Appendix D:
Geotechnical Investigation

Geotechnical Evaluation and Geologic Hazard Assessment

New Library and Learning Resource Center Building
Solano Community College – Fairfield Campus
4000 Suisun Valley Road
Fairfield, California

Solano Community College District
4000 Suisun Valley Road | Fairfield, California 94534

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

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Geotechnical Evaluation and Geologic Hazard Assessment

New Library and Learning Resource Center Building
Solano Community College – Fairfield Campus
4000 Suisun Valley Road
Fairfield, California

Mr. Jason Yi
Solano Community College District
4000 Suisun Valley Road | Fairfield, California 94534

December 12, 2017 | Project No. 403147001

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DRAFT

1 INTRODUCTION

In accordance with your authorization, we have performed a geotechnical evaluation and geologic hazards assessment for the design and construction of the proposed new library/learning resource center building at the Solano Community College District Fairfield Campus at 4000 Suisun Valley Road in Fairfield, California (Figure 1). This report presents the findings and conclusions from our geologic hazards assessment, and our geotechnical recommendations for improvements at the site.

2 SCOPE OF SERVICES

Our scope of services included the following:

- Review of readily available background materials, including geologic maps, aerial photographs, topographic data, and hazard maps.
- Site reconnaissance to observe the general site conditions and to mark the locations for our subsurface exploration.
- Obtained three boring permits from the Solano County Department of Resource Management.
- Reviewed existing utility plans provided. Coordinated with Underground Service Alert (USA) to locate underground utilities in the vicinity of our subsurface exploration.
- Subsurface exploration consisting of four (4) cone penetration test (CPT) soundings and eight (8) exploratory borings. Four (4) of the borings were advanced to 10 feet below the existing ground surface and four (4) were drilled to 20 feet below the existing ground surface. The soundings were advanced to depths of up to 55½ feet. 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 in compliance with the Solano County permit.
- Disposed of the cuttings in a landfill accepting non-hazardous waste.
- Laboratory testing of selected soil samples was performed to evaluate the geotechnical properties of the subsurface materials including in-situ soil moisture content and density, grain size distribution, Atterberg limits, expansion index, consolidation characteristics, shear strength, and soil corrosivity, as appropriate for the subsurface materials encountered.
- Data compilation and engineering analysis of the information obtained from our background review, subsurface evaluation, and laboratory testing.
- Preparation of this geologic hazards assessment and geotechnical evaluation report presenting our findings and conclusions regarding the potential geologic hazards and geotechnical conditions at the project site, and our geotechnical recommendations for proposed improvements.

3 SITE DESCRIPTION

The school campus is located at 4000 Suisun Valley Road in Fairfield, California (Figure 1). The campus is located south of Rockville Road between Suisun Valley Road to the west and Suisun Creek to the east (Figure 1). Existing campus improvements are generally encircled by Solano College Road (a loop road).

The site is located at approximately 38.2357 degrees north latitude and 122.1233 degrees west longitude. The project area is generally occupied by open space, pedestrian walkways, and landscaping. The site is located in a courtyard area surrounded by existing buildings, including Building 100 to the south, Buildings 700 and 800 to the west, Buildings 900 and 1200 to the north, and Buildings 1400 and 2700 to the east. The project area is relatively flat with elevation of about 42 to 45 feet above mean sea level (MSL) (CSW, 2017).

Historical topographic maps and aerial photographs that we reviewed indicate that the site was used for agricultural purposes prior to development of the community college in the early 1970's. We did not observe any tonal lineaments or other features suggestive of active faulting on the historical aerial photographs that we reviewed on Google Earth and the USGS historical aerial photograph website (https://earthexplorer.usgs.gov).

4 PROJECT DESCRIPTION

Based on our review of the Request for Proposal dated September 7, 2017 and Addendum #01, dated September 20, 2017, we understand that the proposed improvements will consist of the construction of a new library and learning center building in the central portion of the campus. The library will be two-stories in height with a building footprint of about 30,000 square feet. Other associated improvements are anticipated to include site work improvements, minor retaining walls, and utility installations.

5 BACKGROUND REVIEW

As part of our evaluation we reviewed in-house reports prepared for other projects located at the campus, including the solar photovoltaic arrays project (Ninyo & Moore, 2013a), the expansion of Building 600 project (Ninyo & Moore, 2013b), and the Building P2 and Building 1200 Theater Renovation project (Ninyo & Moore, 2014). We also reviewed available geotechnical reports from other consultants, including the report conducted for the new science building, Building 2700 (Wallace-Kuhl & Associates, 2015).

6 FIELD EXPLORATION AND LABORATORY TESTING

Our subsurface exploration at the site was performed on November 2, 2017. The subsurface exploration consisted of four (4) CPT soundings advanced to a depth of up to 55½ feet and eight (8) small-diameter borings advanced to a depth of up to about 20 feet below the existing ground surface. The approximate locations of the borings and soundings are presented on Figure 2.

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 in compliance with the Solano County drilling permits. Detailed logs of the borings are presented in Appendix A.

The CPT soundings were performed using a truck-mounted rig with a 20-ton reaction capacity. 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, expansion index, consolidation characteristics, shear strength, and soil 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

Our findings regarding regional geologic setting, site geology, subsurface stratigraphy, and groundwater conditions at the subject site are provided in the following sections.

7.1 Regional Geologic Setting

The campus is located north of Suisun 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

Review of available geologic maps and reports indicates that the project area is underlain by Holocene age alluvial fan deposits (Figure 4). According to regional geologic studies by Bezore et al. (1998a and 1998b) and Graymer et al. (2002), the Holocene age alluvial fan deposits typically consist of silt and clay interbedded with layers of sand and gravel. The alluvial deposits are derived from the bedrock formations exposed in the nearby foothills and local mountains. The local bedrock formations are part of the Pliocene age Sonoma Volcanics and consist of layers of ash flow tuff, andesite, and basalt.

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 logs in Appendix A. Cross sections depicting our interpretation of the subsurface conditions are presented as Figures 5 through 8.

7.3.1 Fill

Fill was encountered in the borings from the ground surface to depths of up to about 2 feet. The fill encountered generally consisted of brown to grayish brown, dry to moist, firm to stiff lean clay.

7.3.2 Alluvium

Alluvium was encountered in the borings and soundings to the depths explored of up to 55½ feet. The alluvium, as encountered, generally consisted of gray, brown and yellowish brown, moist to wet, firm to very stiff, lean clay with occasional layers of sand. Sand layers were encountered in Boring B-1 at a depth of about 17 feet to 19½ feet. The sand layer generally consisted of wet, medium dense, clayey sand with trace amounts of gravel. Several thin sand layers were encountered in the CPT soundings at depths between 18 and 29 feet.

7.4 Groundwater

Groundwater was encountered in Borings B-1 through B-4 at depths of about 9½ feet, 7 feet, 14½ feet, and 16½ feet respectively during drilling. Groundwater was not encountered in the other borings. During pervious evaluations for other projects at the campus, groundwater was encountered at around ½ to 26½ feet below the existing ground surface (Ninyo & Moore, 2013a), 14 feet below the existing ground surface (Ninyo & Moore, 2013b), and 10 to 20 feet below the existing ground surface (Ninyo & Moore, 2014). The investigation for the new science building encountered groundwater at around 11 feet below the existing ground surface (Wallace-Kuhl & Associates, 2015).

Fluctuations in the groundwater level across the site and over time may occur due to seasonal precipitation, variations in topography or subsurface hydrogeologic conditions, or as a result of changes to nearby irrigation practices or groundwater pumping. In addition, seeps may be encountered at elevations above the observed groundwater levels due to perched groundwater conditions, leaking pipes, preferential drainage, or other factors not evident at the time of our exploration.

8 GEOLOGIC HAZARDS AND CONSIDERATIONS

This study considered a number of issues relevant to the proposed construction, including seismic hazards, flood hazards, landsliding and slope stability, naturally occurring asbestos, settlement of compressible soil layers from static loading, unsuitable materials, excavation characteristics, soil corrosivity, and expansive soils. These issues are discussed in the following subsections.

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 and seiches. These potential hazards are discussed in the following subsections.

8.1.1 Historical Seismicity

The site is located in a seismically active region. Figure 3 presents the location of the site relative to the epicenters of historic earthquakes with magnitudes of 5.5 or more from 1800 to 2000. Records of historic ground effects related to seismic activity (e.g. liquefaction, sand boils, lateral spreading, ground cracking) compiled by Knudsen et al. (2000), indicate that no ground effects related to historic seismic activity have been reported for the site vicinity.

8.1.2 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, 2017) 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)
Cordelia fault	107	Strike Slip	6.5	1.2
Green Valley 2011 CFM	108	Strike Slip	6.8	3.4
Great Valley 04b Gordon Valley	104	Reverse	6.7	14.0
Los Medanos - Roe Island	120	Reverse	6.8	11.0
West Napa fault zone (Napa County Airport section)	114	Strike Slip	6.6	13.5
Great Valley 05 Pittsburg Kirby Hills alt2	111	Reverse	6.6	17.1
West Napa fault zone (Browns Valley section)	106	Strike Slip	6.6	14.9
Contra Costa Shear Zone (connector) 2011 CFM	117	Strike Slip	6.5	13.7
Rodgers Creek	103	Strike Slip	7.3	28.9
Vaca fault zone	109	Strike Slip	6.4	15.6

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 Cordelia Fault, which is located approximately ½ mile west of the site.

8.1.3 Strong Ground Motion

Based on historic activity, the potential for future strong ground motion at the site is considered significant. Seismic design criteria to address ground shaking are provided in Section 10.2. The peak ground acceleration (PGA) associated with the Maximum Considered Earthquake Geometric Mean (MCE_G) was calculated in accordance with the American Society of Civil Engineers (ASCE) 7-10 Standard and the 2016 California Building Code (CBC). The MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) was calculated as 0.809g using the USGS seismic design tool (USGS, 2017) that yielded a mapped MCE_G peak ground acceleration of 0.809g for the site and a site coefficient (F_{PGA}) of 1.000 for Site Class D.

8.1.4 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. The Association of Bay Area Governments (ABAG) notes that the campus is in area considered to have a moderate susceptibility to liquefaction based on regional studies (Knudsen et al., 2000; Witter et al., 2006).

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 6 feet below the ground surface, and considering a seismic event producing a PGA of 0.809g resulting from a Magnitude 6.8 earthquake. The results of our analysis, presented in Appendix D, indicate that thin layers of sandy soil 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. Based on the depth and relative thickness (total thickness of 6 inches or less per location) of the liquefiable layers encountered, we do not regard the potential for sand-boil-induced ground subsidence or liquefaction-induced reduction in the bearing capacity of shallow foundations as a design consideration for the project. Other consequences of liquefaction, including dynamic settlement and lateral spreading, are addressed in the following sections.

Estimates of undrained and remolded shear strength based on CPT tip resistance and sleeve friction, respectively, indicate that the cohesive soils during our subsurface exploration are not particularly sensitive. As such we do not regard seismically induced strain-softening behavior as a design consideration.

8.1.5 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.

We evaluated the potential for dynamic settlement for layers with factor of safety against liquefaction of 1.3 or less using the CPT data collected during our subsurface exploration and an in-house developed spreadsheet program based on the method presented by Zhang et al. (2002) for saturated soil and by Robertson and Shao (2010) for dry soil. Our analysis considered a Magnitude 6.8 earthquake producing a PGA of 0.809g and groundwater level 6 feet below the ground surface. The results of our analyses, presented in Appendix D, indicate that the total dynamic settlement following the considered seismic event will be up to approximately ¼ inch following the considered seismic event. For design purposes, we recommend using a total dynamic settlement of 1 inch with a differential settlement of ½ inch over a horizontal distance of 30 feet.

8.1.6 Lateral Spreading

In addition to vertical displacements, seismic ground shaking can induce horizontal displacements as surficial deposits spread laterally by floating atop liquefied subsurface layers. Lateral spreading can occur on sloping ground or on flat ground adjacent to an exposed face. Lateral spreading will not occur unless a liquefiable layer of sufficient lateral continuity is present. Our subsurface exploration did not reveal a liquefiable layer of significant continuity. Furthermore, there are no significant slopes or free face conditions at the site. As such, we do not regard lateral spreading as a design consideration for this project

8.1.7 Seismic Slope Stability

No significant slopes are present on the site, as such, we do not regard seismic slope stability as a design consideration for this project.

8.1.8 Tsunamis and Seiches

Tsunamis are long wavelength seismic sea waves (long compared to ocean depth) generated by the sudden movements of the ocean floor during submarine earthquakes, landslides, or volcanic activity. The project is not located within a tsunami evacuation area as shown on the tsunami evacuation planning maps for California.

Seiches are waves generated in a large enclosed body of water. Based on the inland location and the lack of large enclosed bodies of water near the site, the potential for damage due to tsunamis or seiches is not a design consideration.

8.2 Flood Hazards

Our review of Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FEMA, 2009) found that the site lies within a 0.2% annual chance flood plain (500 year flood zone).

8.3 Landsliding and Slope Stability

The site and surrounding area are relatively flat and the proposed improvements do not include construction of significant slopes. As such, we do not regard landsliding or slope stability a design consideration.

8.4 Naturally Occurring Asbestos

According to State of California guidelines established by the California Department of Toxic Substances and Control (2004 and 2005), a Preliminary Environmental Assessment (PEA) is recommended for school sites that are located within a 10-mile radius of any rock formation that may contain naturally occurring asbestos (NOA). The nearest mapped location of ultramafic rock from which NOA may be found is over 10 miles from the campus (Churchill and Hill, 2000; and Brabb et al., 1998). Based on these conditions, NOA is not a design consideration for this project.

8.5 Static Settlement

We understand that the proposed improvements will be relatively light to moderate and that significant changes to the site grade are not proposed. We anticipate, therefore, that settlement due to sustained loading by the proposed improvements will be tolerable provided that those improvements are supported on shallow foundations designed in accordance with the recommendations in this report.

8.6 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 2 feet below the ground surface during our subsurface exploration. Recommendations for subgrade preparation and foundation embedment recommendations 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.

8.7 Excavation Characteristics

We anticipate that the project will involve excavations of depths up to 5 feet for foundations and utility trenches. We anticipate that heavy earthmoving equipment in good working condition should be able to make the proposed excavations.

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 near surface deposits up to 4 feet in depth should remain stable for a limited period of time. However, 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.

8.8 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. California Department of Transportation (Caltrans) defines a corrosive environment as an area within 1,000 feet of brackish water or where the soil contains more than 500 parts per million (ppm) of chlorides, sulfates of 0.2 (2,000 ppm) percent or more, or pH of 5.5 or less (Caltrans, 2012). 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)

8.9 Expansive Soils

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. Laboratory testing was performed on a select sample of the near-surface soil to evaluate the expansion index. The test was performed in general accordance with the American Society of Testing and Materials (ASTM) Standard D 4829 (Expansion Index). The results of our laboratory testing indicate that the expansion index of the near-surface soil is 100, which is consistent with a high expansion characteristic. To reduce the potential for differential movement and distress to the proposed improvements due to shrink/swell behavior, recommendations are provided for remedial grading, foundation embedment depths, and subgrade preparation.

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 proposed construction is feasible from a geotechnical standpoint. Geotechnical considerations include the following:

- Our subsurface exploration encountered undocumented fill and alluvium. Fill was encountered to depths of up to about 2 feet. The fill generally consisted of gray, brown and yellowish brown, moist to wet, firm to very stiff, lean clay with occasional layers of sand. The alluvium generally consisted of gray, brown and yellowish brown, moist to wet, firm to very stiff, lean clay with occasional layers of sand.

- Undocumented fill and soil containing roots or other organic matter are not suitable as subgrade below foundations. Undocumented fill was generally encountered to depths of up to about 2 feet below the ground surface in the borings. Recommendations for subgrade preparation and foundation embedment depth are provided.

- Groundwater was encountered in Borings B-1 through B-4 at depths of between 7 and 16½ feet below the existing ground surface. Variation and fluctuation in groundwater levels should be anticipated as discussed in Section 7.4.

- The site could experience a relatively large degree of ground shaking during a significant earthquake on a nearby fault. Seismic design criteria are presented in Section 10.2.

- The results of our liquefaction evaluation, presented in Appendix D, indicate that relatively thin layers of sandy soil will liquefy under the considered ground motion. However due to the depth and relative thickness of the liquefiable layers, we do not regard the potential for liquefaction-induced reduction in the bearing capacity of shallow foundations as a design consideration for the project.

- The results of our dynamic settlement analysis, presented in Appendix D, indicate that a total dynamic settlement of approximately ¼ inch will occur due to the assumed ground motion. For design purposes, we recommend using a total dynamic settlement of 1 inch with a differential settlement of ½ inch over a horizontal distance of 30 feet.

- Tsunamis, seiches, and ground surface rupture due to faulting are not design considerations based on the location of the project.

- Excavations that remain unsupported and exposed to water, or encounter seepage, or granular soil may be unstable and prone to sloughing. Recommendations for excavation stabilization are provided.

- Excavations in the fill may encounter debris, rubble, oversize material, buried objects, or other potential obstructions.

- The site is not in a flood hazard zone.

- Landsliding and slope stability are not design considerations based on the relatively flat topographic variation at the site.

- High concentrations of naturally occurring asbestos (NOA) in the natural soils at the site are unlikely based on the nearest mapped location of ultramafic rock from which NOA may be found is over 10 miles from the school campus.

- Static settlement should be tolerable for the proposed improvements provided that the proposed structures are supported on foundations that conform with our recommendations and fill placement to raise grades is less than 2 feet in height.

- Based on the results of our limited soil corrosivity tests during this study and Caltrans corrosion guidelines (2012), the site does not meet the definition of a corrosive environment.

- Expansion Index testing indicates that the near-surface soil on site has a high expansion characteristic. Recommendations are provided for remedial grading, foundation embedment depths, and subgrade preparation to reduce the potential for expansive soil movement below proposed improvements.

10 RECOMMENDATIONS

The following sections present our geotechnical recommendations for the design and construction of the proposed improvements. The project improvements should be designed and constructed in accordance with these recommendations, applicable codes, and appropriate construction practices.

10.1 Earthwork

The earthwork should be conducted in accordance with the relevant grading ordinances having jurisdiction and the following recommendations. The geotechnical engineer should observe earthwork operations. Evaluations performed by the geotechnical engineer during the course of field operations may result in new recommendations, which could supersede the recommendations in this section.

10.1.1 Pre-Construction Conference

We recommend that a pre-construction conference be held to discuss the grading recommendations presented in the report. The owner and/or their representative, the architect, the engineer, Ninyo & Moore, and the contractor should be in attendance to discuss project schedule and earthwork requirements.

10.1.2 Site Preparation

Site preparation should begin with the removal of vegetation, utility lines, debris and other deleterious materials from areas to be graded. Tree stumps and roots should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside of the proposed excavation and fill areas. 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 buried utilities, tree stumps, or obstructions should be backfilled with compacted fill in accordance with the recommendations in the following sections.

10.1.3 Subgrade Observations

Prior to placement of fill, erection of forms or placement of 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 the geotechnical engineer in accordance with the recommendations in this section or the field recommendations of the geotechnical engineer.

Unsuitable materials include, but may not be limited to dry, loose, soft, wet, expansive, organic, or compressible natural soil; and undocumented or otherwise deleterious fill materials. 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 the geotechnical engineer, is exposed.

10.1.4 Remedial Grading for Site Improvements

Laboratory testing indicates that the near-surface soil on site has a high expansion characteristic. To reduce the potential for differential movement and distress to the proposed improvements due to shrink/swell behavior, a zone of material with low expansion potential should be created by removing the existing soil, as-needed, and placing fill with low expansion characteristics below building slabs-on-grade, flatwork, and pavement. The zone of low expansion fill should consist of select, low-expansion import fill conforming with Section 10.1.5. Alternatively, the on-site soil may be chemically treated by mixing the soil with lime as described in Section 10.1.6 to reduce the expansion characteristic and create the zone of low-expansion material.

The lateral limits of overexcavations or chemical treatment should extend a distance of 5 feet or more beyond the limits of the slab-on-grade and 2 feet or more beyond the limits of the flatwork or pavement. The zone of low expansion material should extend to a depth of 24 inches below building slabs-on-grade; and 12 inches below exterior flatwork or pavement. The aggregate base or capillary break gravel under building slabs or exterior flatwork or pavement may be considered as part of the zone of low expansion material. The zone of exclusion/removal or lime treatment should be detailed on the construction plans to reduce the potential that these recommendations are overlooked during construction bidding.

Undocumented fill was encountered in the borings to a depth of up about 2 feet below the existing ground surface. Undocumented fill, where encountered, should be removed from below new building footings. Excavations should be backfilled with controlled low strength material (CLSM) as per Section 10.1.5. Alternatively the footings may be extended to bear on suitable alluvium. The depth of the undocumented fill may vary and extend deeper than observed in the borings. Undocumented fill that can be processed to meet the general criteria in Section 10.1.5 can be re-used as general fill.

10.1.5 Material Recommendations

Materials used during earthwork operations should comply with the requirements listed in Table 3. 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 consistency of import material brought to the site.

Table 3 – Recommended Criteria for Materials

Material and Use	Source	Requirements[1,2]
Select (Low Expansion) Fill: - below building slabs or flatwork	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
	On-site borrow	Treated with lime per Section 10.1.6
Select Structure Backfill - behind retaining walls[5]	Import	Sand Equivalent 20 or more 100 percent passing 3-inch sieve 35 to 100 percent passing No. 4 Sieve 20 to 100 percent passing No. 30 sieve
General Fill: - for uses not otherwise specified	Import	As per Select (Low Expansion) Fill
	On-site borrow	No additional requirements[1]
Controlled Low Strength Material (CLSM)	Import	CSS[4] Section 19-3.02F
Permeable Aggregate - capillary break gravel	Import	Open-graded, clean, compactable crushed rock or angular gravel; nominal size ¾ inch or less
Aggregate Base	Import	Class II; CSS[4] Section 26-1.02
Pipe/Conduit Bedding and Pipe Zone Material -material below pipe invert to 12 inches above pipe	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

[1] In general, fill should not consist of pea-gravel and should be free of rocks or lumps in excess of 6-inches 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, 2012).

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

[5] Placed above a plane rising up and away from the heel of the wall at a 1:1 angle.

