 60%	* SPT IcN1 60%	* Rel Den %	* Ftn Ang deg	. Und Shr tsf	. OCR - -	* Fin Ic %	* Ic SBT Indx	* Nk - -
31.01	78.8	56.7	-	78.6	4.6	-7.9	5.9	silty CLAY to CLAY	115	1.5	38	53	15	-	-	5.5	9.9	37	2.64	15
31.17	71.0	51.0	-	70.9	4.4	-7.1	6.4	silty CLAY to CLAY	115	1.5	34	47	14	-	-	4.9	9.9	40	2.69	15
31.33	71.0	50.8	-	70.9	4.7	-6.3	6.9	silty CLAY to CLAY	115	1.5	34	47	14	-	-	4.9	9.9	41	2.72	15
31.50	80.2	57.2	-	80.1	5.1	-6.8	6.5	silty CLAY to CLAY	115	1.5	38	53	15	-	-	5.6	9.9	39	2.67	15
31.66	91.1	74.6	228.7	91.0	5.2	-7.7	5.9	very stiff fine SOIL	120	2.0	37	46	19	57	38	-	-	33	2.56	30
31.83	81.8	57.9	-	81.6	4.6	-8.1	5.7	clayy SILT to silty CLAY	115	2.0	29	41	15	-	-	5.7	9.9	36	2.62	15
31.99	71.2	50.3	-	71.1	4.0	-7.4	5.7	silty CLAY to CLAY	115	1.5	34	47	13	-	-	4.9	9.9	38	2.66	15
32.15	90.6	73.9	225.7	90.5	5.1	-5.7	5.8	very stiff fine SOIL	120	2.0	37	45	18	57	38	-	-	33	2.55	30
32.32	115.4	93.9	242.5	115.3	6.1	-7.5	5.3	very stiff fine SOIL	120	2.0	47	58	22	65	39	-	-	29	2.46	30
32.48	107.3	87.2	246.6	107.1	6.1	-7.8	5.8	very stiff fine SOIL	120	2.0	44	54	21	62	39	-	-	31	2.51	30
32.65	104.9	85.1	234.2	104.7	5.6	-8.1	5.5	very stiff fine SOIL	120	2.0	43	52	21	62	39	-	-	31	2.50	30
32.81	100.5	81.4	224.9	100.4	5.2	-8.1	5.3	very stiff fine SOIL	120	2.0	41	50	20	60	39	-	-	31	2.50	30
32.97	88.9	71.9	206.6	88.7	4.4	-7.2	5.1	clayy SILT to silty CLAY	115	2.0	36	44	18	-	-	6.2	9.9	32	2.52	15
33.14	93.0	75.0	205.6	92.8	4.5	-8.3	4.9	clayy SILT to silty CLAY	115	2.0	38	46	18	-	-	6.5	9.9	30	2.49	15
33.30	83.4	67.3	172.6	83.3	3.2	-8.8	4.0	clayy SILT to silty CLAY	115	2.0	34	42	16	-	-	5.8	9.9	29	2.46	15
33.47	80.0	64.4	179.9	79.8	3.4	-8.2	4.4	clayy SILT to silty CLAY	115	2.0	32	40	16	-	-	5.5	9.9	31	2.51	15
33.63	112.1	90.1	237.0	111.9	5.9	-9.7	5.3	very stiff fine SOIL	120	2.0	45	56	22	64	39	-	-	29	2.47	30
33.79	108.6	87.2	252.3	108.4	6.5	-9.0	6.1	very stiff fine SOIL	120	2.0	44	54	21	62	39	-	-	32	2.52	30
33.96	98.8	67.2	-	98.6	6.7	-7.7	6.9	very stiff fine SOIL	120	2.0	34	49	18	54	38	-	-	37	2.64	30
34.12	125.6	100.5	271.6	125.5	7.5	-8.0	6.1	very stiff fine SOIL	120	2.0	50	63	24	67	40	-	-	30	2.49	30
34.29	160.5	128.1	289.9	160.3	8.6	-8.6	5.5	very stiff fine SOIL	120	2.0	64	80	30	75	41	-	-	26	2.39	30
34.45	182.8	145.8	309.9	182.6	9.8	-9.4	5.4	very stiff fine SOIL	120	2.0	73	91	33	79	42	-	-	24	2.35	30
34.61	132.6	105.5	222.1	132.4	5.5	-9.4	4.2	very stiff fine SOIL	120	2.0	53	66	24	69	40	-	-	24	2.35	30
34.78	225.8	179.5	237.4	225.6	5.4	-8.1	2.4	silty SAND to sandy SILT	120	4.0	45	56	35	86	43	-	-	13	2.02	16
34.94	404.8	321.2	362.2	404.7	8.7	-5.0	2.1	clean SAND to silty SAND	125	5.0	64	81	59	95	45	-	-	9	1.83	16
35.11	452.3	358.3	360.0	452.2	6.4	-3.3	1.4	clean SAND to silty SAND	125	5.0	72	90	62	95	46	-	-	5	1.65	16
35.27	438.6	346.8	367.1	438.5	7.6	-3.8	1.7	clean SAND to silty SAND	125	5.0	69	88	62	95	46	-	-	7	1.73	16
35.43	421.6	332.8	349.1	421.6	6.8	-0.8	1.6	clean SAND to silty SAND	125	5.0	67	84	59	95	46	-	-	6	1.72	16
35.60	290.1	228.6	265.4	290.0	5.5	-3.2	1.9	clean SAND to silty SAND	125	5.0	46	58	43	94	44	-	-	10	1.87	16
35.76	174.4	137.2	237.6	174.3	6.2	-4.6	3.6	stiff SAND to clayy SAND	115	1.0	100	100	30	-	-	11.5	9.9	20	2.23	16
35.93	91.3	59.7	-	91.2	6.6	-2.3	7.4	silty CLAY to CLAY	115	1.5	40	61	16	-	-	6.3	9.9	40	2.70	15
36.09	53.1	34.7	-	53.1	5.7	-0.3	9.9	silty CLAY to CLAY	115	1.5	23	35	11	-	-	3.6	9.9	55	2.95	15
36.26	84.6	55.1	-	84.7	6.2	1.1	7.5	silty CLAY to CLAY	115	1.5	37	56	15	-	-	5.9	9.9	42	2.72	15
36.42	137.0	107.2	268.6	137.1	7.6	2.0	5.6	very stiff fine SOIL	120	2.0	54	69	25	69	40	-	-	28	2.44	30
36.58	105.4	68.2	-	105.3	6.9	-5.2	6.7	very stiff fine SOIL	120	2.0	34	53	18	54	38	-	-	37	2.63	30
36.75	90.9	58.6	-	91.2	6.2	17.1	7.0	silty CLAY to CLAY	115	1.5	39	61	16	-	-	6.3	9.9	39	2.68	15
36.91	175.5	136.7	225.6	176.2	5.7	37.0	3.3	silty SAND to sandy SILT	120	4.0	34	44	29	77	41	-	-	19	2.19	16
37.08	238.9	185.8	242.6	239.0	5.7	1.2	2.4	silty SAND to sandy SILT	120	4.0	46	60	37	87	43	-	-	13	2.00	16

* Indicates the parameter was calculated using the normalized point stress.
The parameters listed above were determined using empirical correlations.
A Professional Engineer must determine their suitability for analysis and design.

Middle Earth Geo Testing

Project ID: Ninyo & Moore
Data File: SDF(601).cpt
CPT Date: 3/10/2018 11:14:05 AM
GW During Test: 16 ft

Sounding ID: CPT-02
Project No: 403215001
Cone/Rig: DDG1281

. Depth ft	. qc PS tsf	* qc1n PS -	. q1ncs PS -	* qt PS tsf	. Slv Stss tsf	. pore prss tsf (psi)	. Frct Rato %	* Material Behavior Description	. Unit Wght pcf	. Qc to N	* SPT R-N1 60%	. SPT R-N 60%	* SPT IcN1 60%	* Rel Den %	* Ftn Ang deg	. Und Shr tsf	. OCR - -	* Fin Ic %	* Ic SBT Indx	* Nk - -
0.33	1.1	0.0	-	1.1	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	1	1	1	-	-	0.1	9.9	95	3.48	15
0.49	1.1	0.0	-	1.1	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	0	1	0	-	-	0.1	9.9	95	3.48	15
0.66	1.1	0.0	-	1.1	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	0	1	0	-	-	0.1	9.3	95	3.48	15
0.82	1.1	0.0	-	1.1	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	1	1	1	-	-	0.1	7.4	95	3.48	15
0.98	1.1	0.0	-	1.1	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	1	1	1	-	-	0.1	6.1	95	3.48	15
1.15	1.1	0.0	-	1.1	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	1	1	1	-	-	0.1	5.2	95	3.48	15
1.31	1.1	0.4	-	1.1	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	0	1	0	-	-	0.1	4.5	95	4.10	15
1.48	1.1	0.5	-	1.1	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	0	1	0	-	-	0.1	3.9	95	3.95	15
1.64	1.1	0.5	-	1.1	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	0	1	0	-	-	0.1	3.5	95	3.97	15
1.80	1.1	0.8	-	1.1	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	0	1	0	-	-	0.1	3.2	95	3.71	15
1.97	1.1	0.8	-	1.1	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	0	1	0	-	-	0.1	2.9	95	3.72	15
2.13	1.1	0.7	-	1.1	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	0	1	0	-	-	0.1	2.6	95	3.82	15
2.30	1.1	0.6	-	1.1	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	0	1	0	-	-	0.1	2.4	95	3.94	15
2.46	1.1	0.3	-	1.1	0.0	0.0	1.2	sensitive fine SOIL	115	2.0	0	1	1	-	-	0.1	2.2	95	5.04	15
2.62	1.1	0.2	-	1.1	0.0	0.0	0.1	sensitive fine SOIL	115	2.0	0	1	0	-	-	0.1	2.1	95	3.48	15
2.79	3.6	5.7	-	3.6	0.0	0.0	0.8	clayy SILT to silty CLAY	115	2.0	3	2	2	-	-	0.2	7.0	55	2.95	15
2.95	3.1	5.0	-	3.1	0.4	0.0	9.9	Organic SOILS - Peats	100	1.0	5	3	2	-	-	0.3	5.8	95	3.57	10
3.12	59.4	95.3	136.1	59.4	1.0	-0.8	1.7	silty SAND to sandy SILT	120	4.0	24	15	19	65	46	-	-	15	2.08	16
3.28	42.5	68.1	119.4	42.5	0.8	-1.0	2.0	silty SAND to sandy SILT	120	4.0	17	11	15	54	45	-	-	20	2.23	16
3.45	39.8	63.8	94.4	39.7	0.5	-0.9	1.2	silty SAND to sandy SILT	120	4.0	16	10	13	52	44	-	-	16	2.11	16
3.61	19.6	31.4	82.1	19.6	0.4	0.0	1.8	silty SAND to sandy SILT	120	4.0	8	5	8	29	40	-	-	29	2.47	16
3.77	22.5	36.2	87.9	22.5	0.4	0.2	1.9	silty SAND to sandy SILT	120	4.0	9	6	8	33	41	-	-	28	2.43	16
3.94	24.6	39.4	93.5	24.6	0.5	-0.3	2.0	silty SAND to sandy SILT	120	4.0	10	6	9	36	41	-	-	27	2.41	16
4.10	35.7	57.3	93.7	35.7	0.5	-0.2	1.4	silty SAND to sandy SILT	120	4.0	14	9	12	49	43	-	-	19	2.19	16
4.27	22.1	35.4	85.5	22.1	0.4	-0.2	1.8	silty SAND to sandy SILT	120	4.0	9	6	8	33	40	-	-	27	2.42	16
4.43	19.9	32.0	66.2	19.9	0.2	-0.1	1.1	silty SAND to sandy SILT	120	4.0	8	5	7	29	39	-	-	24	2.34	16
4.59	19.2	30.8	59.0	19.2	0.2	-0.1	0.8	silty SAND to sandy SILT	120	4.0	8	5	7	28	39	-	-	22	2.29	16
4.76	18.7	30.1	57.8	18.7	0.1	0.0	0.8	silty SAND to sandy SILT	120	4.0	8	5	7	27	38	-	-	22	2.29	16
4.92	20.6	33.1	49.7	20.6	0.1	-0.1	0.4	silty SAND to sandy SILT	120	4.0	8	5	7	31	39	-	-	17	2.13	16
5.09	21.7	34.8	55.5	21.7	0.1	-0.1	0.6	silty SAND to sandy SILT	120	4.0	9	5	7	32	39	-	-	18	2.17	16
5.25	24.5	39.3	77.5	24.5	0.3	-0.2	1.3	silty SAND to sandy SILT	120	4.0	10	6	9	36	39	-	-	23	2.31	16
5.41	25.1	40.3	106.8	25.1	0.6	-0.2	2.5	clayy SILT to silty CLAY	115	2.0	20	13	10	-	-	1.8	9.9	30	2.48	15
5.58	31.7	50.8	97.3	31.7	0.5	-0.4	1.7	silty SAND to sandy SILT	120	4.0	13	8	11	45	40	-	-	22	2.29	16
5.74	24.7	39.6	93.2	24.7	0.5	-0.3	1.9	silty SAND to sandy SILT	120	4.0	10	6	9	36	39	-	-	27	2.41	16
5.91	23.9	38.4	67.0	23.9	0.2	-0.2	0.9	silty SAND to sandy SILT	120	4.0	10	6	8	35	38	-	-	20	2.23	16
6.07	24.6	39.5	65.2	24.6	0.2	-0.2	0.8	silty SAND to sandy SILT	120	4.0	10	6	8	36	38	-	-	19	2.19	16
6.23	24.3	38.9	63.3	24.3	0.2	-0.2	0.8	silty SAND to sandy SILT	120	4.0	10	6	8	36	38	-	-	18	2.18	16
6.40	23.4	37.2	61.7	23.4	0.2	-0.2	0.7	silty SAND to sandy SILT	120	4.0	9	6	8	34	38	-	-	19	2.20	16
6.56	23.9	37.5	62.7	23.9	0.2	-0.2	0.8	silty SAND to sandy SILT	120	4.0	9	6	8	35	38	-	-	19	2.20	16
6.73	26.1	40.4	65.1	26.1	0.2	-0.2	0.8	silty SAND to sandy SILT	120	4.0	10	7	9	37	38	-	-	18	2.18	16
6.89	27.4	41.9	66.2	27.4	0.2	-0.2	0.8	silty SAND to sandy SILT	120	4.0	10	7	9	38	38	-	-	18	2.16	16
7.05	26.8	40.5	66.8	26.8	0.2	-0.2	0.8	silty SAND to sandy SILT	120	4.0	10	7	9	37	38	-	-	19	2.19	16
7.22	23.4	34.9	68.9	23.4	0.3	-0.3	1.1	silty SAND to sandy SILT	120	4.0	9	6	8	32	37	-	-	23	2.31	16
7.38	25.6	37.8	72.4	25.6	0.3	-0.2	1.1	silty SAND to sandy SILT	120	4.0	9	6	8	35	38	-	-	22	2.29	16