10.1.6 Chemical Treatment

The on-site soil may be chemically treated with quicklime to reduce the expansion characteristic of the soil as an alternative to importing select fill. The quicklime should conform with ASTM standard C977.

On-site materials containing roots or other organic matter are not suitable for chemical treatment and should be stripped from the area at which the treatment is to be performed. The chemical treatment should be performed by an experienced contractor that specializes in the chemical treatment of soil. The chemical agent should be proportioned and spread with a mechanical spreader and mixed into the soil on a mixing table or in place to produce consistent distribution of the agent within the treated layer. The depth of mixing should not exceed 18 inches per lift or the capacity of the mixer if less. Precautions to reduce the potential for dusting of quicklime, such as scheduling or suspending operations to avoid windy weather, should be taken. Casting or tailgating of the chemical agent should not be permitted. The mixer should be equipped with a rotary cutting/mixing assembly, grade checker, and an automatic water distribution system. Mixing or spreading operations should not be performed during inclement weather or when the ambient temperature is less than 35 degrees Fahrenheit or during foggy or rainy weather. Adjacent passes of the mixer should overlap by 4 inches or more.

To reduce the expansive soil characteristic, quicklime should be mixed into the soil at a rate of 3 percent or more by dry weight of soil. The plasticity index of the treated soil should be 12, or less. Mixing and pulverizing should continue until the treated soil does not contain untreated soil clods larger than 1 inch and the quantity of untreated soil clods retained on the No. 4 sieve is less than 40 percent of the dry soil mass. Water should be added as-needed during the mixing process to achieve moisture content above the optimum, as evaluated by ASTM D1557, for the lime-soil mixture. The lime-soil mixture should be re-mixed following a 16-hour mellowing period after the initial mixing. The lime-soil mixture should be compacted within 3 days after initial mixing per Section 10.1.8.

10.1.7 Subgrade Preparation

Subgrade in trenches and below slabs, footings, flatwork, pavement, 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.

Table 4 – Subgrade Preparation Recommendations	
Subgrade Location	**Preparation Recommendations**
Below Footings	• Check for unsuitable materials and remove as-needed per Sections 10.1.2 and 10.1.3. Replace overexcavated soil with CLSM or extend footing as-needed. • Scarify and moisture condition exposed subgrade as-needed to achieve a moisture content 2 points or more above the optimum as evaluated by ASTM D1557. Compact moisture-conditioned subgrade per Section 10.1.8. • Keep in moist condition by sprinkling water.
Below Slabs, Flatwork, and Pavement	• After clearing and grubbing per Section 10.1.2, check for unsuitable materials as per Section 10.1.3. • Perform remedial grading as per Section 10.1.4. Scarify 8 inches then moisture condition and compact as per Section 10.1.8 if in-place lime treatment is not performed. • Keep in moist condition by sprinkling water.
Below Fill	• After clearing and grubbing per Section 10.1.2, check for unsuitable materials as per Section 10.1.3. • Scarify 8 inches then moisture condition and compact as per Section 10.1.8. • Keep in moist condition by sprinkling water.
Utility Trenches	• After clearing per Section 10.1.2, check for unsuitable materials as per Section 10.1.3. • Remove or compact loose/soft material.

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. A thin layer (approximately 3 inches) of lean concrete or controlled low strength material (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.

10.1.8 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 slabs, flatwork, and footings and in locations not already specified	90 percent	+ 2 percent or above
	Below pavement	95 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	Below pavement (within 2 feet of finished grade)	95 percent	+ 2 percent or above
	In locations not already specified	90 percent	+ 2 percent or above
Select or General Fill (not lime-treated)	Below pavement (within 2 feet of finished grade)	95 percent	+ 2 percent or above
	In locations not already specified	90 percent	+ 2 percent or above
Lime-treated subgrade or fill	In locations not already specified	95 percent	At or above optimum
Aggregate Base	Below slabs, hardscape, or pavement	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 lime treated subgrade). The reference density of soil, lime-treated subgrade, and aggregate should be evaluated by ASTM D 1557.

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

Compacted fill 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. Fill 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.

10.1.9 Excavation Stabilization

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). Table 6 lists the OSHA material type classifications and corresponding allowable temporary slope layback inclinations for soil deposits that may be encountered on site. Alternatively, a shoring system conforming to the OSHA Excavation

Rules and Regulations (29 CFR Part 1926) may be used to stabilize excavation sidewalls during construction. The lateral earth pressures listed in Table 6 may be used to design or select an internally-braced shoring system or trench shield conforming to the OSHA guidelines. 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. Dewatering pits or sumps should be used to depress the groundwater level (if encountered) below the bottom of the excavation.

Table 6 – OSHA Material Classifications and Allowable Slopes

Formation	OSHA Classification	Allowable Temporary Slope[1,2,3]	Lateral Earth Pressure on Shoring[4] (psf)
Cohesive Fill & Alluvium (above groundwater)	Type B	1h:1v (45°)	45×D + 72
Granular Fill & Alluvium (above groundwater)	Type C	1½ h:1v (34°)	80×D + 72

Notes:

1 Allowable slope for excavations less than 20 feet deep. Excavation sidewalls in cohesive soil may be benched to meet the allowable slope criteria (measured from the bottom edge of the excavation). The allowable bench height is 4 feet. The bench at the bottom of the excavation may protrude above the allowable slope criteria.
2 In layered soil, layers shall not be sloped steeper than the layer below.
3 Temporary excavations less than 5 feet deep may be made with vertical side slopes and remain unshored if judged to be stable by a competent person (29 CFR, Part 1926.650).
4 'D' is depth of excavation for excavations up to 20 feet deep. Includes a surface surcharge equivalent to two feet of soil.

The shoring system 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. The contractor should be prepared to stabilize the bottom of the excavations. In general, unstable bottom conditions may be mitigated by scarifying the subgrade and aerating the soil to achieve a moisture content near the optimum, 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.

10.1.10 Construction Dewatering

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. Disposal of groundwater should be performed in accordance with the guidelines of the Regional Water Quality Control Board.

10.1.11 Utility Trenches

Trenches constructed for the installation of underground utilities should be stabilized in accordance with our recommendations in Section 10.1.9. Utility trenches should be backfilled with materials that conform to our recommendations in Section 10.1.5. Trench backfill, bedding, and pipe zone fill should be compacted in accordance with Section 10.1.8 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.

10.1.12 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 Criteria

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 7 presents the seismic design parameters for the site in accordance with the CBC (2016) guidelines and adjusted MCER spectral response acceleration parameters (USGS, 2017).

Table 7 – 2016 California Building Code Seismic Design Criteria	
Seismic Design Parameter **Evaluated for 38.2357° North Latitude, 122.1233°West Longitude**	**Value**
Site Class	D[1]
Site Coefficient, Fa	1.0
Site Coefficient, Fv	1.5
Mapped M_{CER} Spectral Response Acceleration at 0.2-second period, S_S	2.114g
Mapped M_{CER} Spectral Response Acceleration at 1.0-second period, S_1	0.748g
Site-Adjusted M_{CER} Spectral Acceleration at 0.2-second period, S_{MS}	2.114g
Site-Adjusted M_{CER} Spectral Acceleration at 1.0-second period, S_{M1}	1.122g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.409g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.748g
Seismic Design Category for Risk Category I, II, or III	D

Note:
[1] For structures with fundamental period of vibration of ½ second or less

10.3 Foundation Recommendations

The new building may be supported on spread footings with slab-on-grade floors. The foundation should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the appropriate governing jurisdictions and applicable building codes should be considered in design of the structures.

10.3.1 Spread Footings

Footings bearing on alluvium or new engineered fill with subgrade prepared in accordance with the recommendations in Section 10.1.7 may be designed using the criteria listed in Table 8. The geotechnical engineer should observe the footing excavations to evaluate bearing materials and subgrade condition before the exposed subgrade is covered.

Table 8 – Recommended Bearing Design Parameters for Footings

Footing	Sustained Loads	Footing Widths[1]	Bearing Depth[2]	Allowable Bearing Capacity[3]	Static Settlement[4]
Wall Footing	2 kips/foot or less	1½ feet or more	2 feet or more	1,500 psf	1 inch total ½ inch differential over 30 feet
	5 kips/foot	3 feet or more	2 feet or more	1,500 psf	1½ inches total ¾ inch differential over 20 feet
Column Footing	10 kips or less	2 feet or more	2 feet or more	2,500 psf	1 inch total ½ inch differential over 20 feet
	50 kips	5 feet or more	2 feet or more	2,500 psf	1½ inches total ¾ inch differential over 20 feet
	130 kips	7 feet or more	2 feet or more	2,500 psf	2 inches total 1 inch differential over 20 feet

Notes:

[1] Assumes square footing shape for column footings.

[2] Below the adjacent finish grade and the existing grade.

[3] Net allowable bearing capacity in pounds per square foot. Listed value includes a Factor of Safety of 3 or more. Allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic loads.

[4] Based on sustained long-term loading conditions. Assumes that if footing width is increased from that shown in table, sustained load is equal to or less than value shown for each case.

[5] Designer can interpolate between the values presented in the Table. For example, for a sustained column footing load of 30 kips, footing width should be 3½ feet or more, bearing depth 2 feet or more, allowable bearing pressure should be 2,500 psf, and static settlement should be anticipated to be 1¼ inch total.

Structures supported on footings consistent with these recommendations should be designed for the total and differential settlements listed in Table 8 for sustained loads plus an additional 1 inch of total dynamic settlement with a differential dynamic settlement of about ½ inch over a lateral span of 30 feet.

Footing settlement due to static loads may be further evaluated using a modulus of subgrade reaction. Recommended values for the modulus of subgrade reaction are provided in Table 9. The designer may interpolate between the values in the table for intermediate footing widths.

Table 9 – Footing Modulus of Subgrade Reaction

Footing[1]	Footing Width					
	1½ feet	2 feet	3 feet	4 feet	7 feet	10 feet

Wall Footing	70 pci	49 pci	31 pci	22 pci	---	---
Column Footing[2]	---	95 pci	59 pci	42 pci	23 pci	16 pci

Notes:
[1] Assumes bearing depth of 24 inches below adjacent finish grade.
[2] Assumes square footing shape for columns
[3] Modulus of Subgrade Reaction in units of pounds per cubic inch.

The spread footings should be reinforced with deformed steel bars as detailed by the project structural engineer. Where footings are located adjacent to utility trenches or other excavations, the footing bearing surfaces should bear below an imaginary plane extending upward from the bottom edge of the adjacent trench/excavation at a 2:1 (horizontal to vertical) angle above the bottom edge of the footing. Footings should be deepened or excavation depths reduced as-needed.

A friction coefficient of 0.30 may be assumed for evaluating frictional resistance to lateral loads. A lateral bearing pressure of 300 psf per foot of depth up to 3,000 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 weight of the material above a plane rising up and away from the bottom edges of the footings at 20 degrees off plumb may be considered, along with the weight of the footing and the material over the footing, when evaluating footing resistance to uplift. A unit weight of 115 pounds per cubic foot (pcf) for soil or aggregate and 150 pcf for normal weight concrete may be assumed for this evaluation.

10.3.2 Slabs-on-Grade

Building floor slabs should be designed by the project structural engineer based on the anticipated loading conditions. Remedial grading beneath slabs-on-grade should be performed in in accordance with Section 10.1.4. The subgrade should be prepared in accordance with Section 10.1.7. Where a vapor regarding system is not used, slabs should be constructed on 6 inches, or more, of aggregate base conforming to Section 10.1.5 and placed in accordance with Section 10.1.8. The slab should be reinforced with deformed steel bars. We recommend that masonry briquettes or plastic chairs be used to aid in the correct placement of slab reinforcement in the upper half of the slab. Refer to Section 10.5 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 10.7 for vapor retarding system recommendations. Joints consistent with ACI guidelines (ACI, 2016) may be constructed at periodic intervals to reduce the potential for random cracking of the slab.

10.3.3 Drilled Piers for Minor Structures

Drilled piers for minor structures such as fences and light poles, embedded 3 to 20 feet below grade, may be designed using the following criteria.

10.3.3.1 Axial Load Resistance

Drilled piers may be designed for an allowable side friction of 200 psf to evaluate resistance to downward axial loads and 135 psf per foot depth for upward axial loads. The allowable side friction includes a factor of safety of 2 for downward loading and 3 for upward loading. The allowable side friction may be increased by one-third when considering loads of short duration such as wind or seismic loads. The spacing between adjacent piers should be equivalent to three pier diameters, or more to mitigate reduction due to group effects. Minor structures supported on shallow pier foundations should be designed for a total and differential settlement due to sustained loads of approximately ½ inch and ¼ inch over a horizontal distance of 30 feet.

10.3.3.2 Lateral Load Resistance

A lateral bearing pressure of 100 pounds per square foot (psf) per foot depth up to 1,500 psf may be used to evaluate resistance to lateral loads and overturning moments in accordance with Section 1806 of the 2016 CBC. The allowable lateral bearing pressure may be increased by one-third for wind or seismic load combinations and by an additional factor of two for structures that can accommodate ½ inch of lateral deflection of the top of the pier foundation.

Drilled piers in a row perpendicular to the direction of lateral loading do not need to be reduced for group effects where the center-to-center pier spacing is equivalent to 3 or more pier diameters. A reduction in the lateral resistance due to group effects should be considered for piers in a column parallel to the direction of loading where the center-to-center spacing between adjacent piers in the column is less than eight pier diameters. The reduction in lateral resistance due to group effects for piers in a column parallel to the direction of loading is influenced by the number of piers in the column and the spacing between piers. The efficiency or available lateral resistance per pier are presented in Table 10 for piers in a column parallel to the direction of loading at various spacings.

Table 10 – Pier Group Efficiency for Lateral Loading Parallel to Load

Piers in Column [1]	3B Pier Spacing [2]	5B Pier Spacing [2]	8B Pier Spacing [2]
2	60 percent	76 percent	100 percent
3	50 percent	70 percent	100 percent
4	45 percent	67 percent	100 percent

Notes:

[1] Number of piers in column parallel to the direction of the anticipated lateral load.

[2] Center to center pier spacing in direction of the anticipated load where 'B' is the pile diameter.

10.3.3.3 Construction Considerations

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.

10.4 Retaining Walls

Gravity and cantilever semi-gravity walls retaining up to 10 feet of soil may be designed for active or at-rest equivalent fluid earth pressures of 40 or 60 psf per foot depth, respectively, for level backfill conditions. Walls that yield or deflect may be designed for active earth pressures. Wall deflection equivalent to about 1 percent of wall height may be needed to reduce at-rest earth pressures to active earth pressures. For rising backfill conditions, the active or at-rest equivalent fluid earth pressures may be increased by 1 psf per foot depth per degree of inclination. Seismic earth pressures do not need to be considered for retaining walls with retained heights of 6 feet or less. For wall heights of more than 6 feet in retained height, an additional equivalent fluid pressure of 20 psf per foot depth may be used to evaluate the seismic earth pressure on yielding retaining walls.

Walls retaining level ground should be designed to resist construction or live load surcharges on the backfill. The lateral earth pressure due to a backfill surcharge of 240 psf should be a uniform horizontal surcharge of 80 psf for yielding conditions and 120 psf for at-rest conditions. An additional backfill surcharge and lateral earth pressure for adjacent footings should be considered, as applicable, where the adjacent footings bear above an imaginary plane that rises up and away from the bottom edge of the wall at a 2:1 (horizontal to vertical) gradient.

Hydrostatic pressures may be neglected, provided that suitable drainage of the retained soil is provided. The retained soil should be drained by weep holes or a subdrain at the base of the wall stem consisting 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 wall. Alternatively, 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 10-mil thick plastic membrane to the back surface of the wall.

Lateral forces may be resisted by friction at the base of the wall footing and passive earth pressure acting on the embedded wall, wall footing, or wall key, if present. Semi-gravity walls on level ground may be designed for a passive equivalent fluid lateral earth pressure of 300 psf per foot depth presuming a lateral deflection equivalent to 2 percent of the wall embedment depth to mobilize the passive condition. Passive earth pressure should be neglected to a depth of 1 foot below the ground surface when evaluating lateral load resistance where the ground surface is not covered by pavement or flatwork. Gravity and semi-gravity cantilever walls may be designed for a coefficient of friction of 0.30 to resist lateral loads and a net allowable bearing capacity of 3,000 psf for a 12-inch, or more, footing width and 12 inches, or more, of embedment below the adjacent grade. The allowable bearing capacity may be increased by one-third for seismic load combinations.

Walls should be designed to withstand a total static settlement of 1 inch with a differential of ½ inch over a 30-foot span. We recommend that the wall and the wall footing be reinforced. Footings should be designed by the structural engineer based on the anticipated loading and usage. We recommend that masonry briquettes or plastic chairs be used to aid in the correct placement of footing reinforcement. Refer to Section 10.5 for the recommended concrete cover over reinforcing steel.

10.5 Pavements and Flatwork

Recommendations for asphalt pavement, concrete pavement, and exterior flatwork are presented in the following sections. The design R-value used for evaluate the pavement sections was assumed based on the soil conditions encountered during our subsurface exploration. The pavement subgrade should be observed by the geotechnical engineer during grading to check that the exposed materials are consistent with the findings from our subsurface exploration and the support characteristics assumed for pavement design. Additional R-value testing may be needed, based on these observations, with subsequent revision to the pavement sections. Recommendations for preparation of subgrade are presented in Section 10.1.6.

Pavement sections were evaluated for a range of traffic indexes or loading conditions. The designer may interpolate between the values provided once a traffic index or loading condition has been selected.

10.5.1 Asphalt Pavement

Ninyo & Moore conducted an analysis to evaluate appropriate asphalt pavement structural sections, following the methodology presented in the Highway Design Manual (Caltrans, 2016). Alternative sections were evaluated. The pavement sections were designed for a 20-year service life, presuming that periodic maintenance, including crack sealing and resurfacing will be performed during the service life of the pavement. Premature deterioration may occur without periodic maintenance. Our recommendations for the pavement sections are presented in Table 11.

Paving operations and base preparation should be observed and tested by Ninyo & Moore. Subgrade enhancement geotextiles, where utilized, should be rolled out flat and tight, without folds or wrinkles, over prepared subgrade in the direction of travel. The geotextile should be pinned to the subgrade with nails and washers or u-shaped sod staples. Adjacent rolls should overlap 12 inches or more. Abutting rolls should overlap in the direction of fill placement to reduce the potential for peeling of the geotextile during fill placement. Aggregate base fill should be pushed over the geotextile into position and compacted. To reduce the potential for displacement of the geotextile or deterioration of the subgrade, construction equipment should not operate on the geotextile with 6 inches of aggregate base cover.

Table 11 – Asphalt Concrete Pavement Structural Sections

Design R-Value	Traffic Index	Alternative 1	Alternative 2	Alternative 3
5	3	5 inches AC	2 inches AC 6 inches AB	2 inches AC 4 inches AB SEG
5	5	7½ inches AC	3 inches AC 10 inches AB	3 inches AC 8 inches AB SEG
5	7	11 inches AC	4 inches AC 16 inches AB	4 inches AC 12 inches AB SEG

Notes:

[1] AC is Type A, Dense-Graded Hot Mix Asphalt complying with Caltrans Standard Specification 39-2 (2015).
[2] AB is Class II Aggregate Base complying with Caltrans Standard Specification 26-1.02 (2015).
[3] SEG is subgrade enhancement geotextile such as Mirafi 600X.

Aggregate base for pavement should be placed in lifts of no more than 8 inches in loose thickness and compacted as per Section 10.1.8. Asphalt concrete should be placed and compacted as per Section 10.1.8. Pavements should be sloped so that runoff is diverted to an appropriate collector (concrete gutter, swale, or area drain) to reduce the potential for ponding of water on the pavement. Concentration of runoff over asphalt pavement should be discouraged.

10.5.2 Concrete Pavement

Portland cement concrete may be used in lieu of asphalt concrete for the proposed pavement sections. Our recommended pavement sections based on methodologies developed by the Portland Cement Associate (PCA) are presented in Table 12 for a 20-year design period with appropriate periodic maintenance. The recommended sections presume that the concrete will have a 28-day flexural strength of 600 psi or an equivalent compressive strength of 5,000 psi at 28 days.

Table 12 – Portland Cement Concrete Pavement Sections

Loading Condition[1]	Equivalent Traffic Index	Design Period	Subgrade Modulus[2]	Concrete Pavement Section
233,000 Annual Vehicles including: 52 annual 48-kip garbage trucks 57 annual 35-kip, 40-foot buses 1 annual 75-kip emergency vehicle	9	20 years	50 pci	6 inches PCC[3] 12 inches AB[4] or 6 inches AB on 12 inches TS[5]
230,000 Annual Vehicles including: 52 annual 48-kip garbage trucks 3,100 annual 35-kip, 40-foot buses 1 annual 75-kip emergency vehicle	9	20 years	50 pci	7 inches PCC[3] 12 inches AB[4] or 6 inches AB on 12 inches TS[5]
5181,000 Annual Vehicles including: 52 annual 48-kip garbage trucks 6,200 annual 35-kip, 40-foot buses 1 annual 75-kip emergency vehicle	9	20 years	50 pci	8 inches PCC[3] 12 inches AB[4] or 6 inches AB on 12 inches TS[5]

Notes:

[1] Assumes 24-ton garbage truck with 12-kip single and 36-kip tandem axles; 38-kip bus with 10- and 25-kip single axles; and a 75-kip ladder truck with 23-kip single and 52 kip tandem axles.

[2] Modulus of Subgrade Reaction in pounds per cubic inch (pci).

[3] PCC is Portland Cement Concrete complying with Caltrans Standard Specification Section 90 (2015).

[4] AB is Class II Aggregate Base complying with Caltrans Standard Specification Section 26 (2015).

[5] TS is chemically treated subgrade consistent with the recommendations in Section.dfhdt.