7.55	29.9	43.6	73.7	29.9	0.3	-0.2	1.0	silty SAND to sandy SILT	120	4.0	11	7	9	40	38	-	-	19	2.21	16
7.71	36.3	52.3	73.1	36.3	0.3	-0.3	0.7	clean SAND to silty SAND	125	5.0	10	7	11	46	39	-	-	15	2.07	16
7.87	34.1	48.6	66.5	34.0	0.2	-0.3	0.6	clean SAND to silty SAND	125	5.0	10	7	10	43	39	-	-	14	2.05	16
8.04	32.0	45.1	64.8	32.0	0.2	-0.3	0.6	silty SAND to sandy SILT	120	4.0	11	8	9	41	38	-	-	16	2.09	16
8.20	29.6	41.3	65.2	29.6	0.2	-0.2	0.8	silty SAND to sandy SILT	120	4.0	10	7	9	38	38	-	-	18	2.16	16
8.37	28.6	39.6	66.8	28.6	0.2	-0.2	0.9	silty SAND to sandy SILT	120	4.0	10	7	8	36	37	-	-	19	2.21	16
8.53	30.2	41.4	62.0	30.2	0.2	-0.2	0.6	silty SAND to sandy SILT	120	4.0	10	8	9	38	38	-	-	17	2.12	16
8.69	33.4	45.3	69.2	33.4	0.3	-0.3	0.8	silty SAND to sandy SILT	120	4.0	11	8	9	41	38	-	-	17	2.14	16
8.86	47.4	63.6	86.0	47.4	0.4	-0.4	0.9	clean SAND to silty SAND	125	5.0	13	9	13	52	40	-	-	14	2.04	16
9.02	56.8	75.6	104.1	56.8	0.7	-0.5	1.2	silty SAND to sandy SILT	120	4.0	19	14	15	58	41	-	-	14	2.05	16
9.19	61.6	81.2	107.6	61.6	0.7	-0.6	1.1	clean SAND to silty SAND	125	5.0	16	12	16	60	41	-	-	13	2.02	16
9.35	42.6	55.7	89.7	42.6	0.5	-0.4	1.3	silty SAND to sandy SILT	120	4.0	14	11	12	48	39	-	-	18	2.18	16
9.51	31.4	40.7	67.7	31.4	0.3	-0.3	0.9	silty SAND to sandy SILT	120	4.0	10	8	9	37	37	-	-	19	2.20	16
9.68	25.6	32.9	67.4	25.6	0.3	-0.3	1.1	silty SAND to sandy SILT	120	4.0	8	6	7	30	36	-	-	24	2.33	16
9.84	21.5	27.4	81.8	21.5	0.4	-0.2	2.0	clayy SILT to silty CLAY	115	2.0	14	11	7	-	-	1.5	9.9	33	2.54	15
10.01	19.8	28.1	87.1	19.8	0.4	-0.2	2.2	clayy SILT to silty CLAY	115	2.0	14	10	7	-	-	1.4	9.9	33	2.56	15
10.17	25.0	31.3	73.3	25.0	0.3	0.5	1.4	silty SAND to sandy SILT	120	4.0	8	6	7	29	35	-	-	27	2.41	16
10.34	32.3	40.2	103.3	32.3	0.7	-0.1	2.4	silty SAND to sandy SILT	120	4.0	10	8	10	37	37	-	-	29	2.46	16
10.50	28.4	39.0	124.4	28.4	0.9	0.0	3.4	clayy SILT to silty CLAY	115	2.0	19	14	10	-	-	2.0	9.9	34	2.58	15
10.66	19.5	31.0	-	19.5	0.8	-0.1	4.3	silty CLAY to CLAY	115	1.5	21	13	8	-	-	1.3	9.9	42	2.72	15
10.83	18.3	22.2	71.9	18.2	0.3	-1.5	1.7	clayy SILT to silty CLAY	115	2.0	11	9	6	-	-	1.2	9.1	35	2.59	15
10.99	18.1	21.8	60.5	18.1	0.2	-0.1	1.2	silty SAND to sandy SILT	120	4.0	5	5	5	17	33	-	-	31	2.50	16
11.16	19.5	23.3	56.1	19.5	0.2	-0.1	0.9	silty SAND to sandy SILT	120	4.0	6	5	5	19	33	-	-	27	2.42	16
11.32	21.5	25.5	55.3	21.5	0.2	-0.1	0.8	silty SAND to sandy SILT	120	4.0	6	5	6	22	34	-	-	25	2.36	16
11.48	22.1	26.1	55.1	22.1	0.2	-0.2	0.8	silty SAND to sandy SILT	120	4.0	7	6	6	23	34	-	-	24	2.35	16
11.65	21.0	24.6	52.5	21.0	0.1	-0.2	0.7	silty SAND to sandy SILT	120	4.0	6	5	6	21	33	-	-	25	2.35	16
11.81	16.8	19.5	46.7	16.8	0.1	-0.2	0.6	silty SAND to sandy SILT	120	4.0	5	4	5	13	32	-	-	27	2.42	16
11.98	12.8	14.8	39.5	12.8	0.1	-0.1	0.4	silty SAND to sandy SILT	120	4.0	4	3	4	5	30	-	-	30	2.48	16
12.14	10.0	12.4	-	10.0	0.1	-0.1	0.7	clayy SILT to silty CLAY	115	2.0	6	5	3	-	-	0.7	4.3	37	2.63	15
12.30	9.9	13.6	-	9.9	0.1	-0.2	1.4	clayy SILT to silty CLAY	115	2.0	7	5	4	-	-	0.6	4.2	43	2.74	15
12.47	10.3	14.0	-	10.3	0.3	-0.2	2.9	silty CLAY to CLAY	115	1.5	9	7	4	-	-	0.7	4.3	52	2.90	15
12.63	14.6	19.6	-	14.6	0.7	-0.3	5.3	silty CLAY to CLAY	115	1.5	13	10	6	-	-	1.0	6.1	54	2.94	15
12.80	19.2	25.4	-	19.2	0.6	-0.2	3.5	clayy SILT to silty CLAY	115	2.0	13	10	7	-	-	1.3	8.1	42	2.73	15
12.96	6.8	8.9	-	6.8	0.5	-0.2	8.5	silty CLAY to CLAY	115	1.5	6	5	4	-	-	0.4	2.6	86	3.35	15
13.12	5.7	7.3	-	5.6	0.1	-0.2	2.9	silty CLAY to CLAY	115	1.5	5	4	3	-	-	0.3	2.1	70	3.16	15
13.29	4.1	5.2	-	4.1	0.1	-0.2	2.2	silty CLAY to CLAY	115	1.5	3	3	2	-	-	0.2	1.4	77	3.25	15
13.45	2.9	3.6	-	2.9	0.1	-0.2	4.3	silty CLAY to CLAY	115	1.5	2	2	2	-	-	0.1	0.9	95	3.57	15
13.62	3.4	4.3	-	3.4	0.2	-0.2	6.7	Organic SOILS - Peats	100	1.0	4	3	2	-	-	0.3	1.1	95	3.59	10
13.78	18.6	22.9	-	18.6	0.4	-0.3	2.1	clayy SILT to silty CLAY	115	2.0	11	9	6	-	-	1.3	7.2	37	2.63	15
13.94	25.5	31.0	-	25.5	0.7	-0.3	2.9	clayy SILT to silty CLAY	115	2.0	16	13	8	-	-	1.7	9.9	36	2.60	15
14.11	45.9	48.9	123.5	45.9	1.3	-0.2	2.8	clayy SILT to silty CLAY	115	2.0	24	23	12	-	-	3.2	9.9	28	2.45	15
14.27	48.7	51.7	156.5	48.7	2.0	-0.6	4.2	clayy SILT to silty CLAY	115	2.0	26	24	13	-	-	3.4	9.9	33	2.55	15
14.44	60.7	64.0	184.1	60.7	2.8	-0.2	4.7	clayy SILT to silty CLAY	115	2.0	32	30	16	-	-	4.2	9.9	32	2.52	15
14.60	75.6	79.3	197.1	75.6	3.3	-2.1	4.4	clayy SILT to silty CLAY	115	2.0	40	38	19	-	-	5.3	9.9	28	2.44	15
14.76	103.3	107.7	214.1	103.3	4.0	-2.2	3.9	stiff SAND to clayy SAND	115	1.0	100	100	24	-	-	6.8	9.9	23	2.31	16
14.93	138.7	143.8	237.0	138.7	4.7	-1.1	3.4	silty SAND to sandy SILT	120	4.0	36	35	30	79	43	-	-	19	2.19	16
15.09	143.4	147.8	267.8	143.4	5.9	-1.5	4.2	very stiff fine SOIL	120	2.0	74	72	32	80	43	-	-	21	2.25	30
15.26	145.0	148.7	282.8	145.0	6.6	0.0	4.6	very stiff fine SOIL	120	2.0	74	73	33	80	43	-	-	22	2.29	30
15.42	181.2	184.8	306.2	181.2	7.4	-1.2	4.1	stiff SAND to clayy SAND	115	1.0	100	100	39	-	-	11.9	9.9	19	2.20	16