Appropriate jointing of the concrete pavement can reduce the random occurrence of cracks. Joints should be laid out in a regular square pattern. Contraction, construction, and isolation joints should be detailed and constructed in accordance with the guidelines of the American Concrete Institute (ACI) Committee 302 (Manual of Concrete Practice [MCP], 2016). We recommend spacing contraction joints at 12 feet or less for 6-inch thick slabs and 16 feet or less for 8-inch thick slabs. Contraction joints formed by premolded inserts, grooving plastic concrete, or saw-cutting at initial hardening, should extend to a depth equivalent to 25 percent of the slab thickness and 1 inch or more for thin slabs. Contraction joints should be reinforced with smooth, 1-inch diameter, 14-inch long dowels placed across the joint at mid-slab height and spaced at 12 inches on center along the joint. However, contraction joints that are parallel and adjacent to pavement edges that are unrestrained by curbs or adjacent pavements should instead be reinforced with 30-inch long, No. 6 deformed steel bars placed across the joint at mid-slab height and spaced at 12 inches on center along the joint. Isolation joints subject to traffic loading should be thickened by 20 percent of the nominal thickness at the edge of the pavement with a 40:1 taper (horizontal to vertical) to the nominal slab thickness. Construction joints subject to traffic loading should be reinforced with smooth dowels as for contraction joints. Construction joints within the middle third of the typical joint spacing pattern should be reinforced with 30-inch long, No. 6 deformed steel bars placed across the joint near the middle of the slab and spaced at 30 inches on center. To reduce the potential for subsurface water intrusion into the subgrade and base layer, curbs or similar cutoff devices should be provided and joints should include a formed or sawcut reservoir for placement of foam backer rod and recessed, self-leveling silicone sealant. Periodic maintenance of the pavement should include sealing cracks that develop and replacement of joint sealant as needed.

Distributed reinforcing steel may be placed 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 or isolation joints and should consist of No. 3 deformed bars at 18 inches on center, both ways in the upper portion of the slab. Masonry blocks or plastic chairs should be used to maintain the position of the reinforcement during concrete placement with 1½ inches of concrete cover over the steel.

10.5.3 Exterior Flatwork

Remedial grading should be performed below exterior flatwork per Section 10.1.4. Concrete walkways and other exterior flatwork not subject to vehicular loading should be 4 inches thick (or more) over 6 inches of aggregate base. The concrete thickness should be increased to 6 inches at driveways. 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.

10.6 Concrete

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).

10.7 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.

10.8 Surface Drainage and Site Maintenance

Surface drainage on the site should generally be provided so that water is diverted away from structures and is not permitted to pond. Positive drainage should be established adjacent to structures to divert surface water to an appropriate collector (graded swale, v-ditch, or area drain) with a suitable outlet. 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.

10.9 Review of Construction Plans

The recommendations provided in this report are based on preliminary design information for the proposed construction. We recommend that a copy of the plans be provided to Ninyo & Moore for review before bidding to check the interpretation of our recommendations and that the designed improvements are consistent with our assumptions. It should be noted that, upon review of these documents, some recommendations presented in this report might be revised or modified to meet the project requirements.

10.10 Construction Observation and Testing

The recommendations provided in this report are based on subsurface conditions encountered in relatively widely spaced exploratory borings. During construction, the geotechnical engineer or his representative in the field should be allowed to check the exposed subsurface conditions. During construction, the geotechnical engineer or his representative should be allowed to:

- Observe preparation and compaction of subgrade.
- Observe mitigation of unsuitable materials by excavation.
- Check and test imported materials prior to use as fill.
- Observe placement and compaction of fill and aggregate base.
- Perform field density tests to evaluate fill and subgrade compaction.
- Observe foundation excavations for bearing materials and cleaning prior to placement of reinforcing steel and concrete.
- Observe condition of water vapor retarding system prior to concrete placement.

The recommendations provided in this report assume that Ninyo & Moore will be retained as the geotechnical consultant during the construction phase of the project. If another geotechnical consultant is selected, we request that the selected consultant provide a letter to the architect and the owner (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the recommendations contained in this report.

11 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. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

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.

12 REFERENCES

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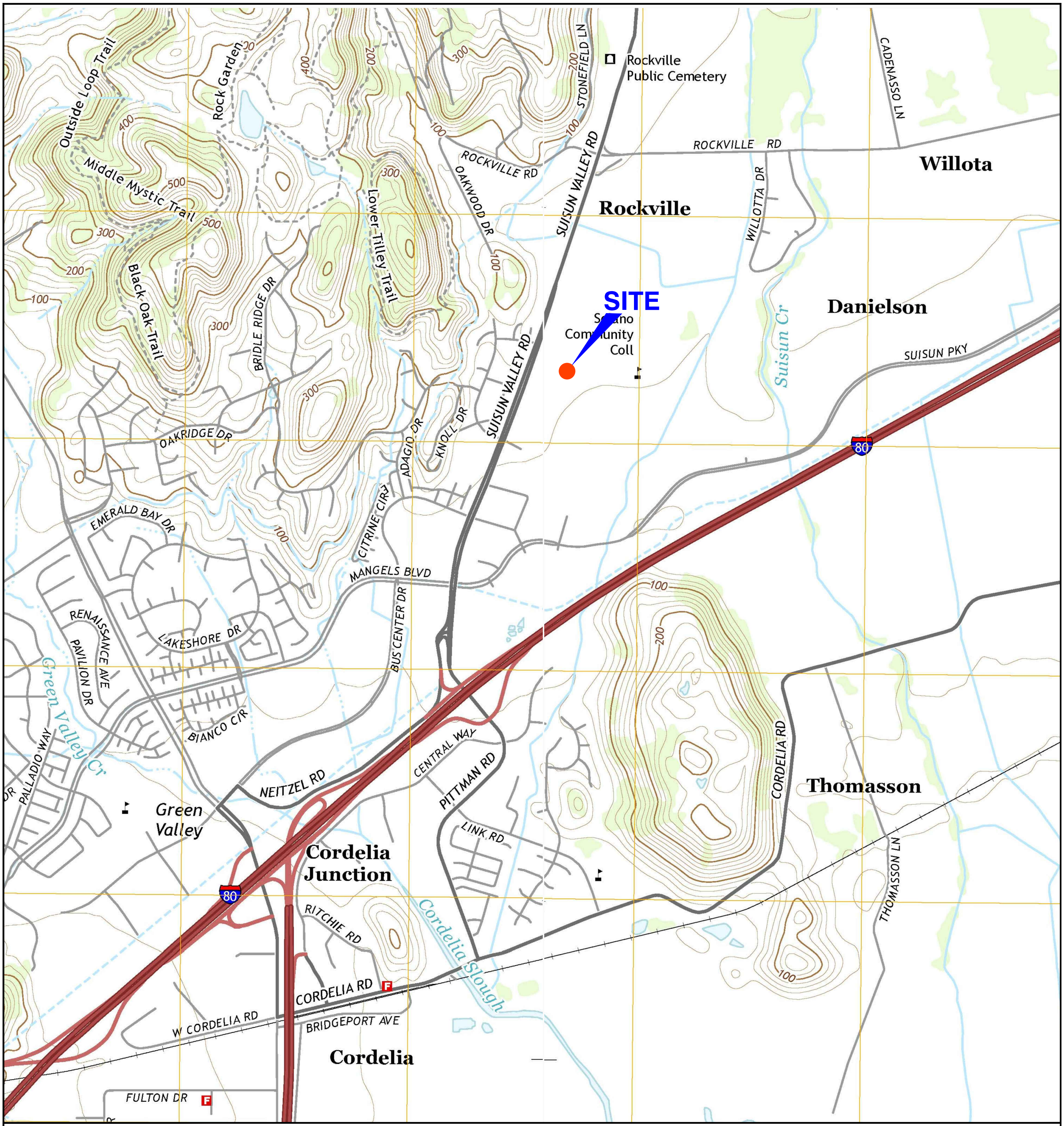
FIGURES



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

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FIGURE 1

SITE LOCATION

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS

4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA

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Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

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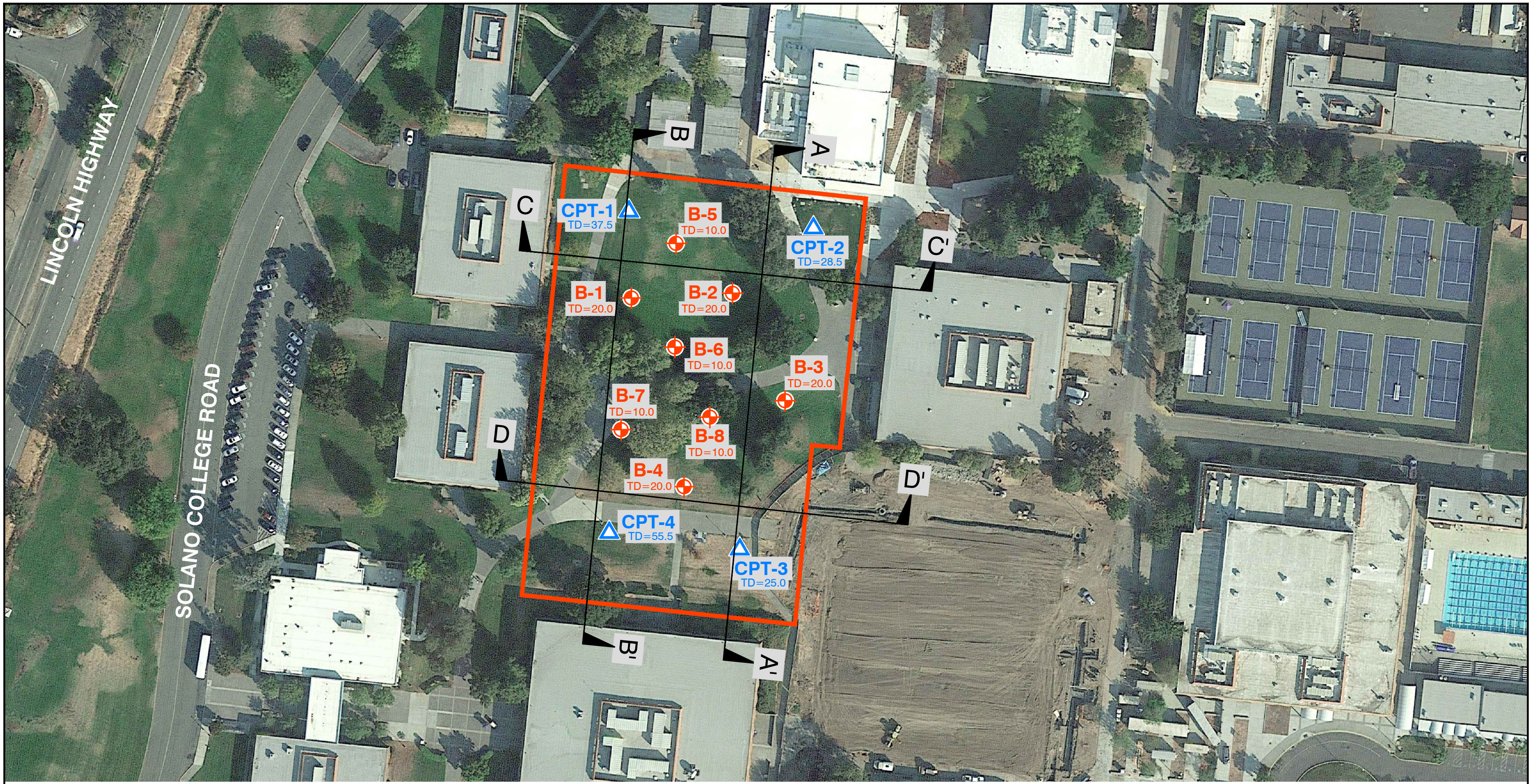

LEGEND

🟥	SITE BOUNDARY	
B-8 TD=10.0 ⊕	BORING; TD=TOTAL DEPTH IN FEET	
CPT-4 TD=25.0 △	CONE PENETRATION TEST (CPT) TD=TOTAL DEPTH IN FEET	
D D'	CROSS SECTION LOCATION	

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: GOOGLE EARTH, 2017.

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FIGURE 2

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Geotechnical & Environmental Sciences Consultants

EXPLORATION LOCATIONS

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS
4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA

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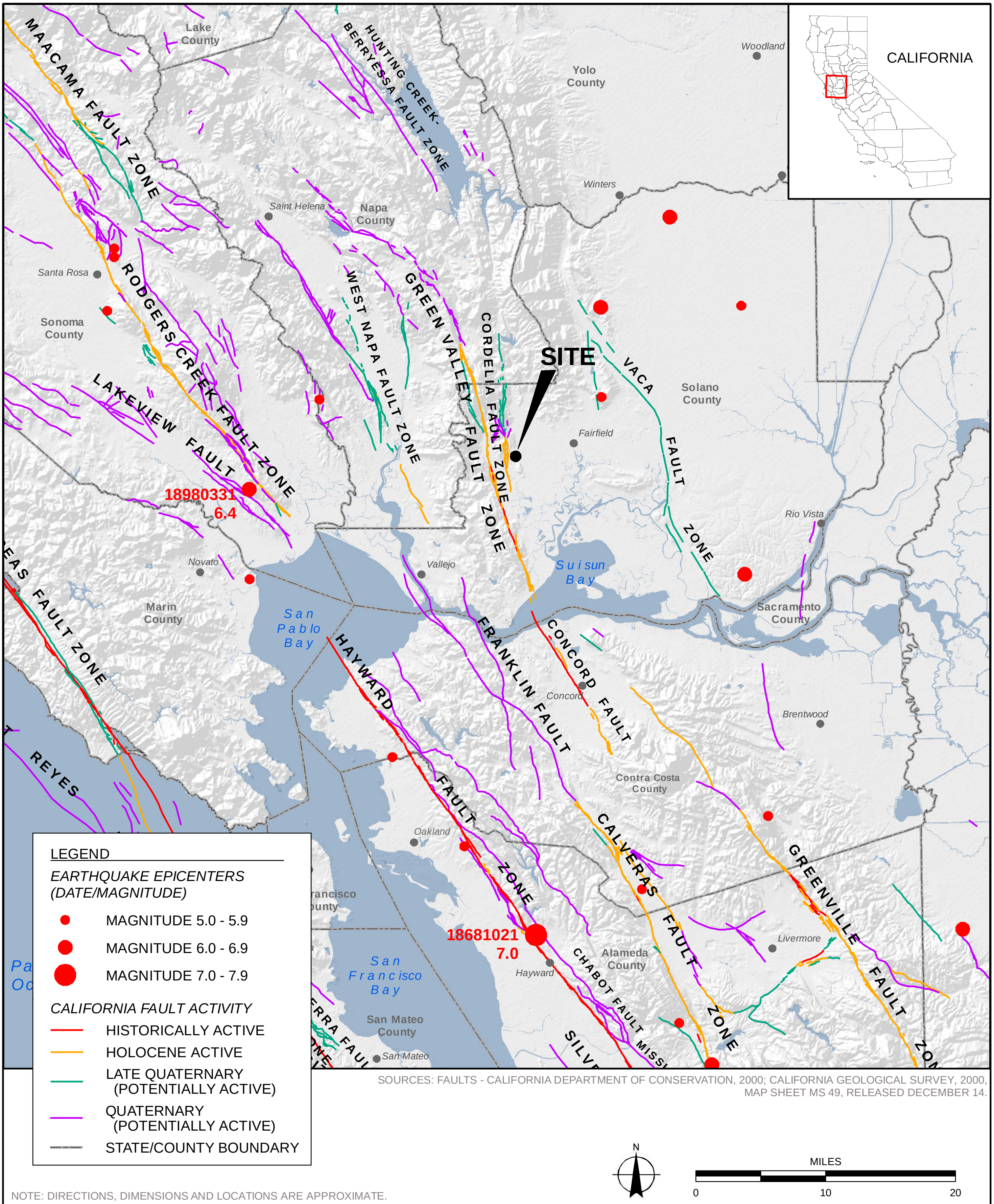

SOURCES: FAULTS - CALIFORNIA DEPARTMENT OF CONSERVATION, 2000; CALIFORNIA GEOLOGICAL SURVEY, 2000, MAP SHEET MS 49, RELEASED DECEMBER 14.

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 3

FAULT LOCATIONS AND EARTHQUAKE EPICENTERS

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS
4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA

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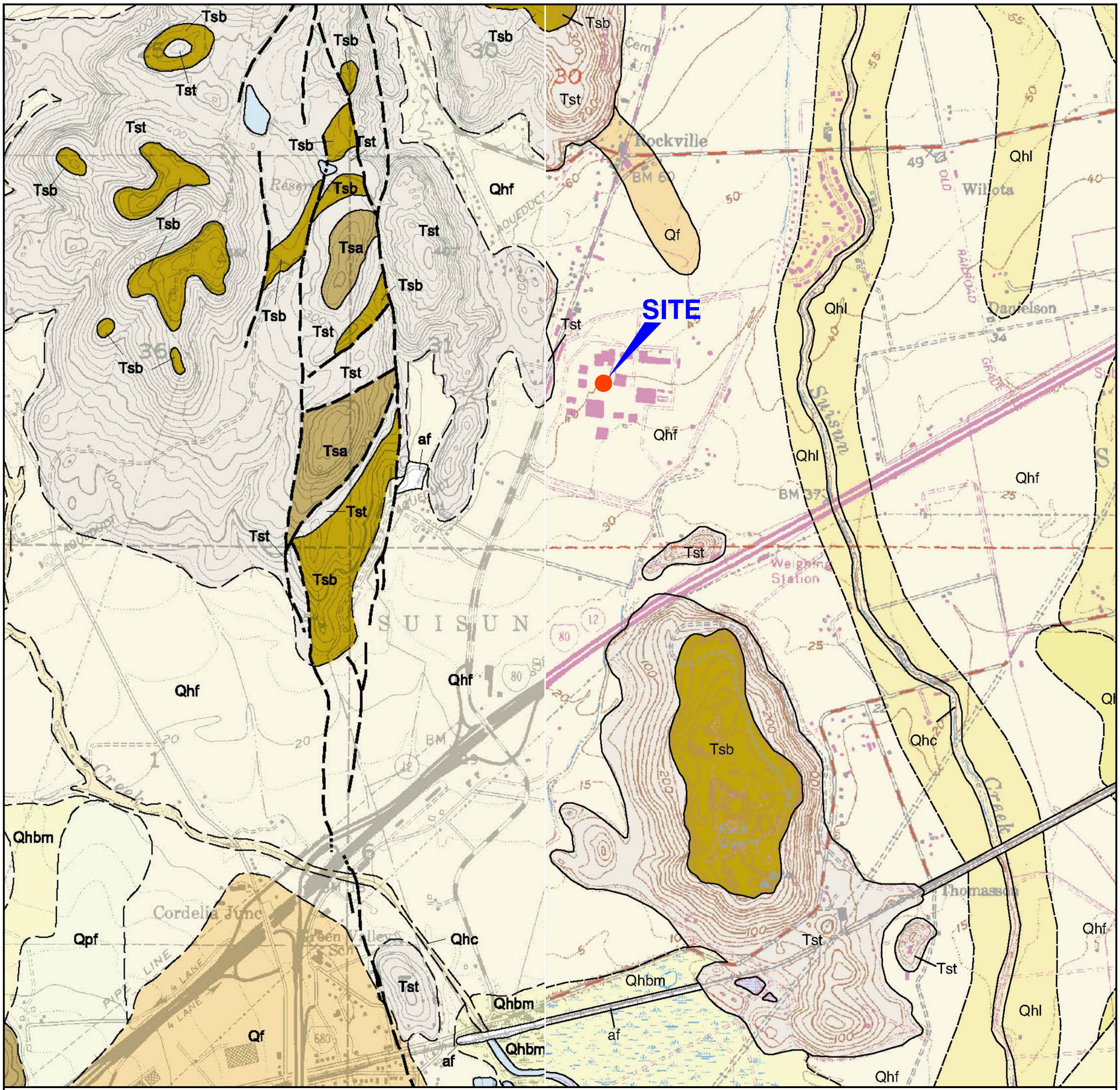

REFERENCE: STEPHEN P. BEZORE, DAVID L. WAGNER AND JANET M. SOWERS, 1998.

LEGEND

Qhf	ALLUVIAL FAN DEPOSITS (HOLOCENE)
Qhl	ALLUVIAL LEVEE DEPOSITS (HOLOCENE)
Qf	ALLUVIAL FAN DEPOSITS (PLEISTOCENE/HOLOCENE)
Tst	ASH FLOW TUFF (PLIOCENE)
Tsa	ANDESITE (PLIOCENE)
Tsb	BASALT (PLIOCENE)
—— - -	GEOLOGIC CONTACT

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 4

REGIONAL GEOLOGY

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS
4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

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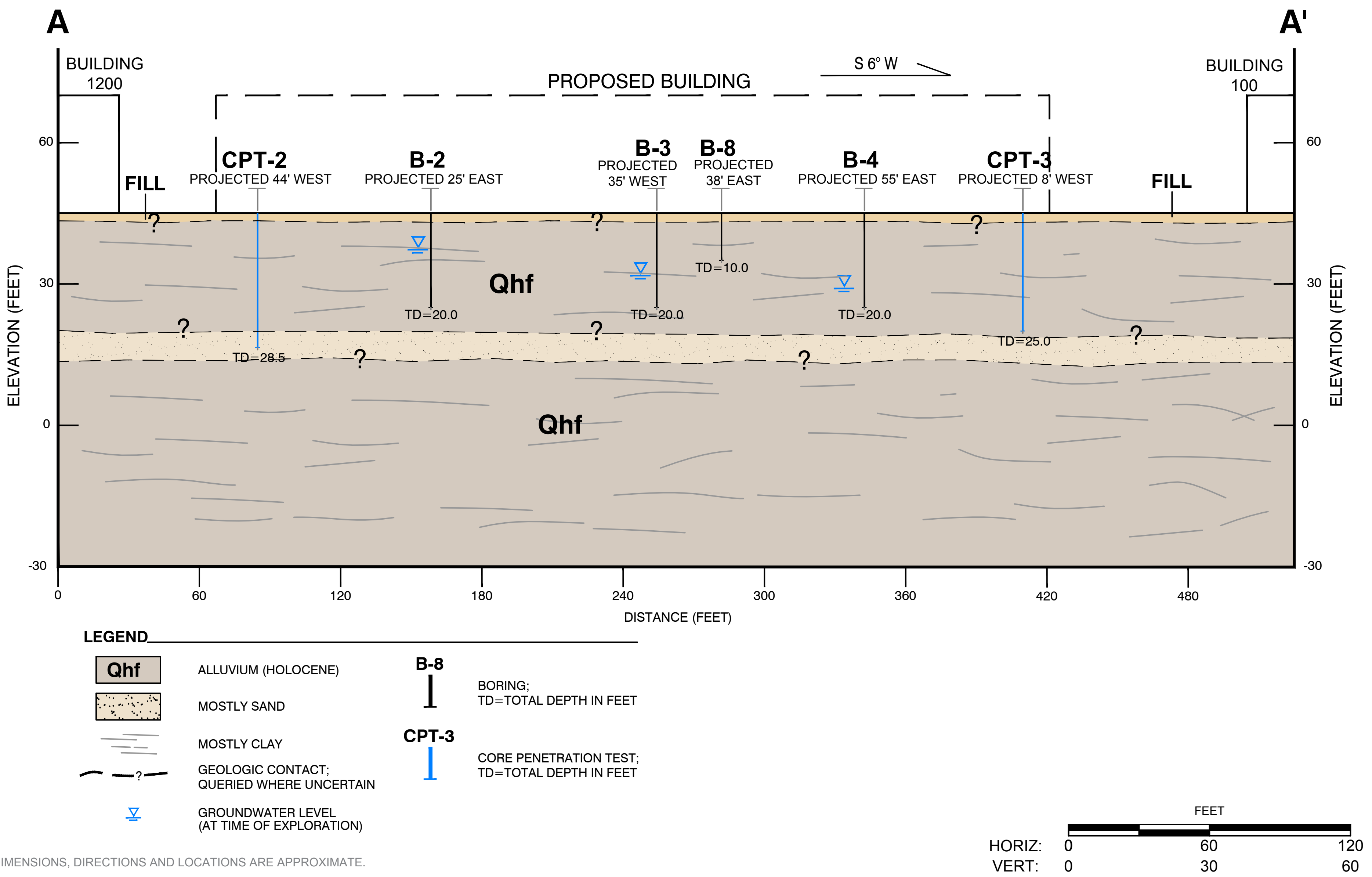

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 5

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CROSS SECTION A-A'

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS
4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA

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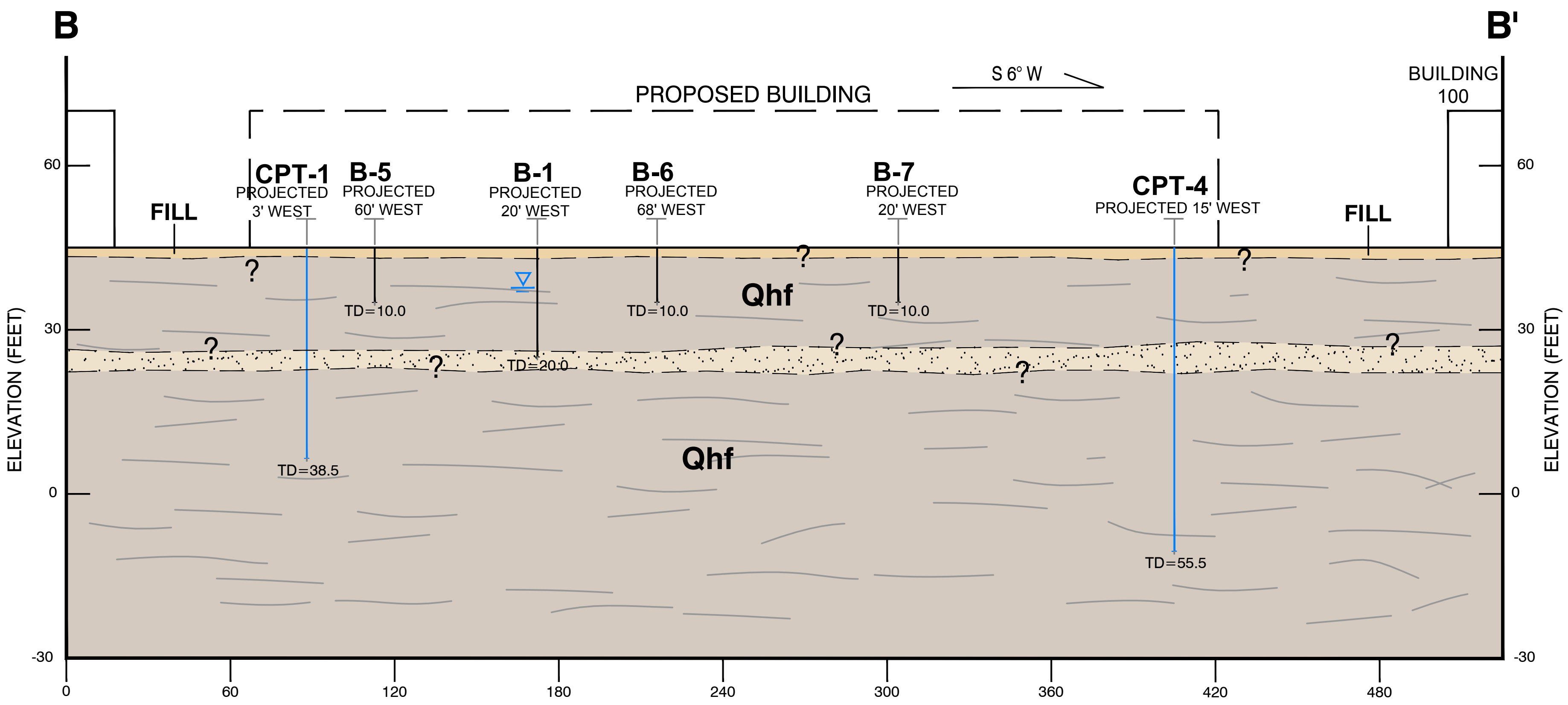

LEGEND

Qhf	ALLUVIUM (HOLOCENE)
▨	MOSTLY SAND
≡	MOSTLY CLAY
—?—	GEOLOGIC CONTACT; QUERIED WHERE UNCERTAIN
▽	GROUNDWATER LEVEL (AT TIME OF EXPLORATION)
B-7	BORING; TD=TOTAL DEPTH IN FEET
CPT-4	CORE PENETRATION TEST; TD=TOTAL DEPTH IN FEET

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

FEET

HORIZ: 0 60 120

VERT: 0 30 60

FIGURE 6

CROSS SECTION B-B'

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS

4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA

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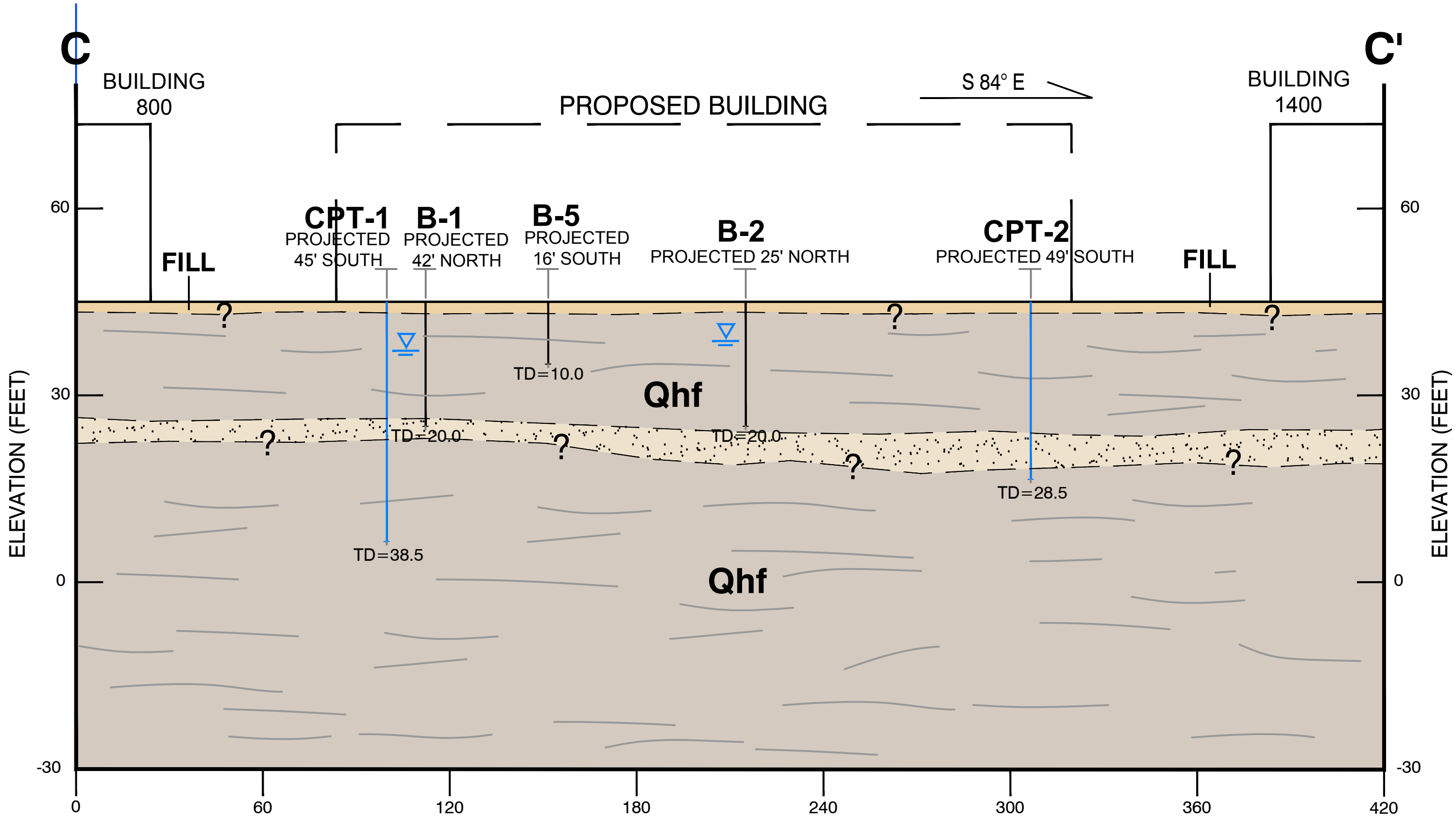

LEGEND

Qhf	ALLUVIUM (HOLOCENE)	**B-5**	BORING; TD=TOTAL DEPTH IN FEET
▨	MOSTLY SAND	**CPT-2**	CORE PENETRATION TEST; TD=TOTAL DEPTH IN FEET
≡	MOSTLY CLAY		
— —?—	GEOLOGIC CONTACT; QUERIED WHERE UNCERTAIN		
▽	GROUNDWATER LEVEL (AT TIME OF EXPLORATION)		

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

FEET

HORIZ:	0	60	120
VERT:	0	30	60

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FIGURE 7

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CROSS SECTION C-C'

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS
4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA

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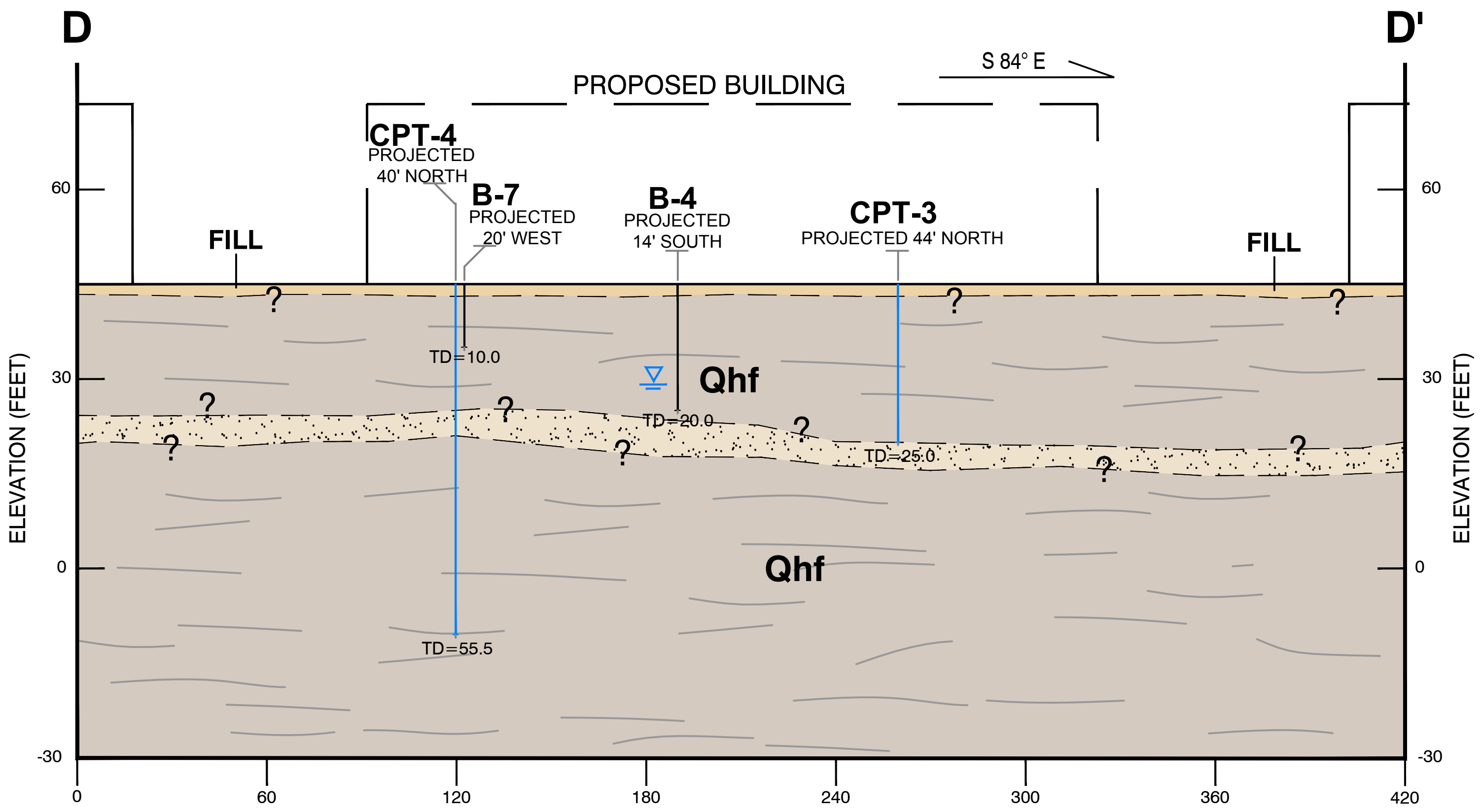

LEGEND

Qhf	ALLUVIUM (HOLOCENE)	B-7	BORING; TD=TOTAL DEPTH IN FEET
▨	MOSTLY SAND	CPT-4	CORE PENETRATION TEST; TD=TOTAL DEPTH IN FEET
≡	MOSTLY CLAY		
—?—	GEOLOGIC CONTACT; QUERIED WHERE UNCERTAIN		
▽	GROUNDWATER LEVEL (AT TIME OF EXPLORATION)		

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

FEET

HORIZ: 0 60 120
VERT: 0 30 60

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FIGURE 8

CROSS SECTION D-D'

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS
4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA

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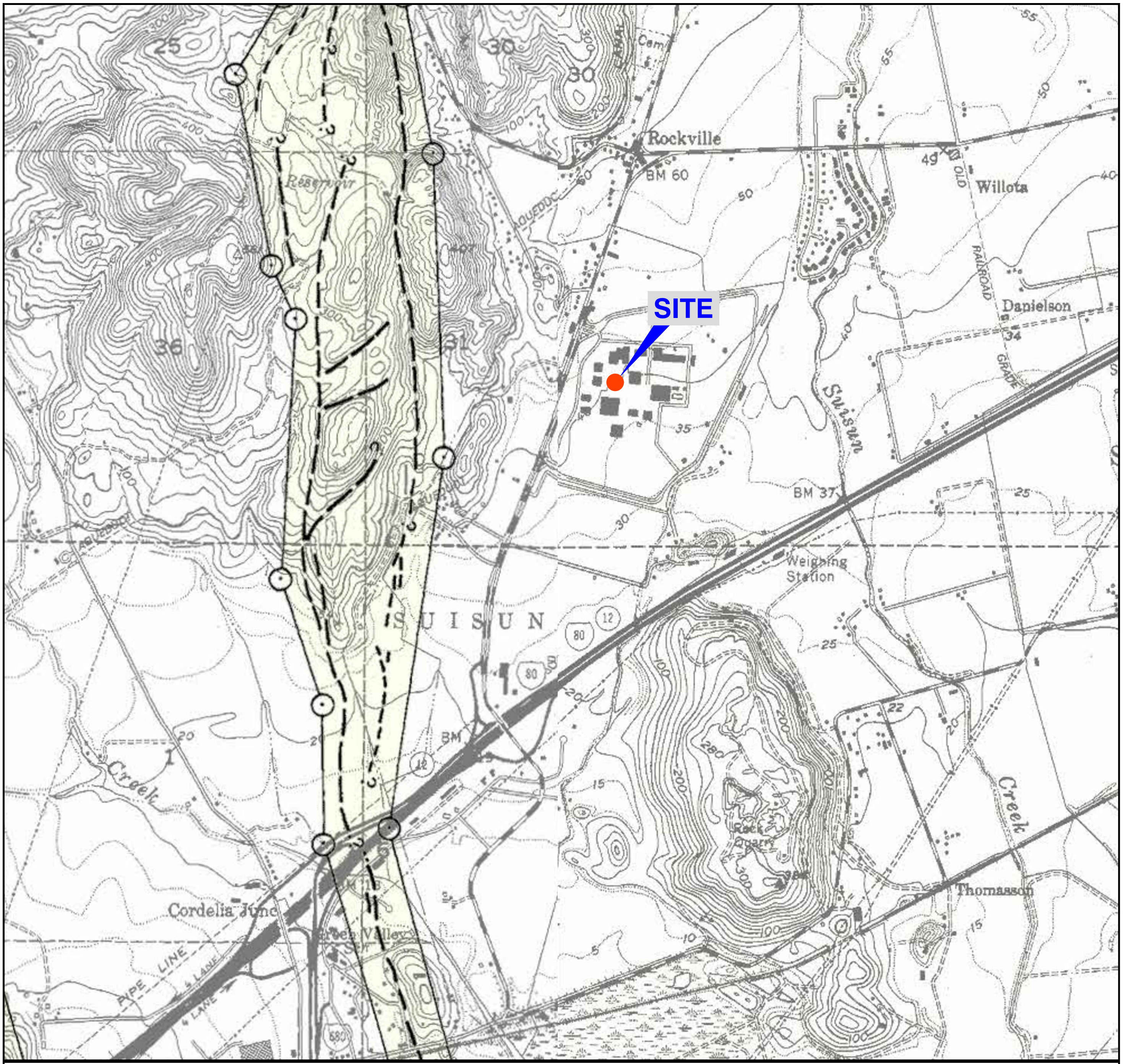
Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DRAFT



REFERENCE: STATE OF CALIFORNIA SPECIAL STUDIES ZONES, 1993.

LEGEND

Potentially Active Faults

1906 C

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

Special Studies Zone Boundaries

These are delineated as straight-line segments that connect encircled turning points so as to define special studies zone segments.

Seaward projection of zone boundary.

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 9

SEISMIC HAZARD ZONES

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS
4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA

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Geotechnical & Environmental Sciences Consultants

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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 methods.

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 methods.

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.
15						▨	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
20								The total depth line is a solid line that is drawn at the bottom of the boring.

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BORING LOG

DRAFT

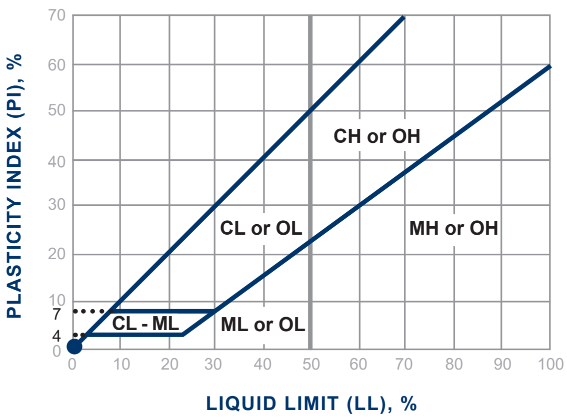
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions	
			Group Symbol	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		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	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		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	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

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DATE DRILLED 11/02/17 BORING NO. B-1

GROUND ELEVATION 47' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 4.5" SSA, CME-54 (Geo-Ex), 3" HA top 5'

DRIVE WEIGHT 140 LBS (wireline) DROP 30 INCHES

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	■					▨	CL	FILL: Grayish brown, moist, firm, lean CLAY; trace organics.
	■					▨	CL	ALLUVIUM: Brown, moist, firm, lean CLAY.
		■	19			▨		
		■	18	24.2 ▽	100.1	▨		
10						▨		Wet.
		■	13	30.3	89.8	▨		
						▒	SC	Olive gray, wet, medium dense, clayey SAND; trace gravel.
		■	31			▨	CL	Brown, wet, stiff, lean CLAY; trace sand.
20								Total Depth = 20 feet. Backfilled with cement grout using the tremie method shortly after drilling. Notes: Groundwater was first encountered at approximately 9.5 feet into the borehole. It 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.
30								
40								

FIGURE A- 1

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DATE DRILLED 11/02/17 BORING NO. B-2

GROUND ELEVATION 46' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 4.5" SSA, CME-54 (Geo-Ex), 3" HA top 5'

DRIVE WEIGHT 140 LBS (wireline) DROP 30 INCHES

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	■					▨	CL	FILL: Brown, moist, stiff, lean CLAY; trace organics; trace sand.
	■					▨	CL	ALLUVIUM: Brown, moist, stiff, lean CLAY.
		■	26			▨		Very stiff.
				▽		▨		
		■	25			▨		Wet; trace sand, trace gravel.
10						▨		
		■	13			▨		Stiff.
		■	23			▨		Very stiff; little gravel.
20								Total Depth = 20 feet. Backfilled with cement grout using the tremie method shortly after drilling. Notes: Groundwater was first encountered at approximately 7 feet into the borehole. It 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.
30								
40								

FIGURE A- 2

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DATE DRILLED 11/02/17 BORING NO. B-3

GROUND ELEVATION 47' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 4.5" SSA, CME-54 (Geo-Ex), 3" HA top 5'

DRIVE WEIGHT 140 LBS (wireline) DROP 30 INCHES

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	■					▨	CL	<u>FILL:</u> Brown, moist, firm, lean CLAY; trace organics.
	■					▨	CL	<u>ALLUVIUM:</u> Gray, moist, very stiff, lean CLAY. Firm.
		■	25			▨		Very stiff.
		■	15			▨		Stiff.
10						▨		
		■	19	27.9 ▽	91.5	▨		Wet; trace organics.
		■	11	33.8	83.1	▨		Trace sand.
20								Total Depth = 20 feet. Backfilled with cement grout using the tremie method shortly after drilling.
30								
40								

<u>Notes:</u>

Groundwater was first encountered at approximately 14.5 feet into the borehole. It 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.