* Indicates the parameter was calculated using the normalized point stress.
The parameters listed above were determined using empirical correlations.
A Professional Engineer must determine their suitability for analysis and design.

Middle Earth Geo Testing

Slatter-2035 N Pacific Ave-Geo

Project ID: Ninyo & Moore
Data File: SDF(601).cpt
CPT Date: 3/10/2018 11:14:05 AM
GW During Test: 16 ft

Sounding ID: CPT-02
Project No: 403215001
Cone/Rig: DDG1281

Depth ft	qc PS tsf	* qc1n PS -	q1ncs PS -	* qt PS tsf	Slv Stss tsf	pore prss (psi)	Frct Rato %	* Material Behavior Description	Unit Wght pcf	Qc to N	* SPT R-N1 60%	SPT R-N 60%	* SPT IcN1 60%	* Rel Den %	* Ftn Ang deg	Und Shr tsf	OCR -	* Fin Ic %	* Ic SBT Indx	* Nk -
15.58	180.7	183.3	300.9	180.7	7.2	0.4	4.0	stiff SAND to clayy SAND	115	1.0	100	100	39	-	-	11.9	9.9	19	2.19	16
15.75	81.3	84.3	-	81.1	6.1	-7.2	7.6	very stiff fine SOIL	120	2.0	42	41	22	61	40	-	-	36	2.61	30
15.91	71.8	72.1	177.8	71.6	2.8	-6.6	4.0	clayy SILT to silty CLAY	115	2.0	36	36	17	-	-	5.0	9.9	28	2.44	15
16.08	82.0	82.1	190.4	81.8	3.3	-5.5	4.0	clayy SILT to silty CLAY	115	2.0	41	41	19	-	-	5.7	9.9	26	2.40	15
16.24	88.6	88.5	213.7	88.5	4.0	-2.0	4.6	clayy SILT to silty CLAY	115	2.0	44	44	21	-	-	6.2	9.9	27	2.42	15
16.40	112.5	112.1	247.6	112.4	5.3	-2.3	4.8	very stiff fine SOIL	120	2.0	56	56	26	71	41	-	-	25	2.37	30
16.57	105.4	104.8	254.6	105.3	5.5	-2.1	5.3	very stiff fine SOIL	120	2.0	52	53	25	69	41	-	-	28	2.43	30
16.73	138.3	137.2	277.7	138.2	6.6	-2.6	4.8	very stiff fine SOIL	120	2.0	69	69	31	77	42	-	-	23	2.32	30
16.90	132.9	131.6	294.5	132.9	7.2	-3.1	5.5	very stiff fine SOIL	120	2.0	66	66	30	76	42	-	-	26	2.38	30
17.06	147.3	145.4	319.1	147.2	8.3	-0.4	5.7	very stiff fine SOIL	120	2.0	73	74	33	79	43	-	-	25	2.37	30
17.23	203.8	200.8	363.2	203.8	10.3	-0.1	5.1	very stiff fine SOIL	120	2.0	100	100	44	90	44	-	-	21	2.25	30
17.39	218.2	214.4	404.0	218.2	12.3	2.6	5.7	very stiff fine SOIL	120	2.0	100	100	47	92	45	-	-	22	2.28	30

* Indicates the parameter was calculated using the normalized point stress.
The parameters listed above were determined using empirical correlations.
A Professional Engineer must determine their suitability for analysis and design.

Middle Earth Geo Testing

APPENDIX C

Laboratory Testing

APPENDIX C

LABORATORY TESTING

Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488-00. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

Moisture Content

The moisture content of samples obtained from the exploratory borings was evaluated in accordance with ASTM D 2216. The test results are presented on the boring logs in Appendix A.

In Place Density Tests

The dry density of relatively undisturbed samples obtained from the exploratory borings was evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

Gradation Analysis

A gradation analysis test was performed on a selected representative soil sample in general accordance with ASTM D 422. The grain size distribution curves are shown on Figure C-1. The test results were utilized in evaluating the soil classification in accordance with the Unified Soil Classification System (USCS).

Atterberg Limits

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

Unconfined Compression Tests

An unconfined compression test was performed on a relatively undisturbed sample in general accordance with ASTM D 2166. The test results are shown on Figure C-3.

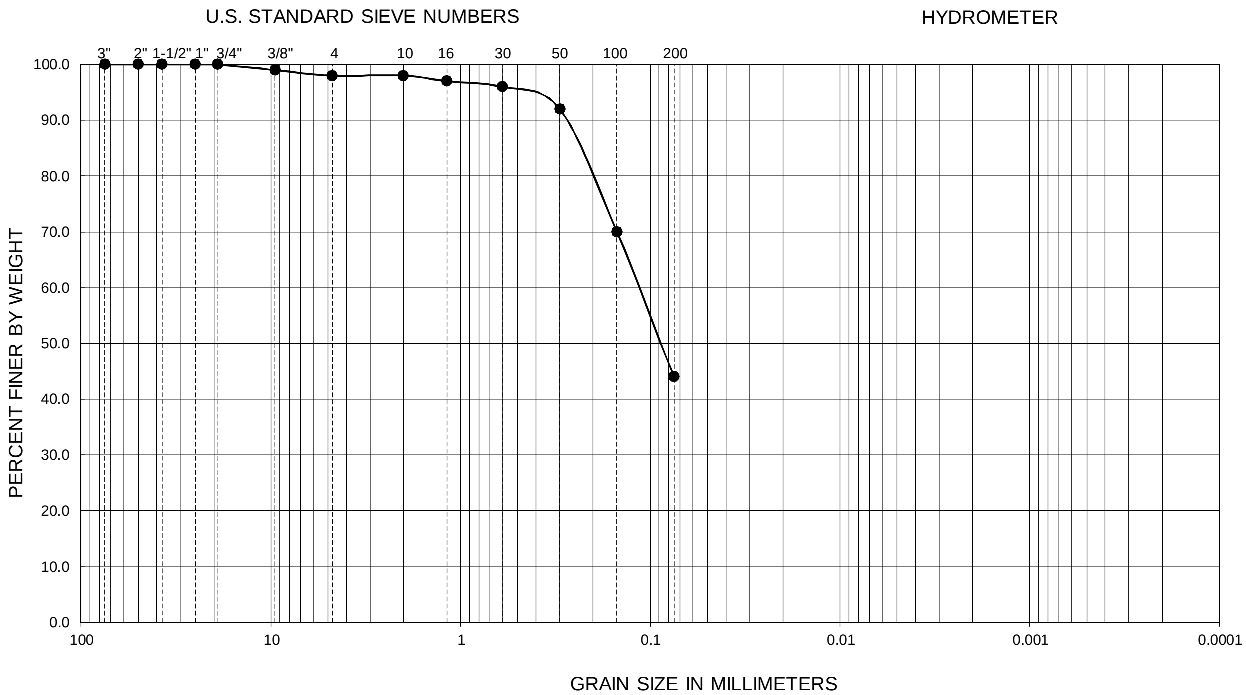
Direct Shear Tests

Direct shear tests were performed on relatively undisturbed samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of selected materials. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figures C-4 and C-5.