FIGURE A- 3

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DATE DRILLED 11/02/17 BORING NO. B-4

GROUND ELEVATION 48' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 4.5" SSA, CME-54 (Geo-Ex), 3" HA top 5'

DRIVE WEIGHT 140 LBS (wireline) DROP 30 INCHES

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	■					▨	CL	FILL: Brown, dry, firm, lean CLAY; trace organics, little sand.
	■					▨	CL	ALLUVIUM: Brown, moist, firm, lean CLAY. Cobble.
						▨		Very stiff.
		■	24			▨		
		■	17	23.5	97.9	▨		Stiff.
10						▨		
						▨		Yellowish brown.
		■	17			▨		
				▽		▨		
		■	10	32.3	83.9	▨		Firm; wet.
20								Total Depth = 20 feet.
30								
40								

Total Depth = 20 feet.

Backfilled with cement grout using the tremie method shortly after drilling.

Notes:

Groundwater was first encountered at approximately 16.5 feet into the borehole. It 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.

FIGURE A- 4

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DATE DRILLED 11/02/17 BORING NO. B-5
GROUND ELEVATION 45' ± (MSL) SHEET 1 OF 1
METHOD OF DRILLING 4.5" SSA, CME-54 (Geo-Ex), 3" HA top 5'
DRIVE WEIGHT 140 LBS (wireline) DROP 30 INCHES
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	■					▨	CL	FILL: Brown, moist, stiff, lean CLAY; trace organics, trace sand.
	■					▨	CL	ALLUVIUM: Brown, moist, stiff, lean CLAY; trace sand.
		■	16	22.3	98.5	▨		
		■	13			▨		
10								Total Depth = 10 feet. Backfilled with cement grout shortly after drilling. Notes: Groundwater, though not encountered at the time of drilling, it 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.
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FIGURE A- 5

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DATE DRILLED 11/02/17 BORING NO. B-6
GROUND ELEVATION 47' ± (MSL) SHEET 1 OF 1
METHOD OF DRILLING 4.5" SSA, CME-54 (Geo-Ex), 3" HA top 5'
DRIVE WEIGHT 140 LBS (wireline) DROP 30 INCHES
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	■					▨	CL	FILL: Brown, moist, stiff, lean CLAY; trace sand.
	■					▨	CL	ALLUVIUM: Brown, moist, stiff, lean CLAY with sand.
		■	23			▨		Very stiff.
		■	16			▨		Stiff.
10								Total Depth = 10 feet. Backfilled with cement grout shortly after drilling.
20								
30								
40								

Notes:

Groundwater, though not encountered at the ime of 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.

FIGURE A- 6

SOLANO COMMUNITY COLEGE - FAIRFIELD CAMPUS
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DATE DRILLED 11/02/17 BORING NO. B-7

GROUND ELEVATION 49' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 4.5" SSA, CME-54 (Geo-Ex), 3" HA top 5'

DRIVE WEIGHT 140 LBS (wireline) DROP 30 INCHES

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	■					▨	CL	<u>FILL:</u> Brown, moist, firm, lean CLAY; trace organics.
	■					▨	CL	<u>ALLUVIUM:</u> Brown, moist, firm, lean CLAY. Gray, little sand.
		■	40	19.3	101.7	▨		Very stiff.
		■	38			▨		Brown.
10								Total Depth = 10 feet. Backfilled with cement grout shortly after drilling. <u>Notes:</u> Groundwater, though not encountered at the ime of 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.
20								
30								
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FIGURE A- 7

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4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA

DATE DRILLED 11/02/17 BORING NO. B-8

GROUND ELEVATION 48' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 4.5" SSA, CME-54 (Geo-Ex), 3" HA top 5'

DRIVE WEIGHT 140 LBS (wireline) DROP 30 INCHES

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	■					▨	CL	FILL: Brown, dry, firm, lean CLAY; trace organics.
	■					▨	CL	ALLUVIUM: Gray, moist, very stiff, lean CLAY. Stiff.
		■	50	17.7	84.4	▨		Trace sand.
		■	15			▨		Brown.
10								Total Depth = 10 feet. Backfilled with cement grout shortly after drilling. Notes: Groundwater, though not encountered at the ime of 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.
20								
30								
40								

FIGURE A- 8

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APPENDIX B

Cone Penetration Testing

APPENDIX B

CONE PENETRATION TESTING

Field Procedure for Cone Penetration Testing

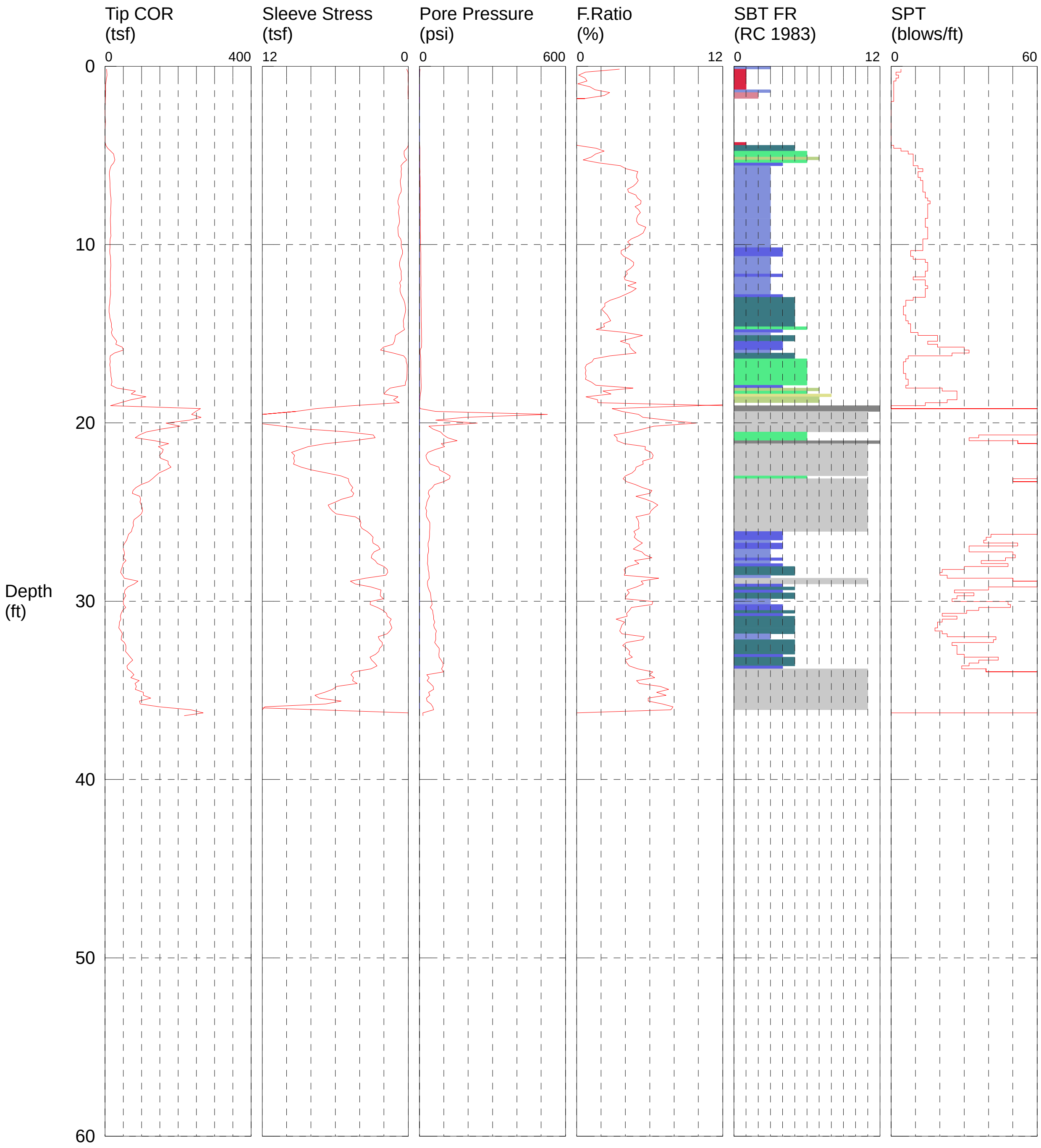
A penetrometer with a conical tip having an apex angle of 60 degrees and a cone base area of 10 square centimeters was hydraulically pushed through the soil using the reaction mass of a 20-ton rig at a constant rate of about 20 millimeter per second in accordance with ASTM D 5778. The penetrometer was 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.

SOUNDING

SOUNDING
CUSTOMER: Taber Drilling
OPERATOR: Tim
CONE ID: DDG1361
LOCATION:

JOB NUMBER: 403147001
HOLE NUMBER: CPT-1b
TEST DATE: 11/2/2017 8:34:59 AM
COMMENT: Auto Enhance On
COMMENT: Filter On

COMMENT:
GPS (LAT,LON,ALT): 0.00,0.00,0.0
LOCATION:
LOCATION:
LOCATION:



🟥 1 sensitive fine grained
🟥 2 organic material
🟦 3 clay

🟦 4 silty clay to clay
🟩 5 clayey silt to silty clay
🟩 6 sandy silt to clayey silt

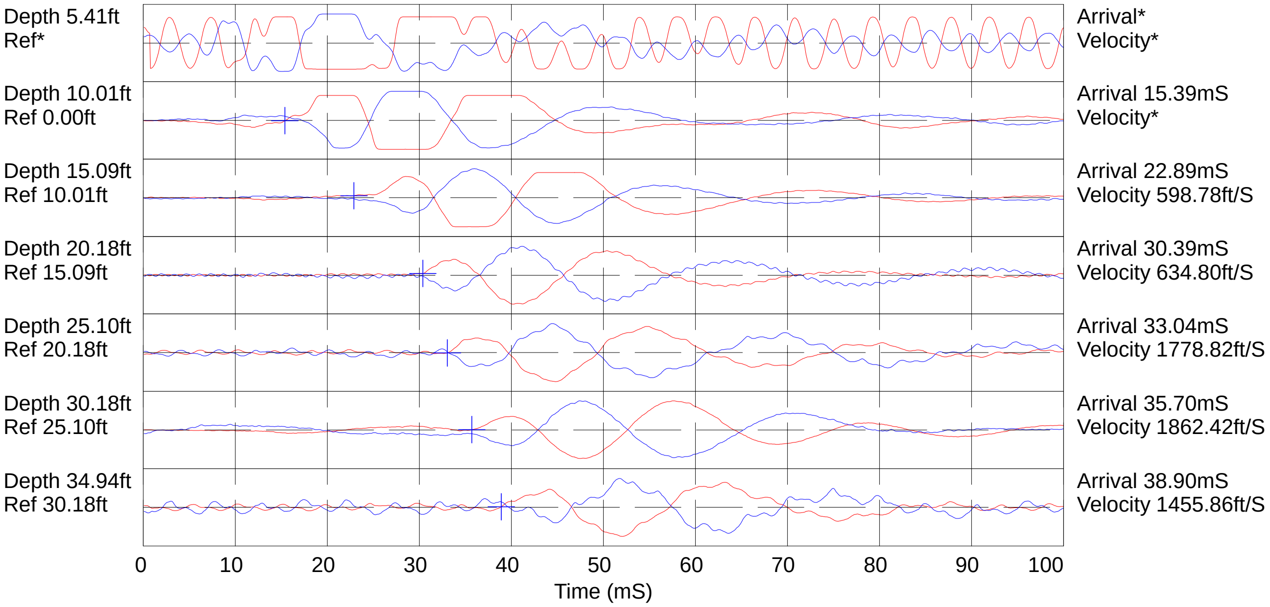
🟩 7 silty sand to sandy silt
🟨 8 sand to silty sand
🟫 9 sand

🟧 10 gravelly sand to sand
⬜ 11 very stiff fine grained (*)
⬛ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

DRAFT

SEISMIC TEST



Hammer to Rod String Distance (ft): 6.56
* = Not Determined

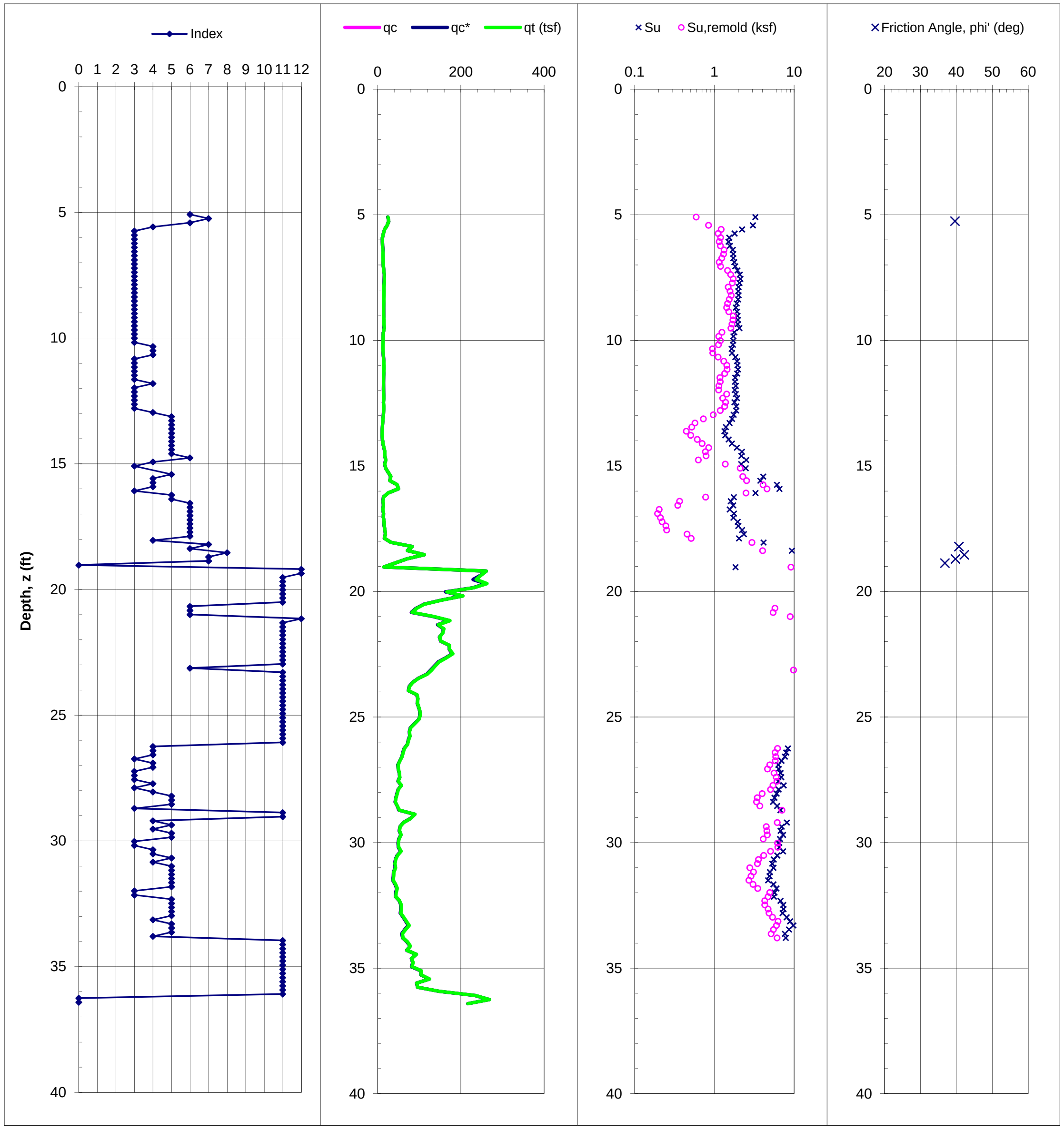
COMMENT:

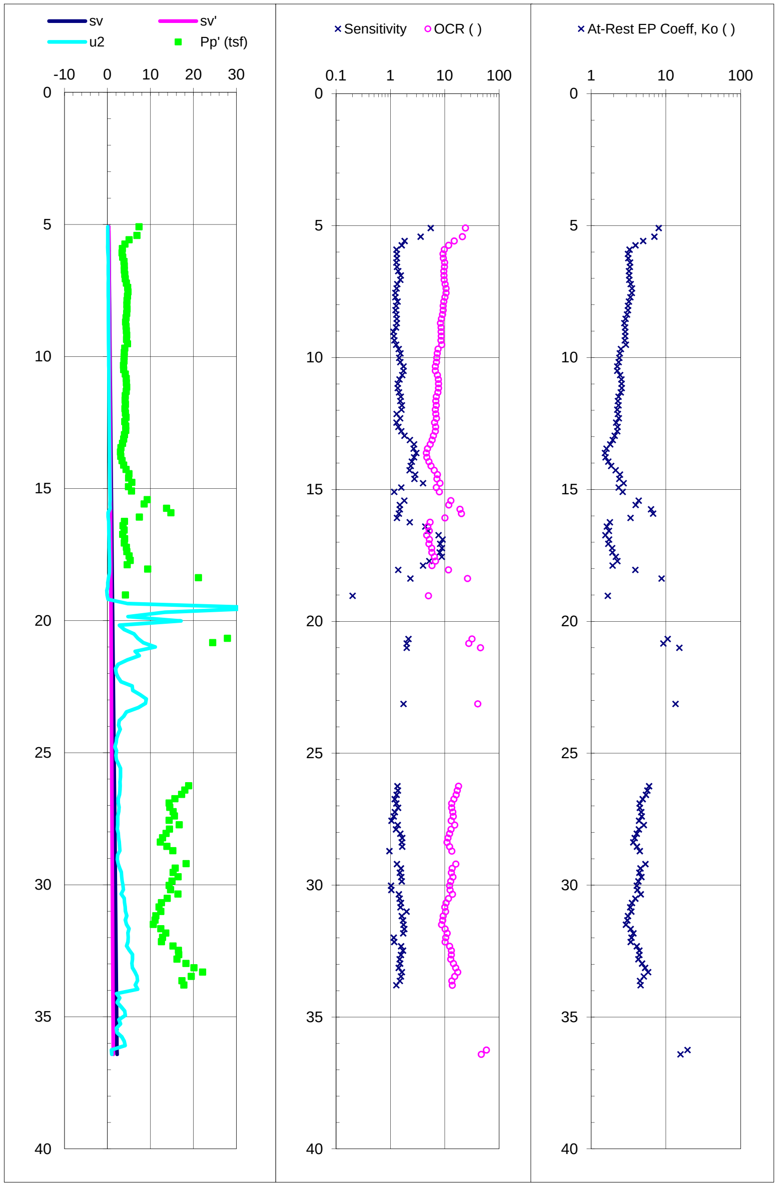
CONE PENETRATION TEST DATA

CPT Sounding: CPT-1
Location: CPT-1

Elevation at top of sounding (ft, MSL)	44
Depth to GWT during CPT evaluation (ft)	8
Cone Diameter, dc (mm)	35.7
Net End Area Ratio ( )	0.80
Atmospheric Pressure (tsf)	1

Project Name:	Solano Community College		
Project Number:	403147001		
Calculation By:	RH	Date:	11/7/2017
Checked By:	TPS	Date:	11/10/2017





File: cpt data vers2.xls
Sheet: CPT1

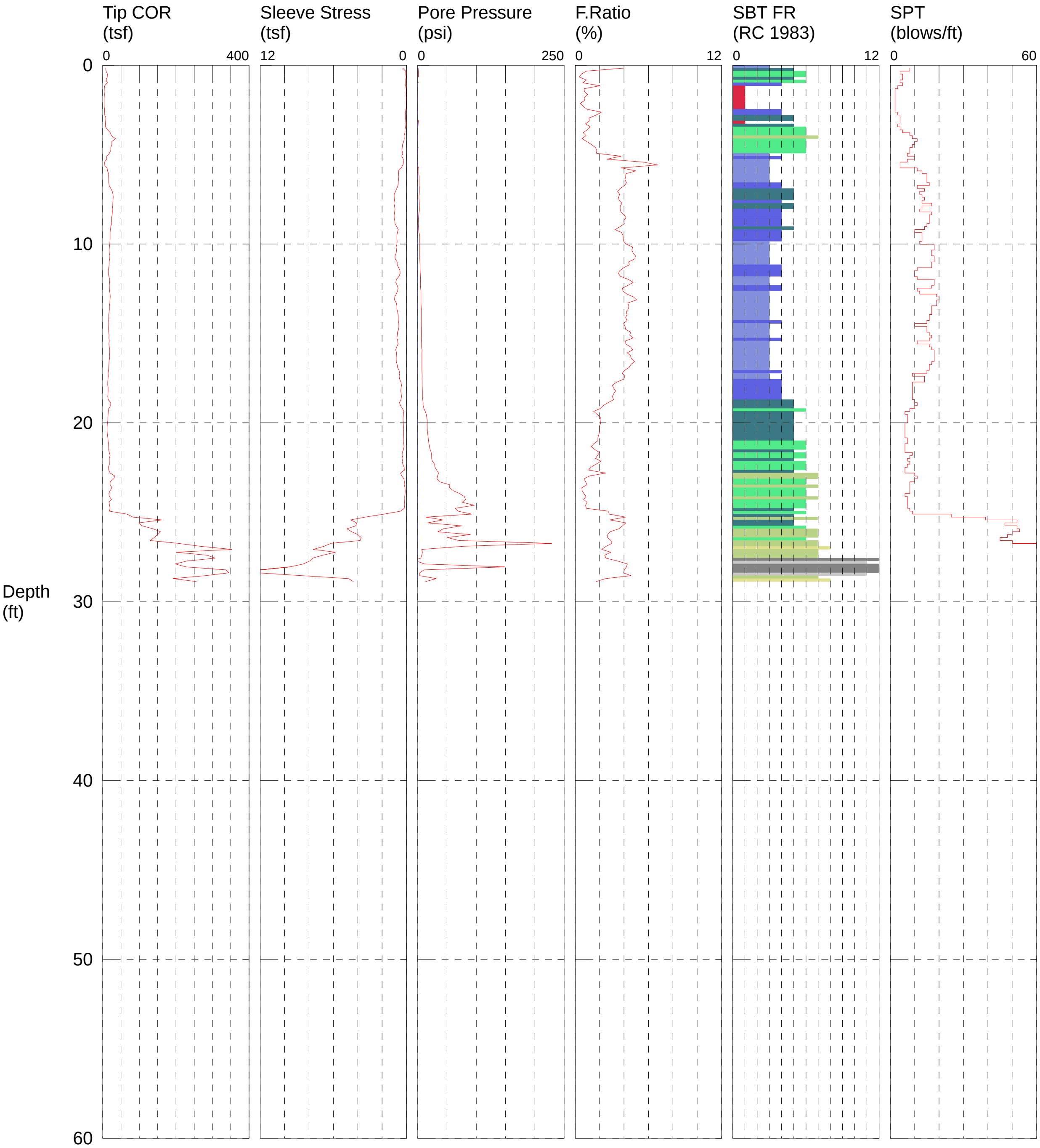
DRAFT

SOUNDING

SOUNDING
CUSTOMER: Taber Drilling
OPERATOR: Tim
CONE ID: DDG1361
LOCATION:

JOB NUMBER: 403147001
HOLE NUMBER: CPT-2
TEST DATE: 11/2/2017 10:21:30 AM
COMMENT: Auto Enhance On
COMMENT: Filter On

COMMENT:
GPS (LAT,LON,ALT): 0.00,0.00,0.0
LOCATION:
LOCATION:
LOCATION:



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

*SBT/SPT CORRELATION: UBC-1983

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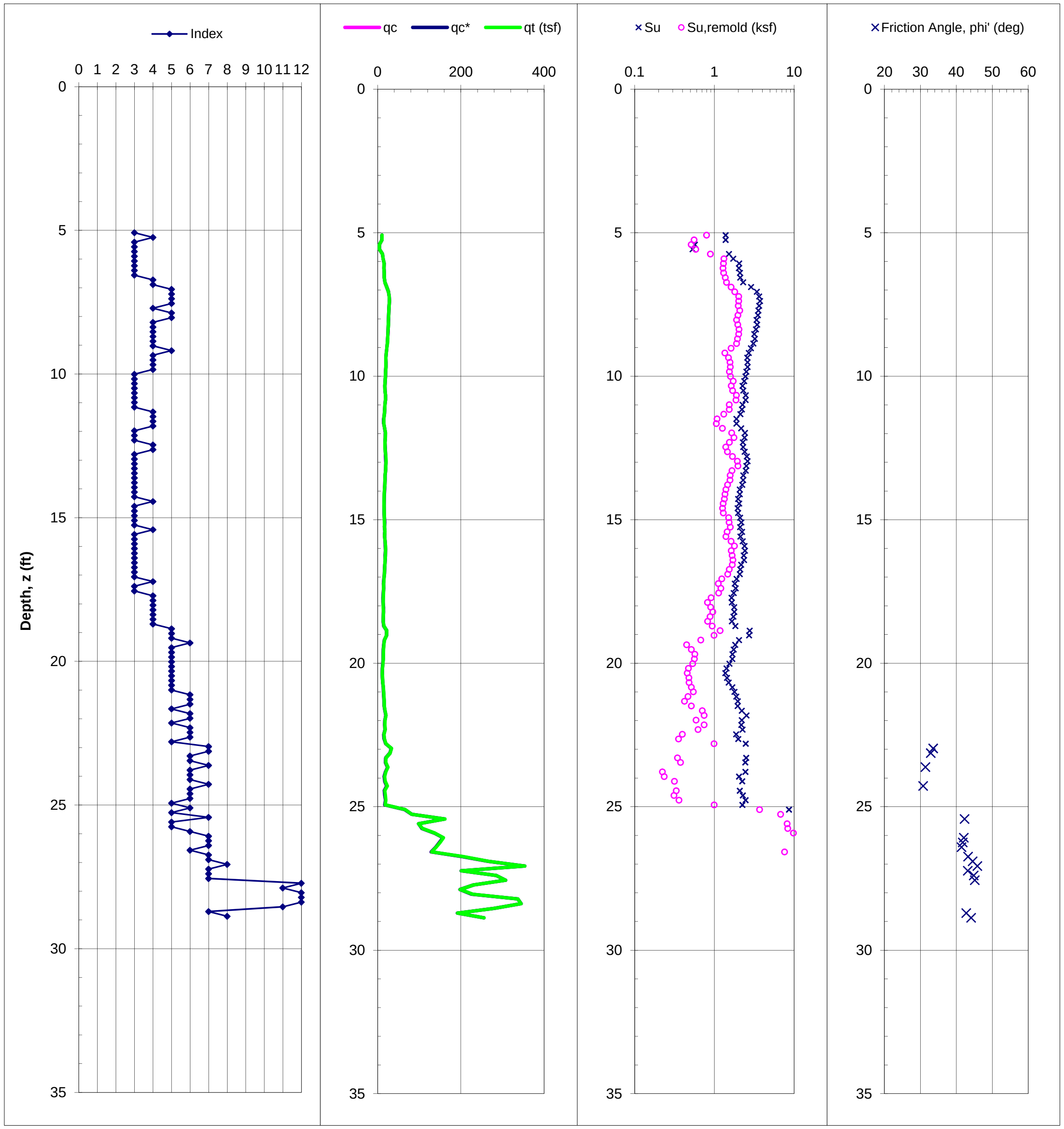
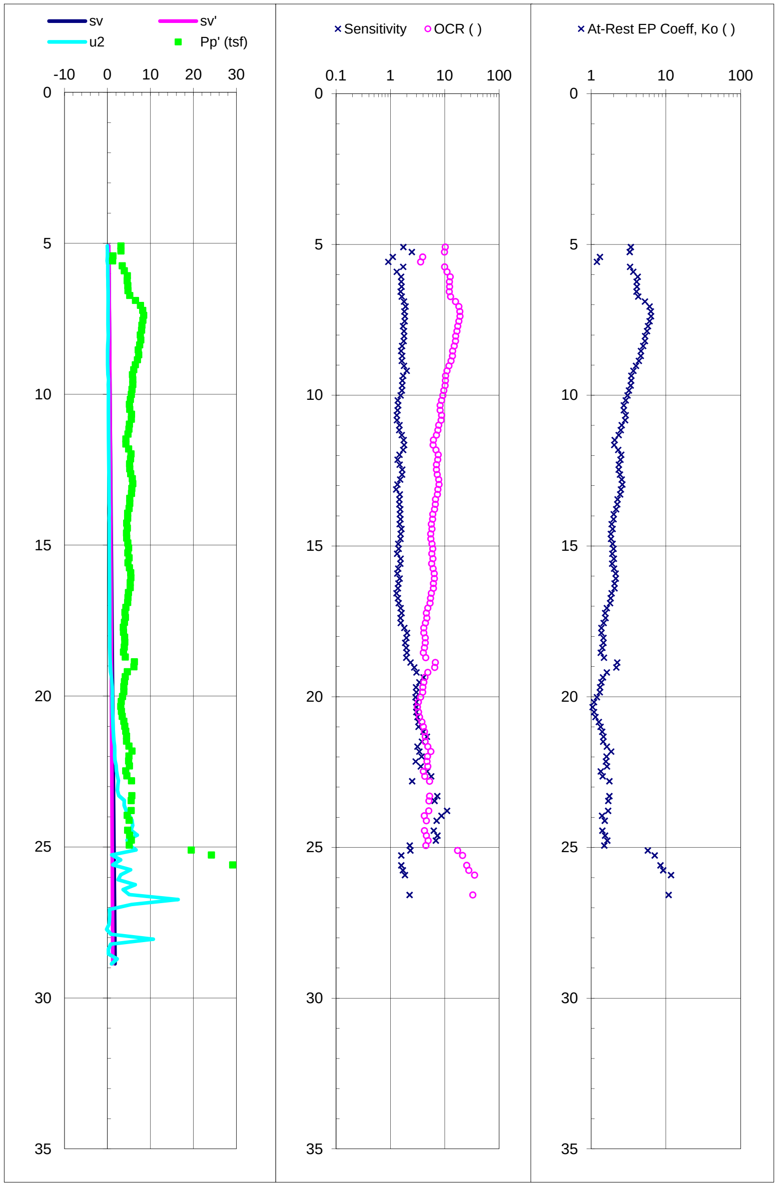
CONE PENETRATION TEST DATA

CPT Sounding: CPT-2
Location: CPT-2

Elevation at top of sounding (ft, MSL)	43
Depth to GWT during CPT evaluation (ft)	12
Cone Diameter, dc (mm)	35.7
Net End Area Ratio ( )	0.80
Atmospheric Pressure (tsf)	1

Project Name:	Solano Community College		
Project Number:	403147001		
Calculation By:	RH	Date:	11/7/2017
Checked By:	TPS	Date:	11/10/2017





File: cpt data vers2.xls
Sheet: CPT2

DRAFT

SOUNDING

SOUNDING
CUSTOMER: Taber Drilling
OPERATOR: Tim
CONE ID: DDG1361
LOCATION:

JOB NUMBER: 403147001
HOLE NUMBER: CPT-3
TEST DATE: 11/2/2017 12:10:06 PM
COMMENT: Auto Enhance On
COMMENT: Filter On

COMMENT:
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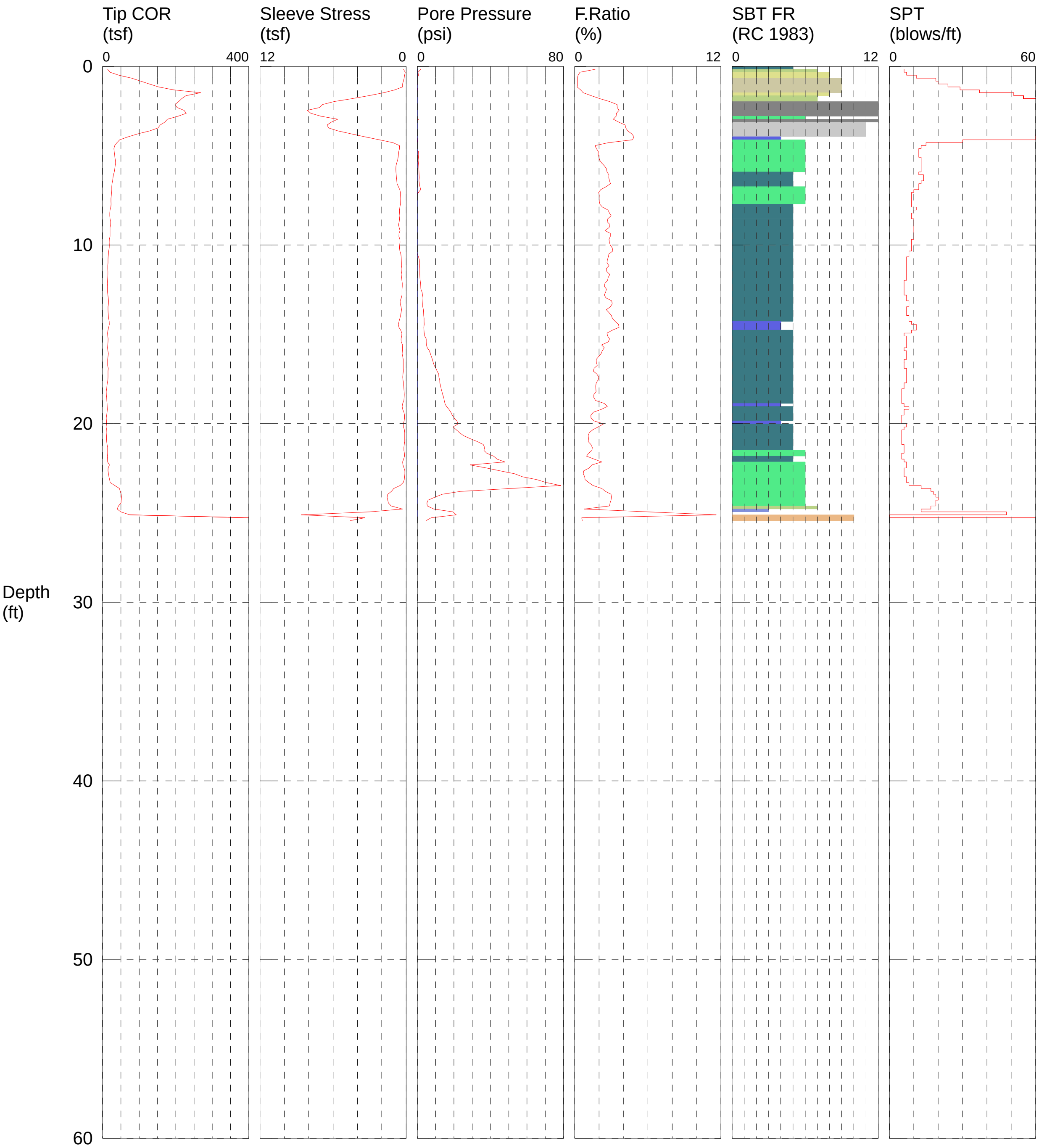
LOCATION:
LOCATION:
LOCATION:



🟥 1 sensitive fine grained
🟥 2 organic material
🟦 3 clay

🟦 4 silty clay to clay
🟩 5 clayey silt to silty clay
🟩 6 sandy silt to clayey silt

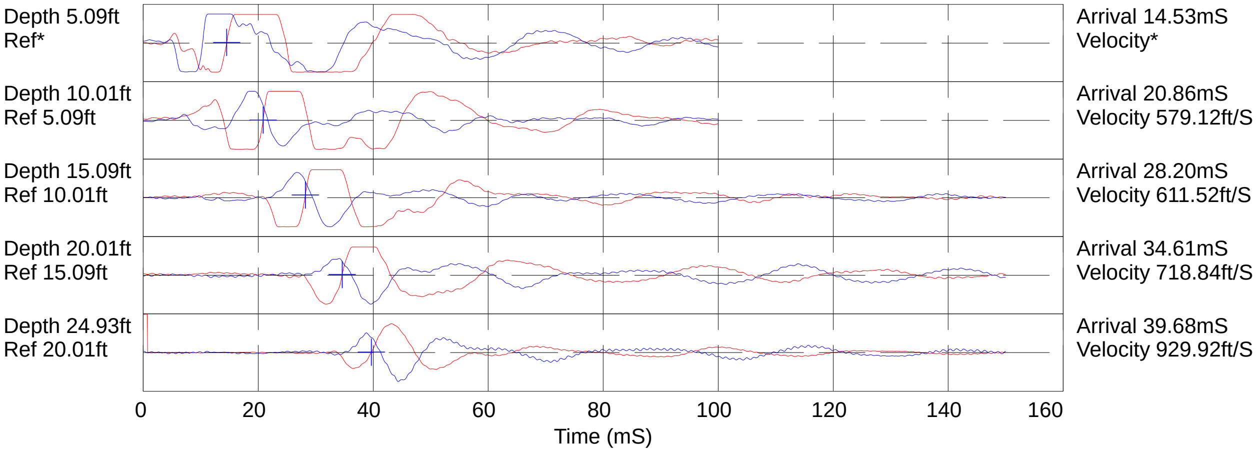
🟩 7 silty sand to sandy silt
🟨 8 sand to silty sand
🟫 9 sand

🟧 10 gravelly sand to sand
⬜ 11 very stiff fine grained (*)
⬛ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

DRAFT

SEISMIC TEST



Hammer to Rod String Distance (ft): 6.56
* = Not Determined

COMMENT:

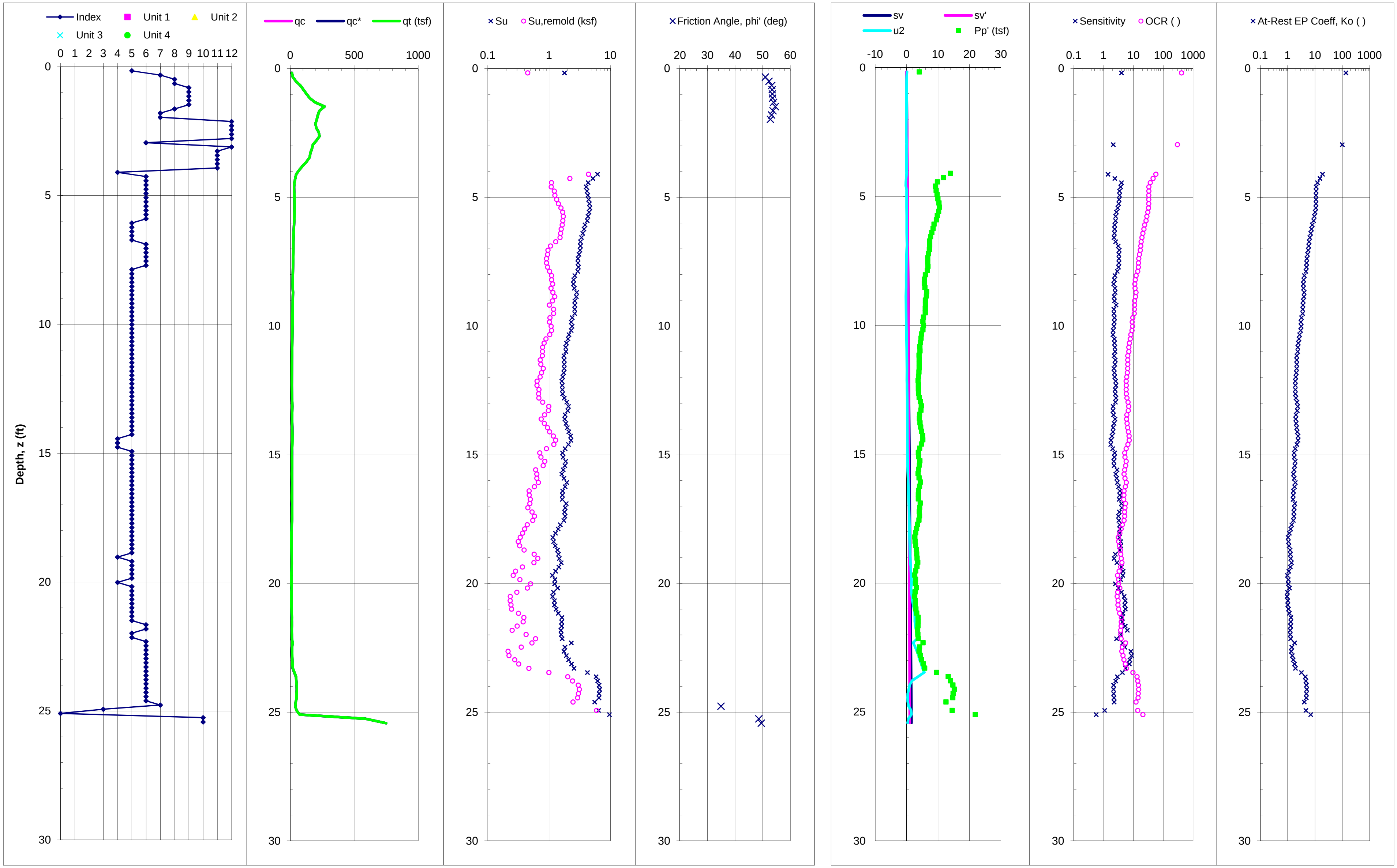
DRAFT

CONE PENETRATION TEST DATA

CPT Sounding:	CPT-3
Location:	CPT-3

Elevation at top of sounding (ft, MSL)	43
Depth to GWT during CPT evaluation (ft)	9
Cone Diameter, dc (mm)	35.7
Net End Area Ratio ( )	0.80
Atmospheric Pressure (tsf)	1

Project Name:	Solano Community College		
Project Number:	403147001		
Calculation By:	RH	Date:	11/7/2017
Checked By:	TPS	Date:	11/10/2017



File: cpt data vers2.xls
Sheet: CPT3

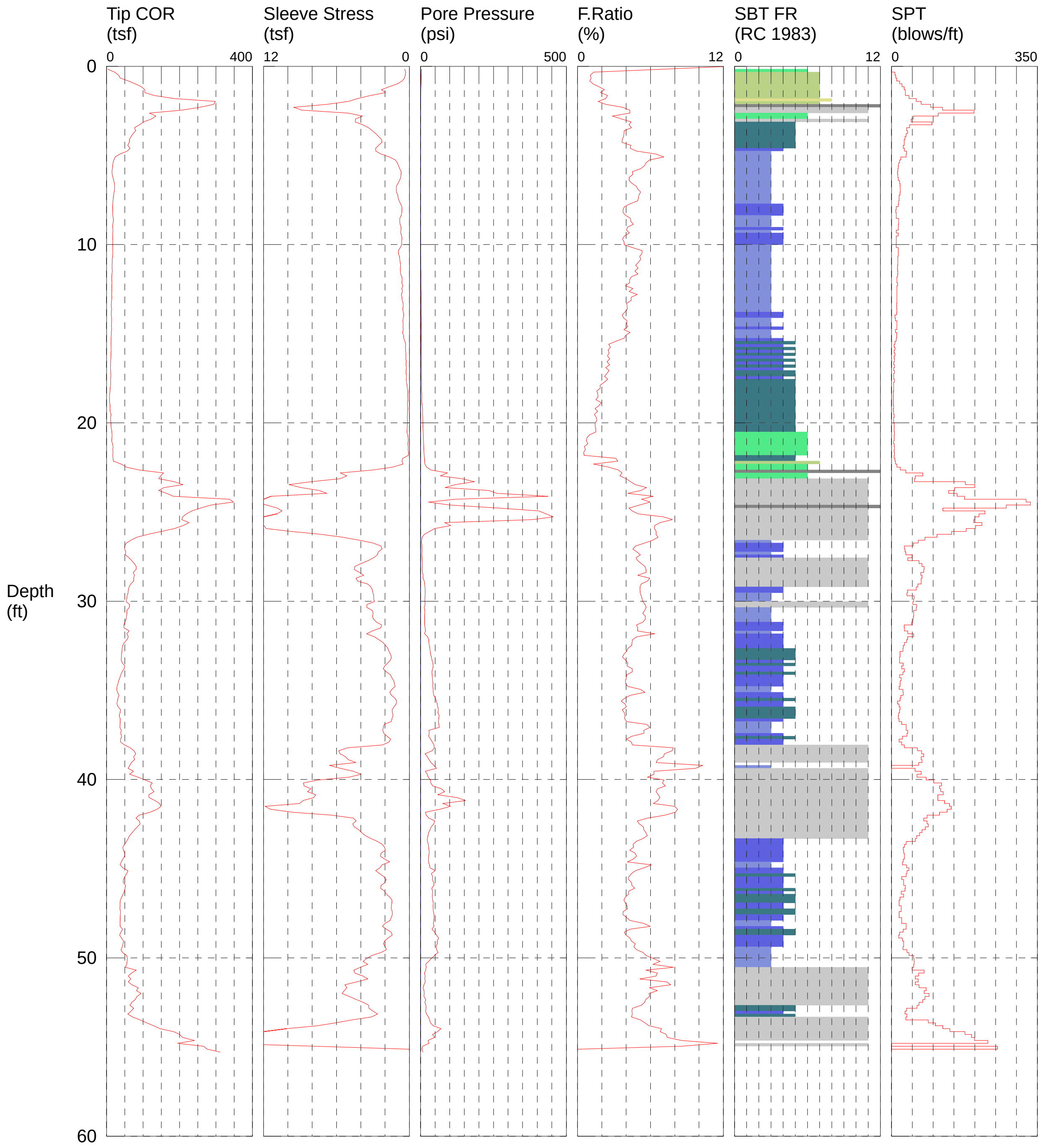
DRAFT

SOUNDING

SOUNDING
CUSTOMER: Taber Drilling
OPERATOR: Tim
CONE ID: DDG1361
LOCATION:

JOB NUMBER: 403147001
HOLE NUMBER: CPT-4
TEST DATE: 11/2/2017 1:30:31 PM
COMMENT: Auto Enhance On
COMMENT: Filter On

COMMENT:
GPS (LAT,LON,ALT): 0.00,0.00,0.0
LOCATION:
LOCATION:
LOCATION:



🟥 1 sensitive fine grained
🟫 2 organic material
🟦 3 clay
🟦 4 silty clay to clay
🟩 5 clayey silt to silty clay
🟩 6 sandy silt to clayey silt
🟩 7 silty sand to sandy silt
🟨 8 sand to silty sand
🟫 9 sand
🟧 10 gravelly sand to sand
⬜ 11 very stiff fine grained (*)
⬛ 12 sand to clayey sand (*)

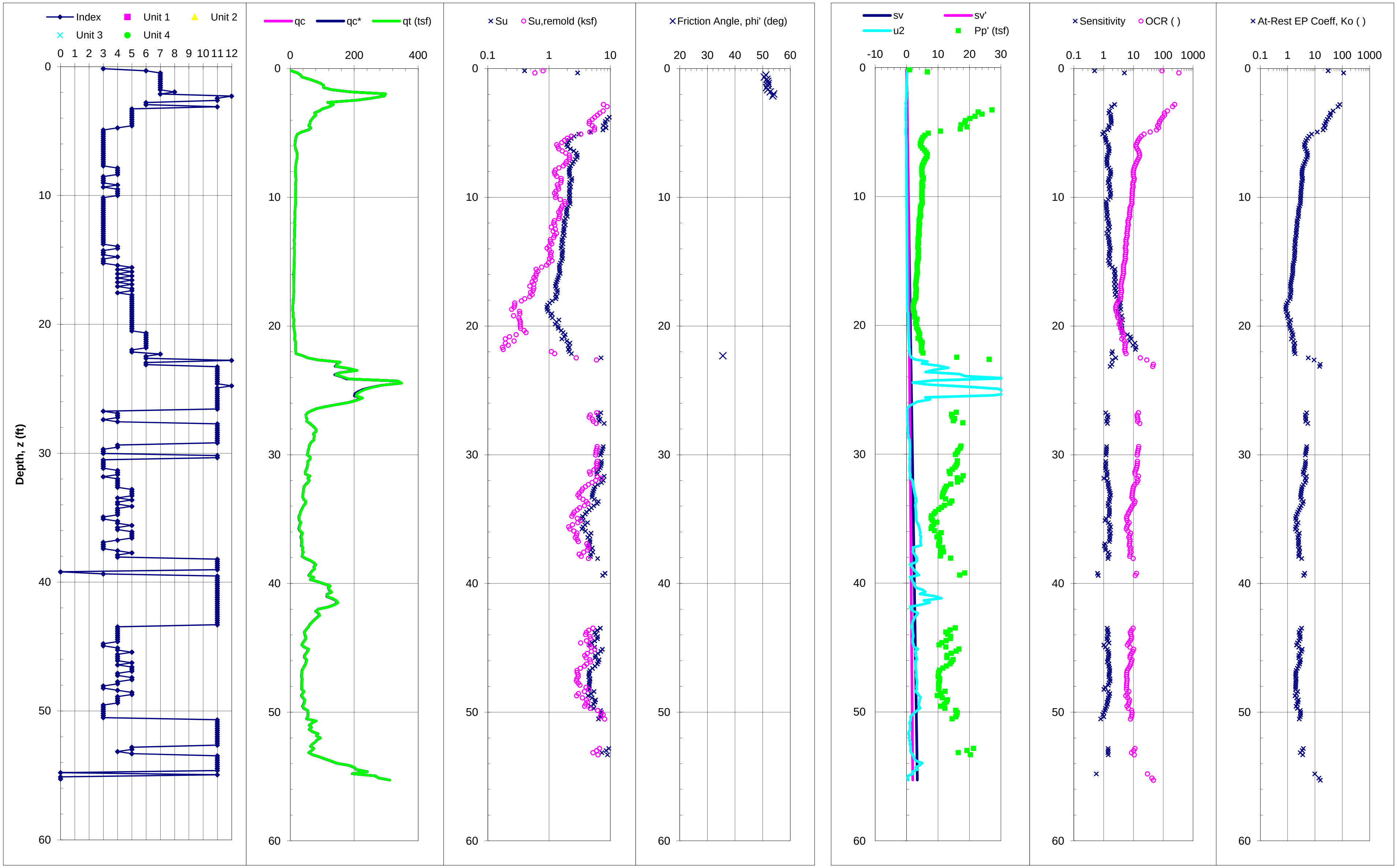
*SBT/SPT CORRELATION: UBC-1983

CONE PENETRATION TEST DATA

CPT Sounding: CPT-4
Location: CPT-4

Elevation at top of sounding (ft, MSL)	45
Depth to GWT during CPT evaluation (ft)	8
Cone Diameter, dc (mm)	35.7
Net End Area Ratio ( )	0.80
Atmospheric Pressure (tsf)	1

Project Name:	Solano Community College		
Project Number:	403147001		
Calculation By:	RH	Date:	11/7/2017
Checked By:	TPS	Date:	11/10/2017



File: cpt data vers2.xls
Sheet: CPT4

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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. 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 logs of the exploratory borings 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

Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422. The grain-size distribution curves are shown on Figures C-1 through C-5. 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-6.

Expansion Index Test

The expansion index of a selected material was evaluated in general accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1 inch thick by 4 inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The test results are presented on Figure C-7.

Consolidation Tests

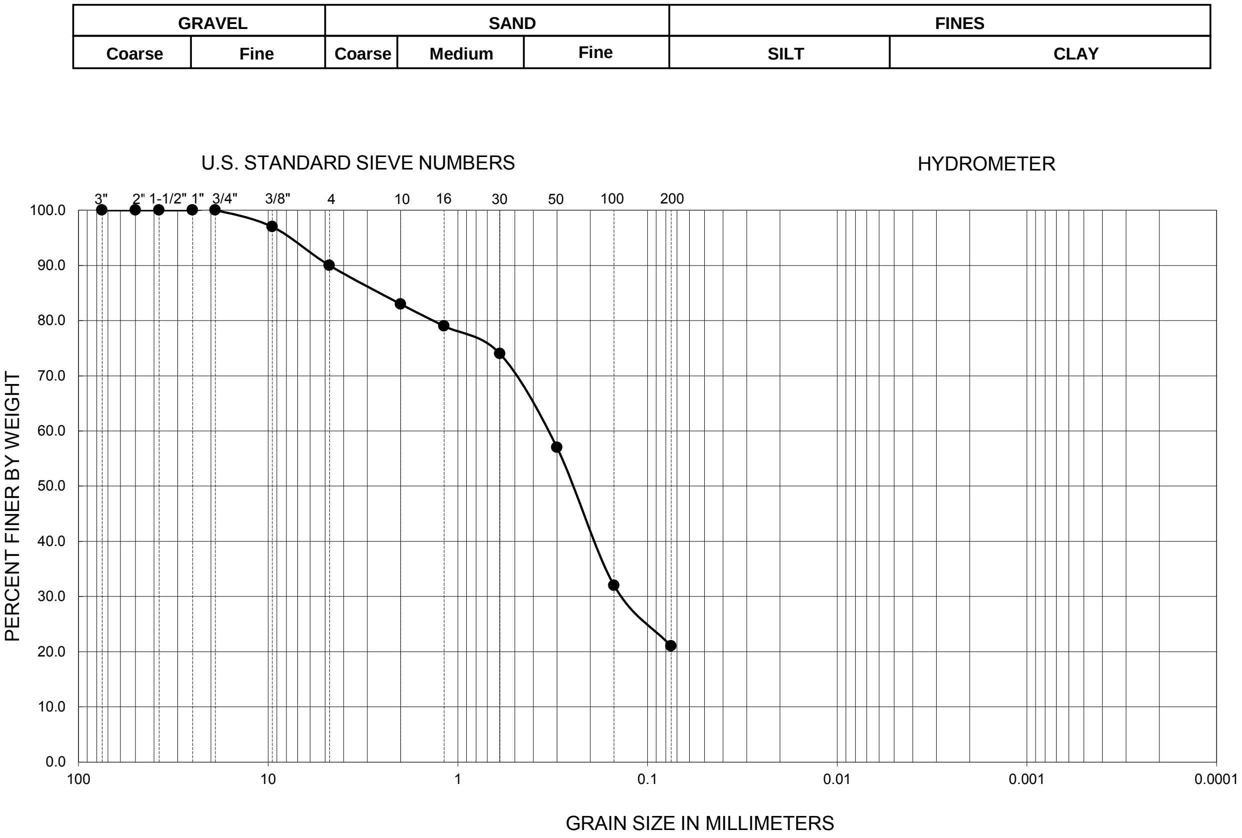
A consolidation test was performed on a selected relatively undisturbed soil sample in general accordance with ASTM D 2435. The sample was inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the test are summarized on Figure C-8.

Unconfined Compression Test

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-9.

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 content of selected samples were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure C-10.



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-1	19.0-19.5	--	--	--	--	0.14	0.33	--	--	21	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE C-1

GRADATION TEST RESULTS

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS
4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA

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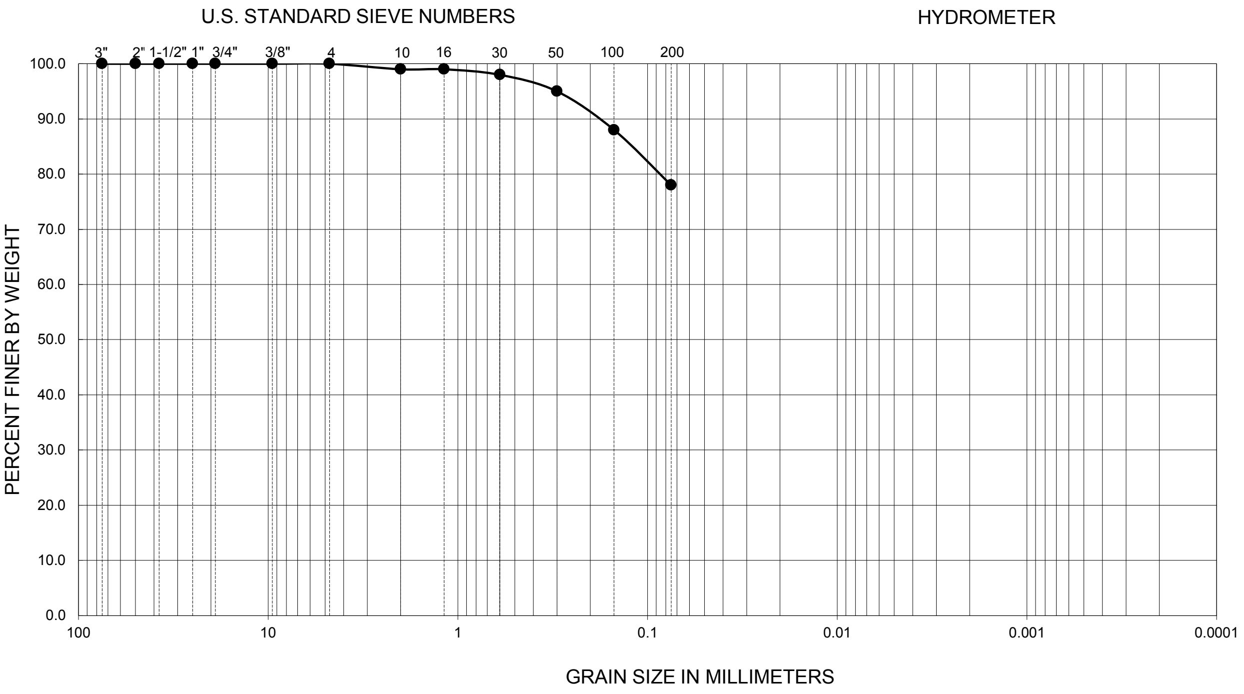
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



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-2	9.0-9.5	36	16	20	--	--	--	--	--	78	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE C-2

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GRADATION TEST RESULTS

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS
4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA

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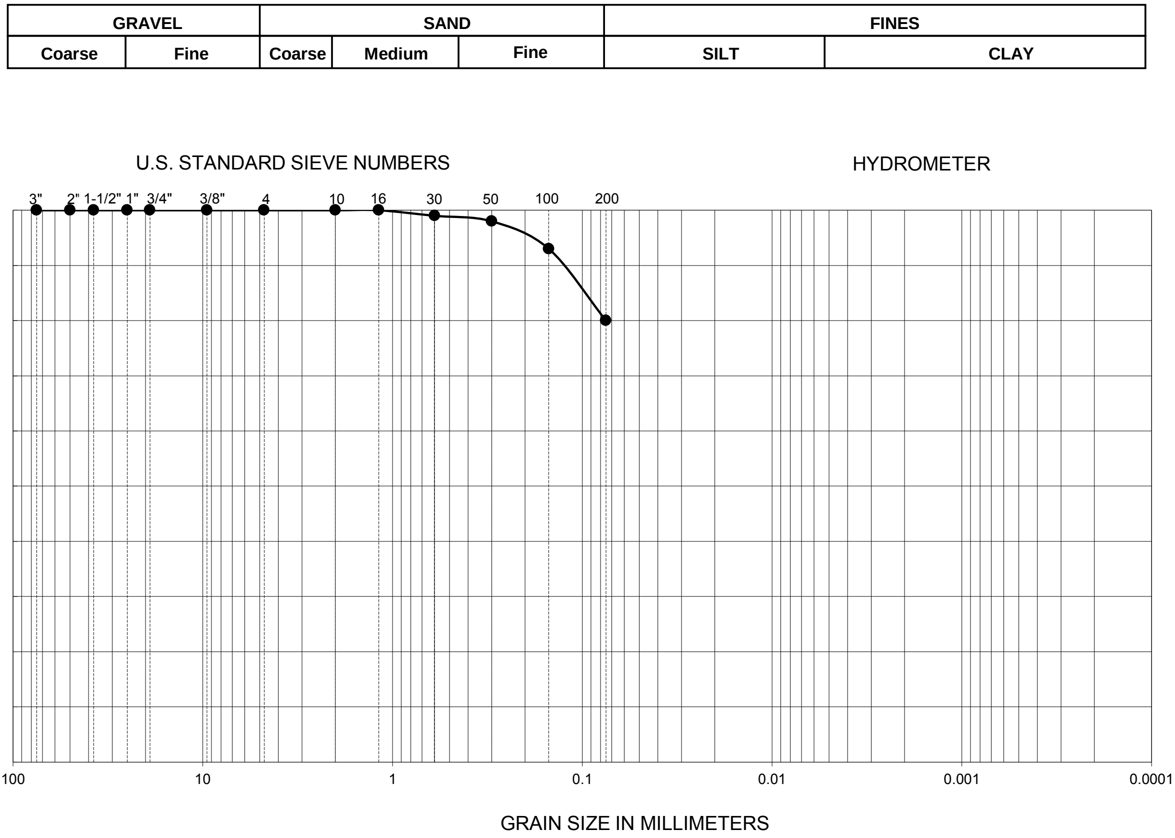

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-3	19.0-19.5	35	16	19	--	--	--	--	--	80	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE C-3

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GRADATION TEST RESULTS

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS
4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA

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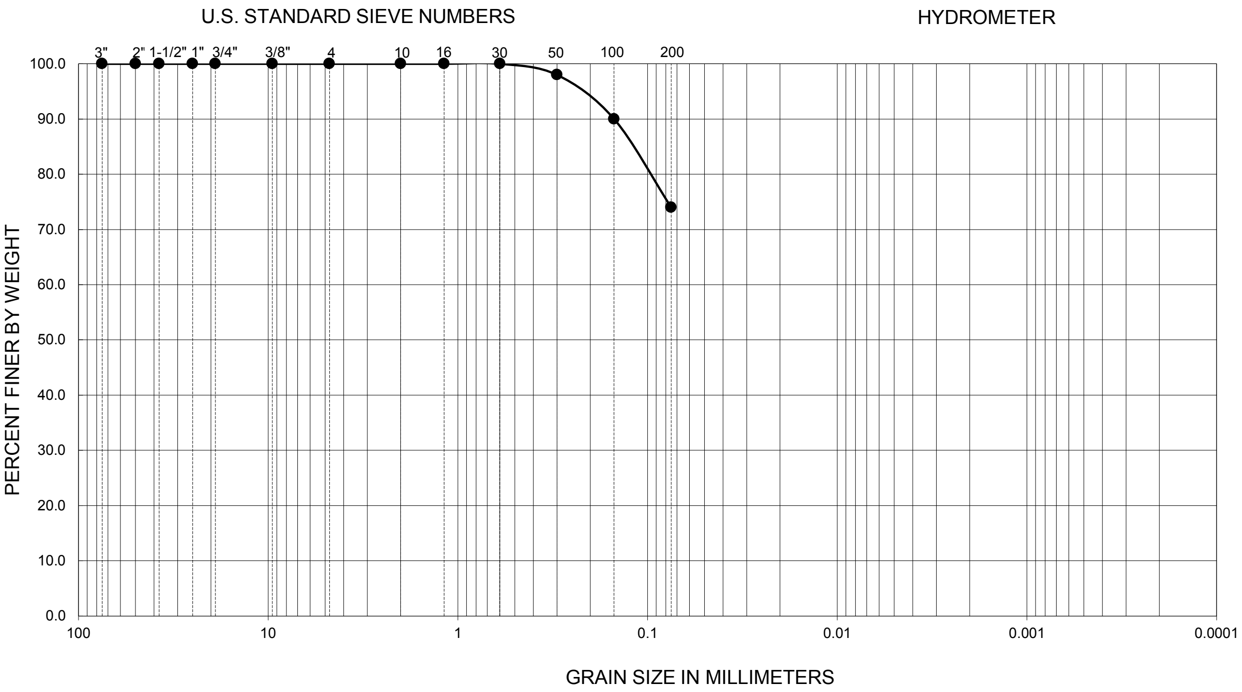
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



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-6	0.0-5.0	34	15	19	--	--	--	--	--	74	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE C-4

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GRADATION TEST RESULTS

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS
4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA

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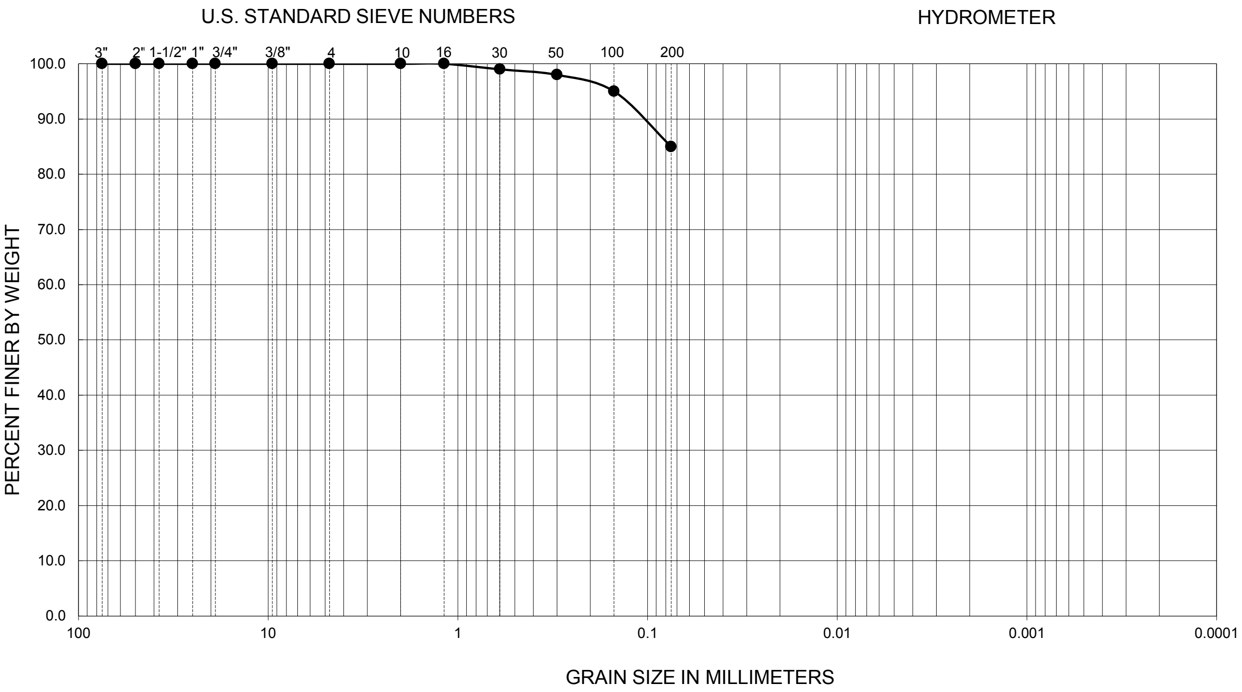
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