Soil Corrosivity Tests

Soil pH, and resistivity tests were performed on a representative sample in general accordance with California Test (CT) 643. The soluble sulfate and chloride contents of the selected sample were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure C-6.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-3	5.0-5.5	--	--	--	--	--	0.13	--	--	44	SC

PERFORMED IN ACCORDANCE WITH ASTM D 422

FIGURE C-1

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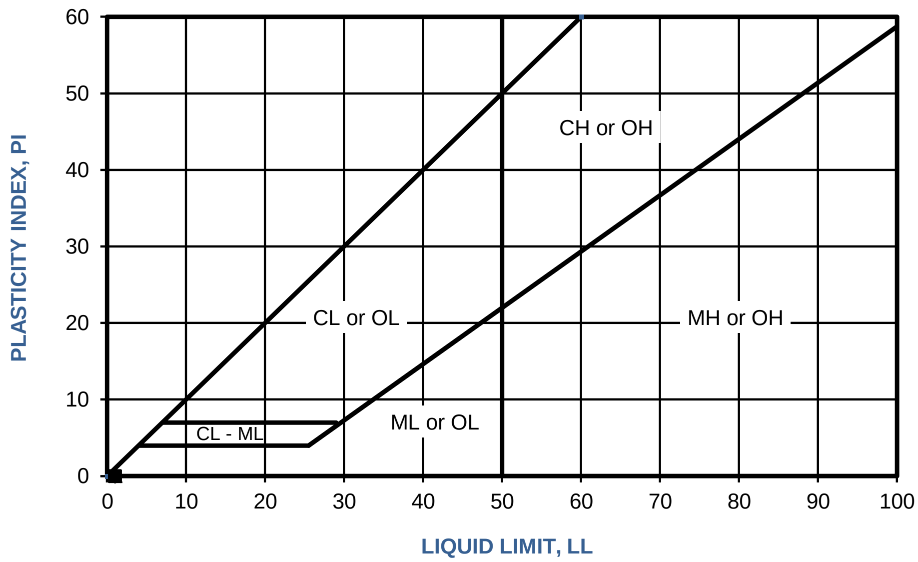
GRADATION TEST RESULTS
2035 NORTH PACIFIC AVENUE
SANTA CRUZ, CALIFORNIA

403215001 | 5/18

SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-1	8.5-9.0	N/A	N/A	N/A	NP	N/A
■	B-3	13.5-14.0	N/A	N/A	N/A	NP	N/A

NP - INDICATES NON-PLASTIC



PERFORMED IN ACCORDANCE WITH ASTM D 4318

FIGURE C-2

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ATTERBERG LIMITS TEST RESULTS
2035 NORTH PACIFIC AVENUE
SANTA CRUZ CALIFORNIA

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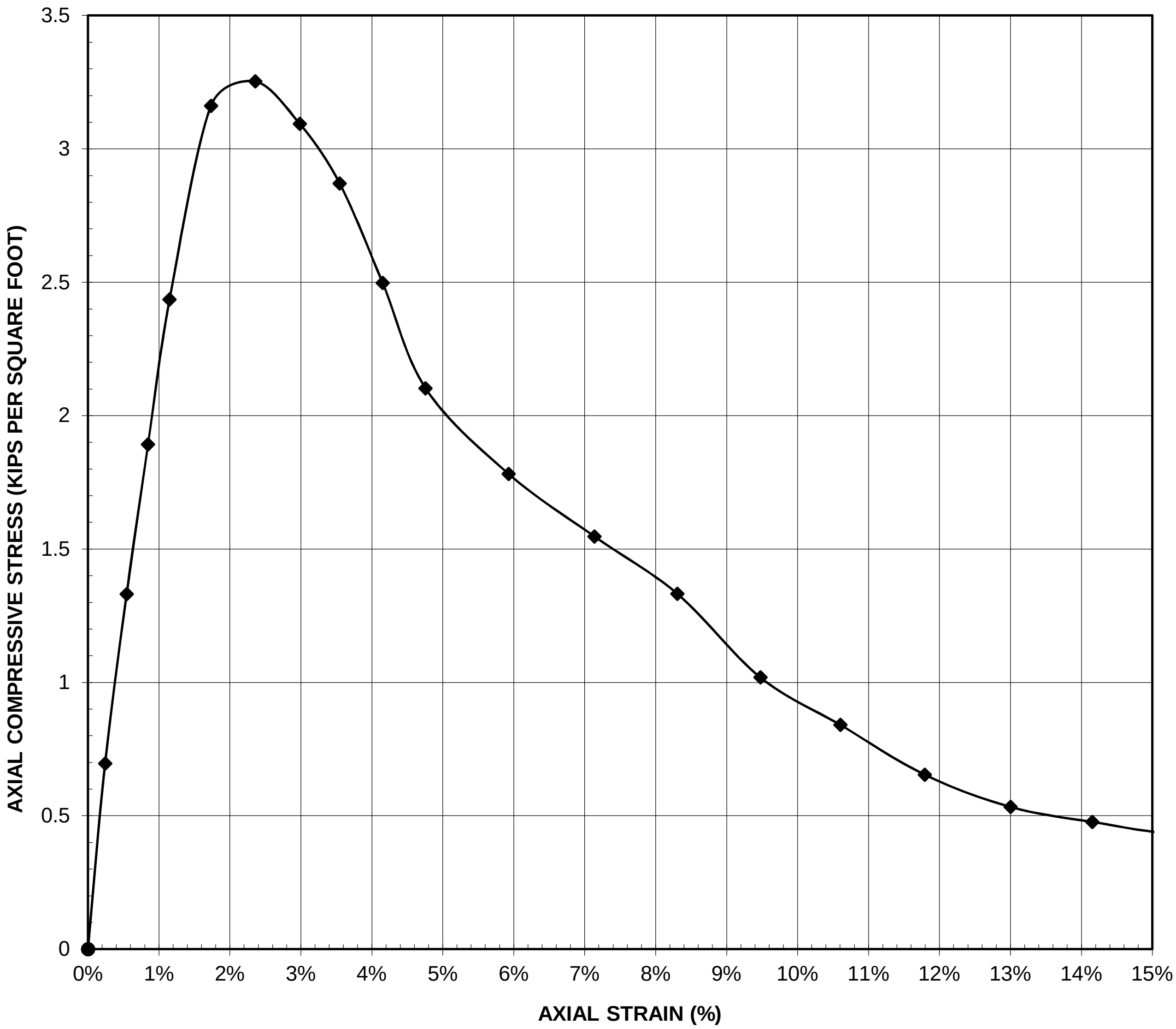

SYMBOL	DESCRIPTION	SOIL TYPE	SAMPLE LOCATION	SAMPLE DEPTH (ft.)	MOISTURE CONTENT w, (%)	DRY DENSITY γ_d, (pcf)	STRAIN RATE (%/min.)	UNDRAINED SHEAR STR s_u, (ksf)
◆	Gray weathered Mudstone	N/A	B-3	9.5-10.0	29.3	88.9	1.97	1.63

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2166

FIGURE C-3

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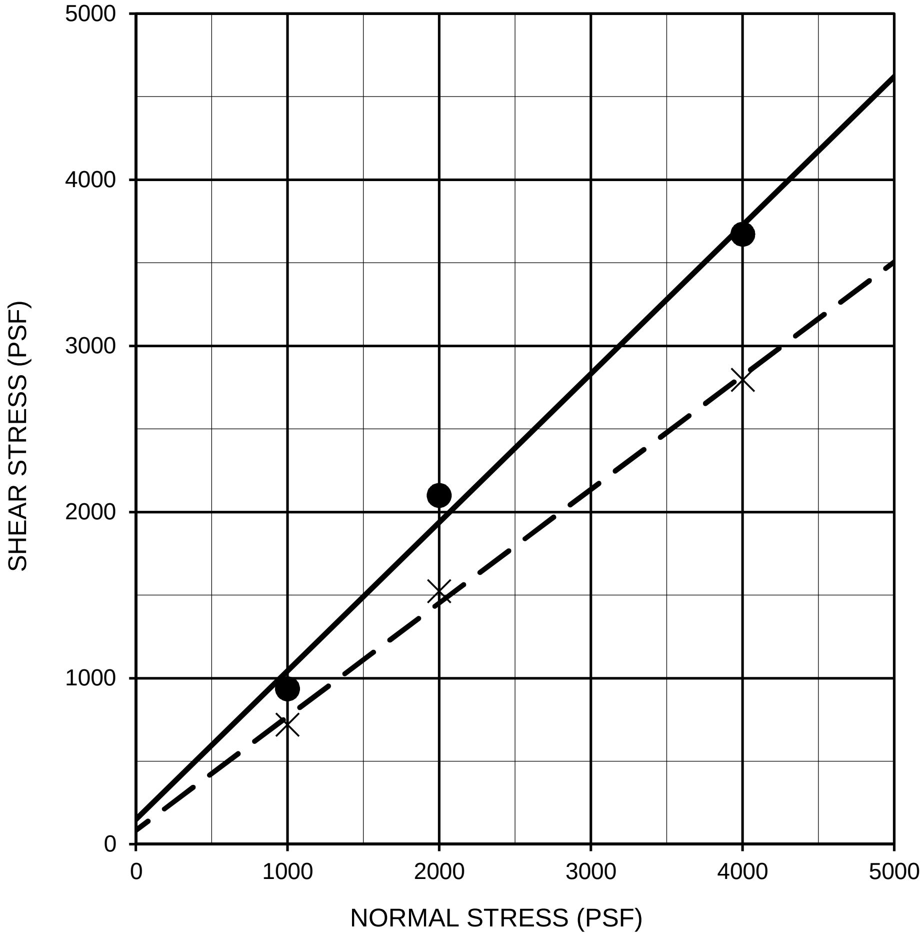
UNCONFINED COMPRESSION RESULTS
2035 NORTH PACIFIC AVENUE
SANTA CRUZ, CALIFORNIA



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
MUDSTONE	——●——	B-2	5.5-6.0	Peak	150	42	MUDSTONE
MUDSTONE	– – X – –	B-2	5.5-6.0	Ultimate	80	34	MUDSTONE

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE C-4

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DIRECT SHEAR TEST RESULTS

2035 NORTH PACIFIC AVENUE
SANTA CRUZ, CALIFORNIA

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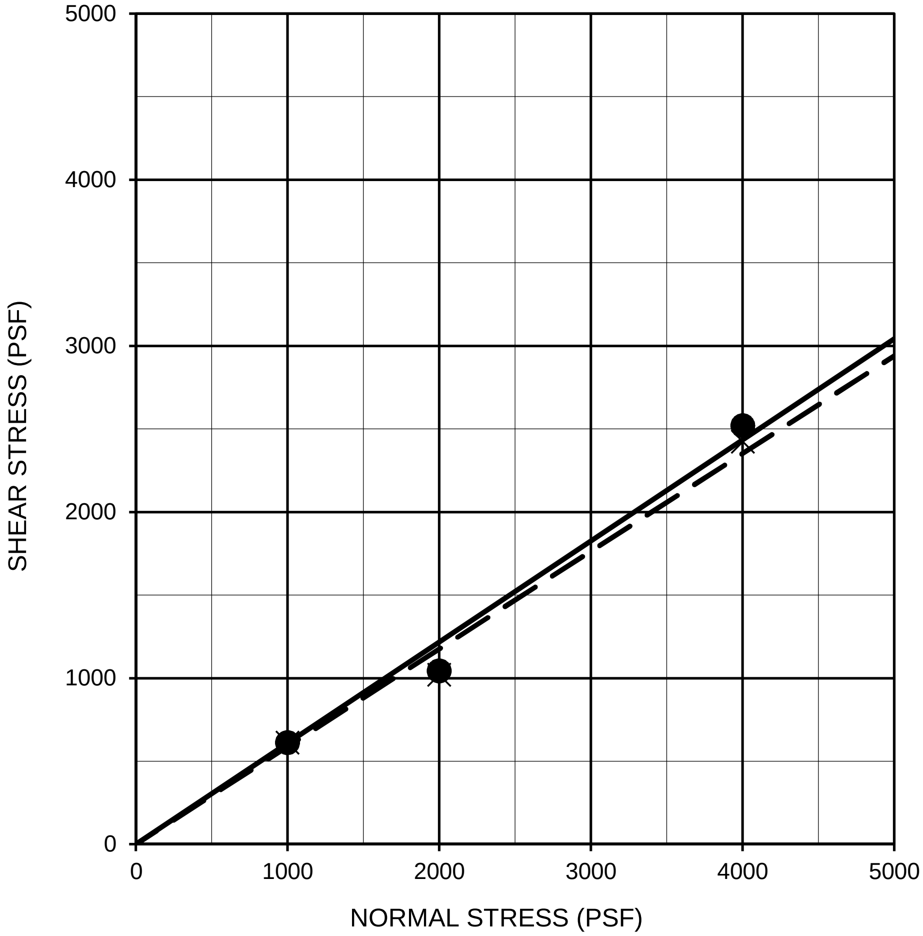

Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
sandy SILT	——●——	B-3	5.5-6.0	Peak	150	33	ML
sandy SILT	– – X – –	B-3	5.5-6.0	Ultimate	80	32	ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE C-5

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DIRECT SHEAR TEST RESULTS

2035 NORTH PACIFIC AVENUE
SANTA CRUZ, CALIFORNIA

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1]	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
B-3	0.0-2.5	6.6	1,400	10	0.001	220

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643

[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417

[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

FIGURE

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CORROSIVITY TEST RESULTS

2035 NORTH PACIFIC AVENUE.
SANTA SRUZ, CALIFORNIA

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APPENDIX D

Calculations

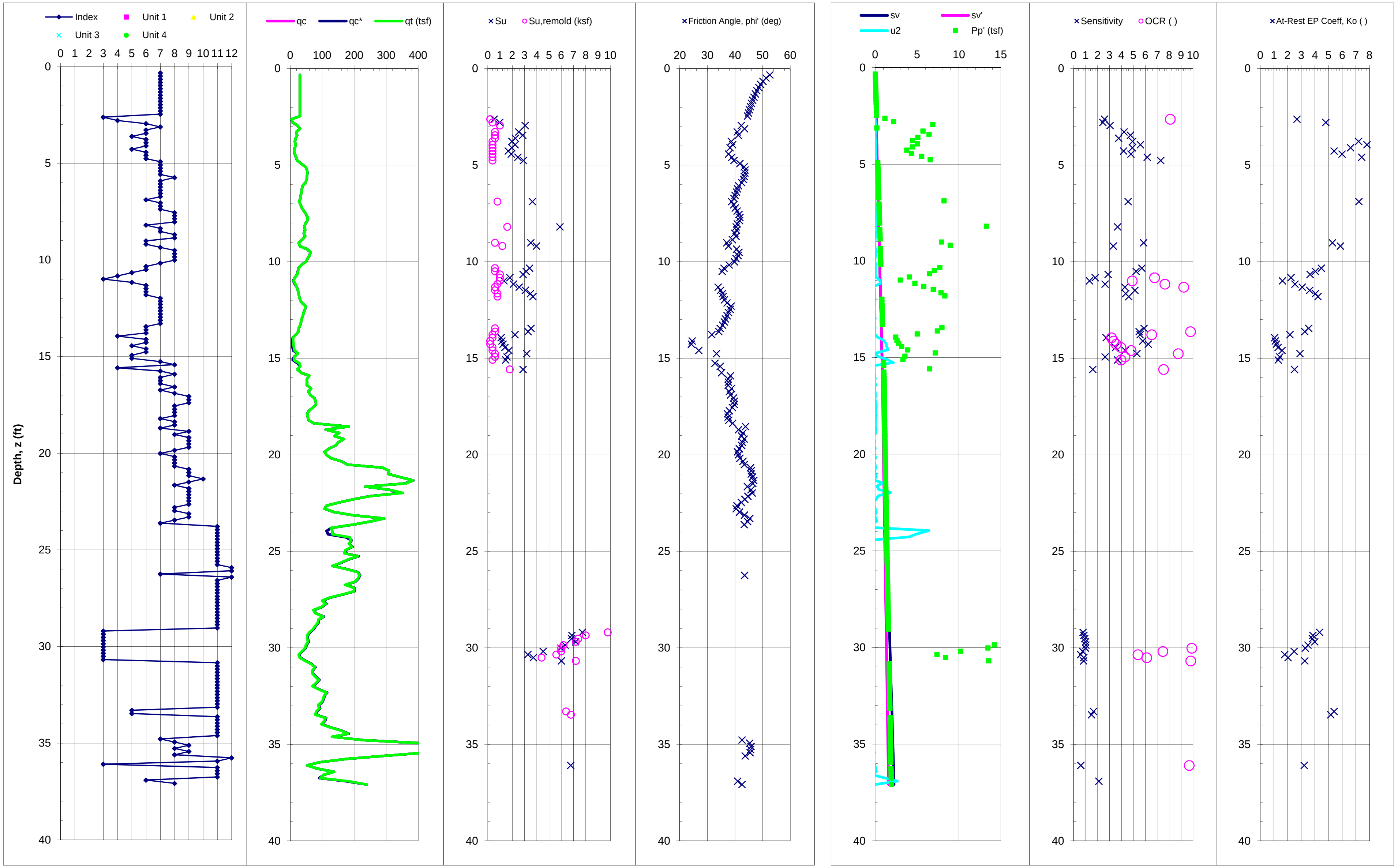
CONE PENETRATION TEST DATA

CPT Sounding: CPT-1

Location:

Elevation at top of sounding (ft, MSL)	20
Depth to GWT during CPT evaluation (ft)	16
Cone Diameter, dc (mm)	35.7
Net End Area Ratio ( )	0.80
Atmospheric Pressure (tsf)	1

Project Name:	2035 NORTH PACIFIC AVE		
Project Number:	403215001		
Calculation By:	TPS	Date:	5/14/2018
Checked By:		Date:	

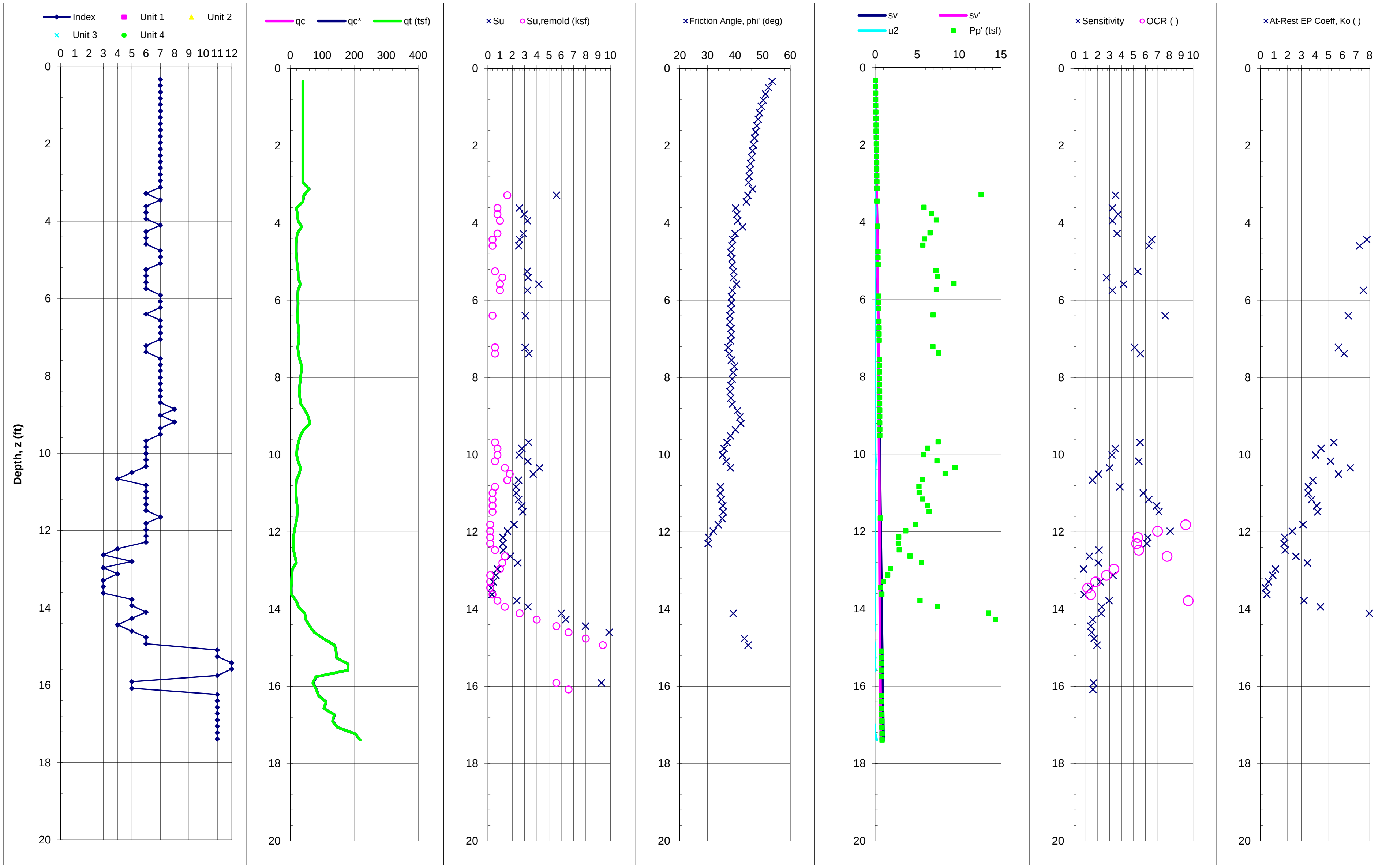


CONE PENETRATION TEST DATA

CPT Sounding: CPT-2
Location:

Elevation at top of sounding (ft, MSL)	8
Depth to GWT during CPT evaluation (ft)	7.4
Cone Diameter, dc (mm)	35.7
Net End Area Ratio ( )	0.80
Atmospheric Pressure (tsf)	1

Project Name:	2035 NORTH PACIFIC AVE		
Project Number:	403215001		
Calculation By:	TPS	Date:	5/14/2018
Checked By:		Date:	



File: rcpt data vers5.xlsx
Sheet: CPT2

5/15/2018

LIQUEFACTION AND DYNAMIC SETTLEMENT EVALUATION BY CPT

Project Name:	2035 NORTH PACIFIC AVE		
Project Number:	403215001		
Calculation By:	TPS	Date:	5/14/2018
Checked By:		Date:	

CPT Sounding: CPT-1
Location:

Depth to GWT during CPT evaluation (ft)	16
Design Depth to GWT (ft)	10
Atmospheric Pressure (tsf)	1.0581
Design EQ Peak Ground Acceleration, amax (g)	0.5
Design Earthquake Moment Magnitude, Mw	7
Magnitude Scaling Factor, MSF	1.19
At-Rest Coefficient Lateral EP, Ko	0.5
Number of Strain Cycles, Nc	10.85

Estimated dry soil dynamic settlement (in)	0.08
Estimated saturated soil dynamic settlement (in)	2.56
Total estimated dynamic settlement (in)	2.64

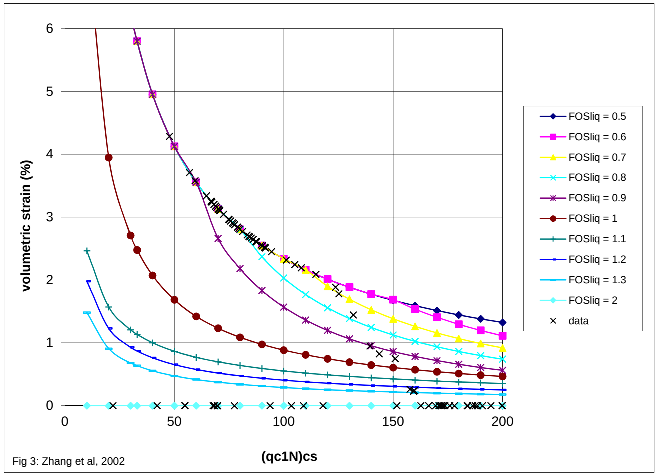
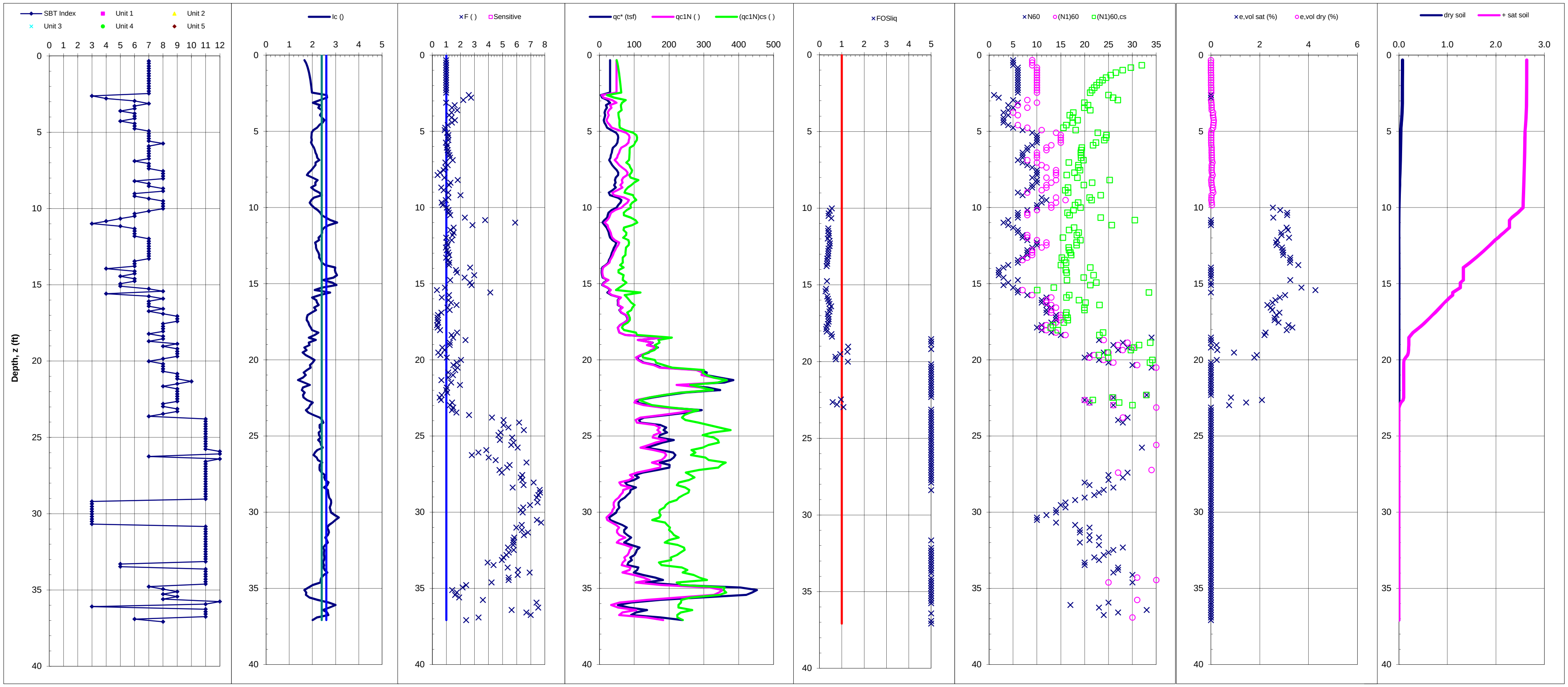


Fig 3: Zhang et al, 2002



LIQUEFACTION AND DYNAMIC SETTLEMENT EVALUATION BY CPT

Project Name:	2035 NORTH PACIFIC AVE		
Project Number:	403215001		
Calculation By:	TPS	Date:	5/14/2018
Checked By:		Date:	

CPT Sounding: CPT-2
Location:

Depth to GWT during CPT evaluation (ft)	16
Design Depth to GWT (ft)	10
Atmospheric Pressure (tsf)	1.0581
Design EQ Peak Ground Acceleration, amax (g)	0.5
Design Earthquake Moment Magnitude, Mw	7
Magnitude Scaling Factor, MSF	1.19
At-Rest Coefficient Lateral EP, Ko	0.5
Number of Strain Cycles, Nc	10.85

Estimated dry soil dynamic settlement (in)	0.16
Estimated saturated soil dynamic settlement (in)	0.82
Total estimated dynamic settlement (in)	0.98

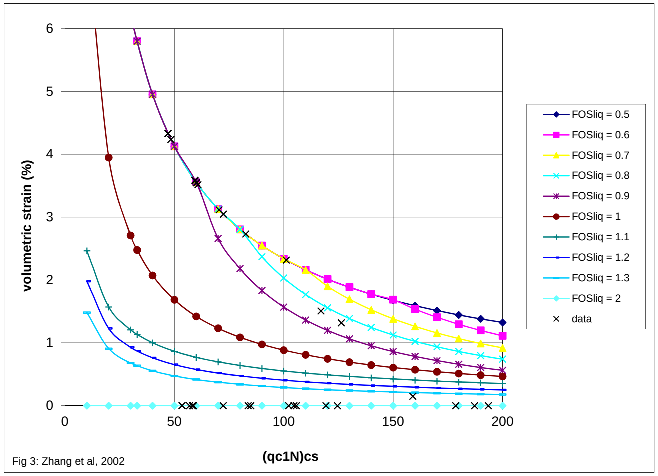
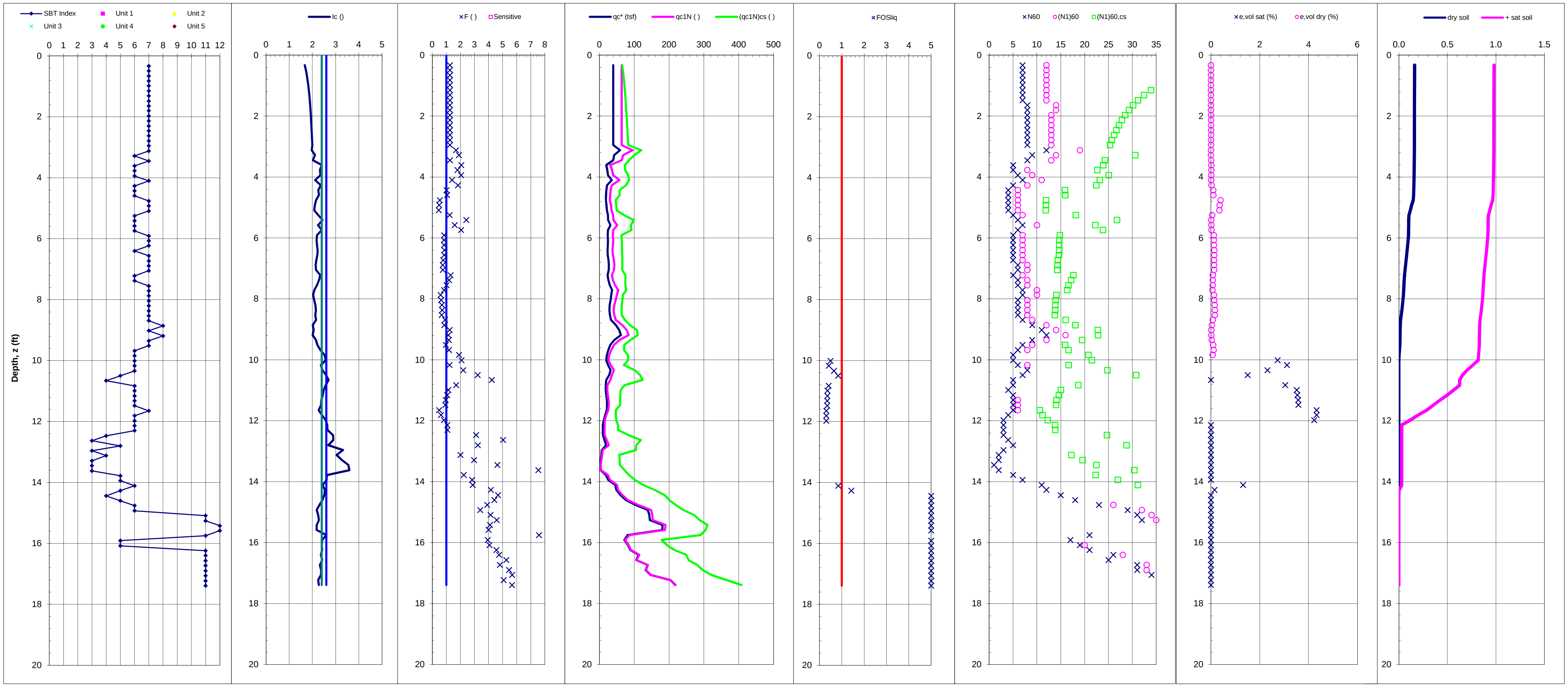


Fig 3: Zhang et al, 2002



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