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-8	5.5-6.0	--	--	--	--	--	--	--	--	85	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE C-5

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GRADATION TEST RESULTS

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS
4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA

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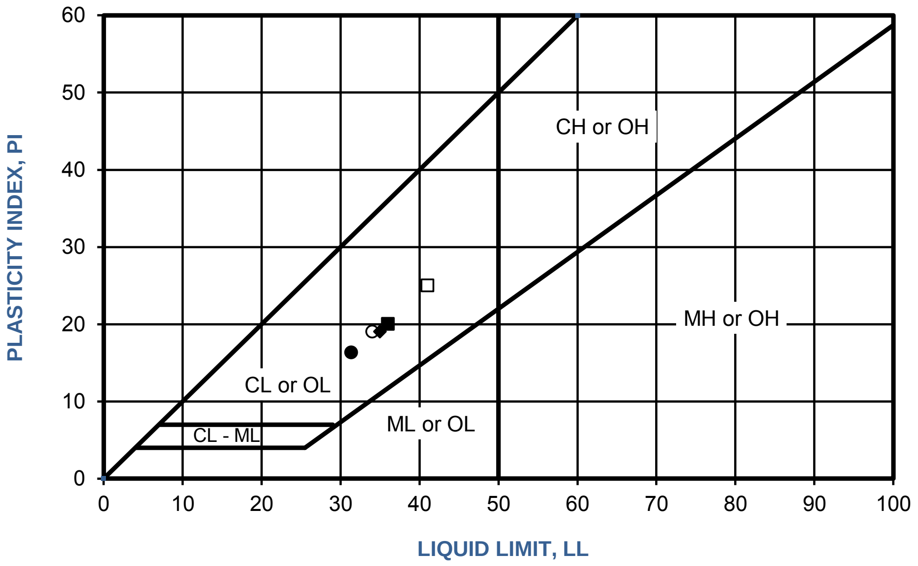
SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-1	5.5-6.0	31	15	16	CL	CL
■	B-2	9.0-9.5	36	16	20	CL	CL
◆	B-3	19.0-19.5	35	16	19	CL	CL
○	B-6	0.0-5.0	34	15	19	CL	CL
□	B-8	6.0-6.5	41	16	25	CL	CL



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

FIGURE C-6

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ATTERBERG LIMITS TEST RESULTS

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS
4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
B-2	0.0-5.0	12.7	101.1	27.9	0.100	100	High

PERFORMED IN GENERAL ACCORDANCE WITH ☐ UBC STANDARD 18-2 ☑ ASTM D 4829

FIGURE C-7

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EXPANSION INDEX TEST RESULTS

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS
4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA

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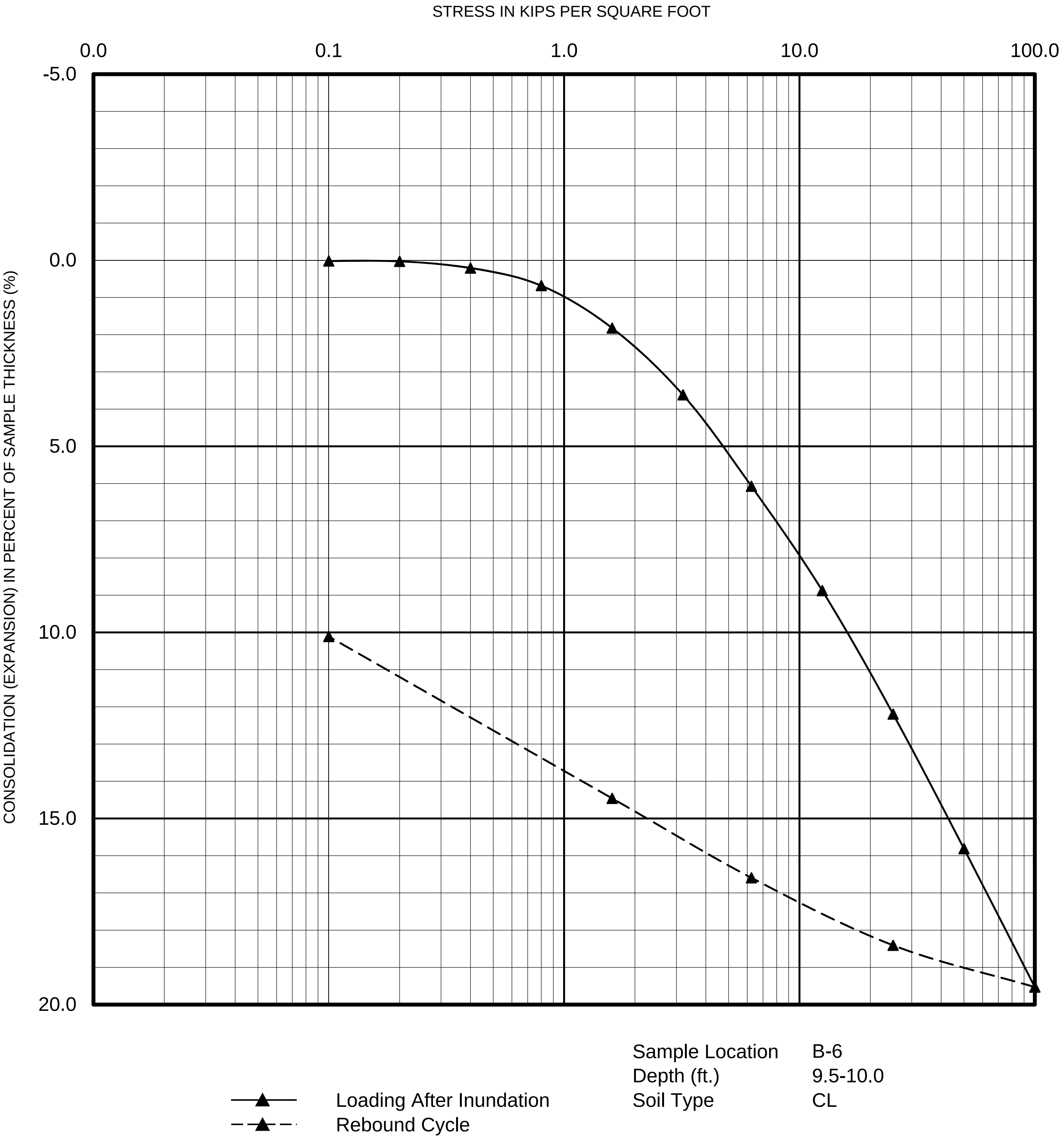

Sample Location	B-6
Depth (ft.)	9.5-10.0
Soil Type	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE C-8

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CONSOLIDATION TEST RESULTS

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS
4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA
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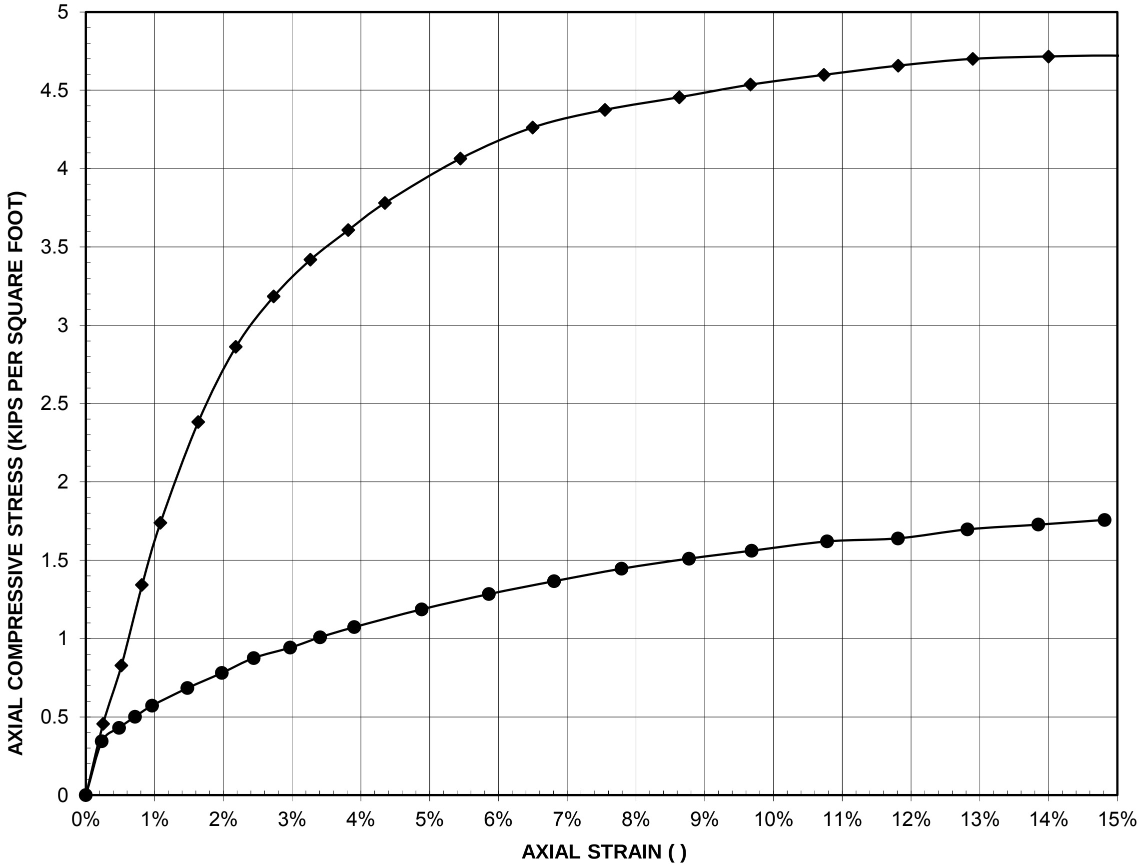

Symbol	Description	Soil Type	Soil Location	Soil Depth (ft)	Initial Moisture (%)	Initial Dry Density (pcf)	Rate of Strain (%/min)	Shear Strength s_u (ksf)
◆	**Brown lean CLAY**	**CL**	**B-4**	**6.0-6.5**	**22.1**	**102.8**	**1.1**	**2.4**
●	**Brown lean CLAY**	**CL**	**B-5**	**9.5-10.0**	**26.4**	**96.5**	**1.0**	**0.9**

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2166

FIGURE C-9

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UNCONFINED COMPRESSION RESULTS

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS
4000 SUISUN VALLEY ROAD, FAIRFIELD, CALIFORNIA

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-8	0.0-5.0	7.0	2,200	20	0.002	105

[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 C-10

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CORROSIVITY TEST RESULTS

SOLANO COMMUNITY COLLEGE - FAIRFIELD CAMPUS
4000 SUISUN VALLEY ROAD, FAIRFIELD CALIFORNIA

403147001 | 12/17

APPENDIX D

Calculations

LIQUEFACTION AND DYNAMIC SETTLEMENT EVALUATION BY CPT

Project Name:	Solano Community College		
Project Number:	403147001		
Calculation By:	RH	Date:	11/7/2017
Checked By:	TPS	Date:	11/10/2017

CPT Sounding: CPT-1
Location: CPT-1

Depth to GWT during CPT evaluation (ft)	8
Design Depth to GWT (ft)	6
Atmospheric Pressure (tsf)	1.0581
Design EQ Peak Ground Acceleration, amax (g)	0.809
Design Earthquake Moment Magnitude, Mw	6.8
Magnitude Scaling Factor, MSF	1.28
At-Rest Coefficient Lateral EP, Ko	0.5
Number of Strain Cycles, Nc	9.34

Estimated dry soil dynamic settlement (in)	0.01
Estimated saturated soil dynamic settlement (in)	0.19
Total estimated dynamic settlement (in)	0.20

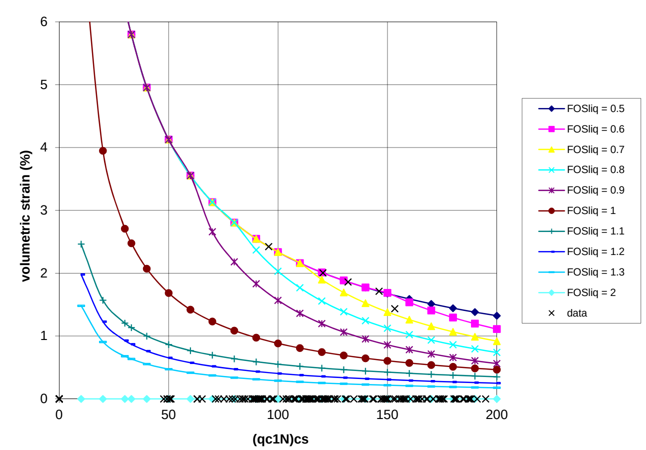
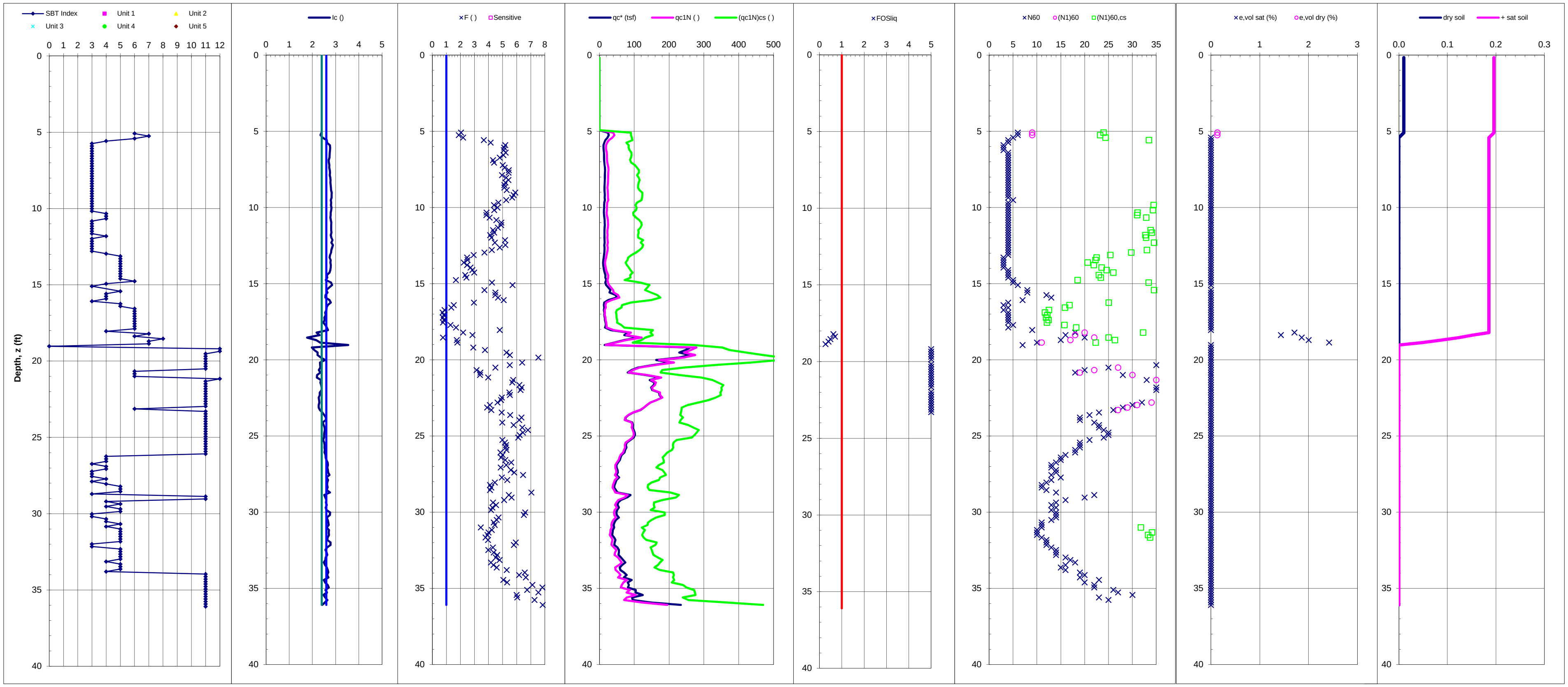


Fig 3: Zhang et al, 2002



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LIQUEFACTION AND DYNAMIC SETTLEMENT EVALUATION BY CPT

Project Name:	Solano Community College		
Project Number:	403147001		
Calculation By:	RH	Date:	11/7/2017
Checked By:	TPS	Date:	11/10/2017

CPT Sounding: CPT-2
Location: CPT-2

Depth to GWT during CPT evaluation (ft)	12
Design Depth to GWT (ft)	6
Atmospheric Pressure (tsf)	1.0581
Design EQ Peak Ground Acceleration, amax (g)	0.809
Design Earthquake Moment Magnitude, Mw	6.8
Magnitude Scaling Factor, MSF	1.28
At-Rest Coefficient Lateral EP, Ko	0.5
Number of Strain Cycles, Nc	9.34

Estimated dry soil dynamic settlement (in)	0.00
Estimated saturated soil dynamic settlement (in)	0.25
Total estimated dynamic settlement (in)	0.25

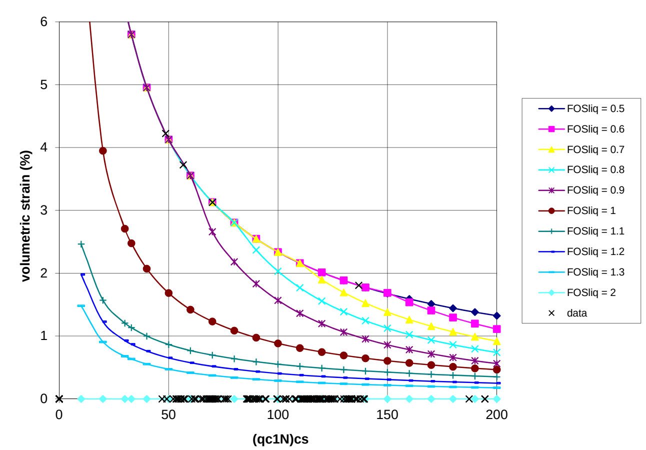
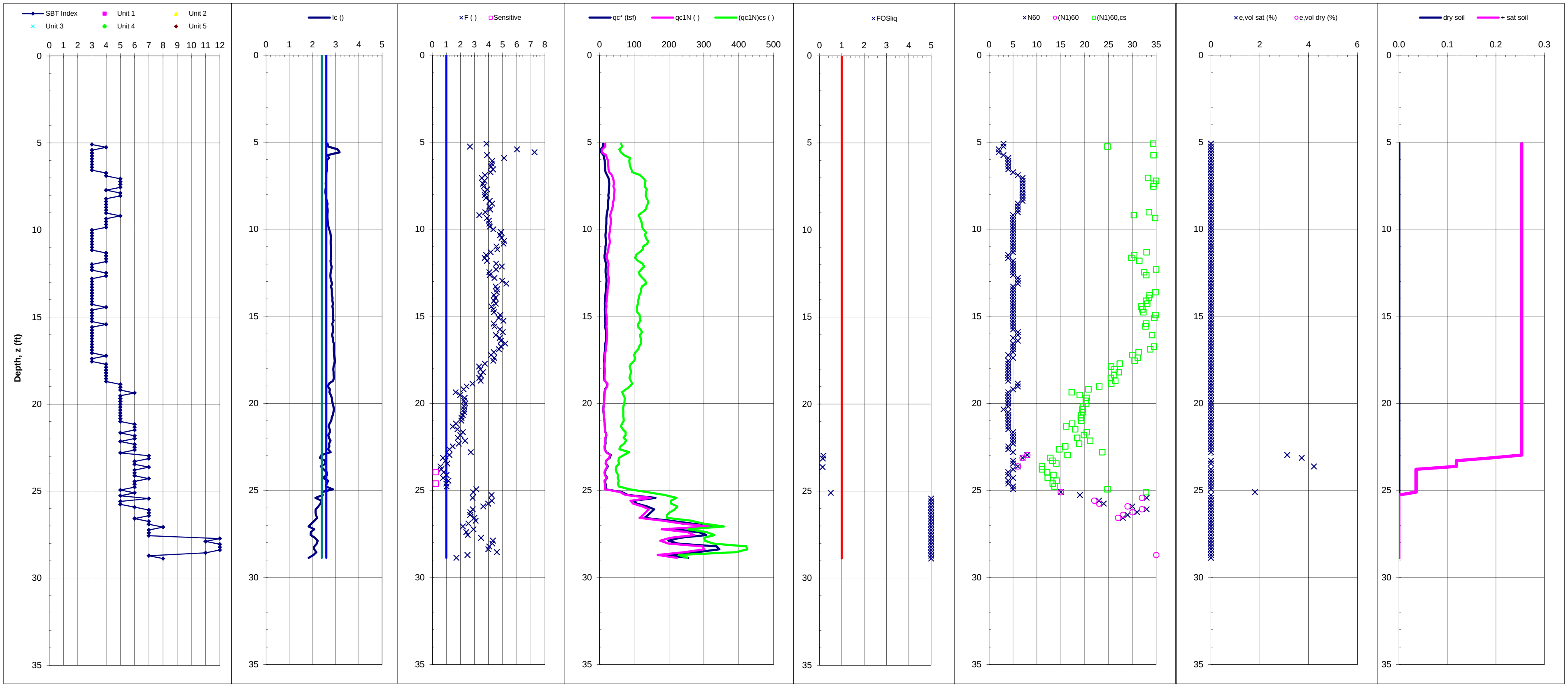


Fig 3: Zhang et al, 2002



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LIQUEFACTION AND DYNAMIC SETTLEMENT EVALUATION BY CPT

Project Name:	Solano Community College		
Project Number:	403147001		
Calculation By:	RH	Date:	11/7/2017
Checked By:	TPS	Date:	11/10/2017

CPT Sounding:	CPT-3
Location:	CPT-3

Depth to GWT during CPT evaluation (ft)	9
Design Depth to GWT (ft)	6
Atmospheric Pressure (tsf)	1.0581
Design EQ Peak Ground Acceleration, amax (g)	0.809
Design Earthquake Moment Magnitude, Mw	6.8
Magnitude Scaling Factor, MSF	1.28
At-Rest Coefficient Lateral EP, Ko	0.5
Number of Strain Cycles, Nc	9.34

Estimated dry soil dynamic settlement (in)	0.03
Estimated saturated soil dynamic settlement (in)	0.21
Total estimated dynamic settlement (in)	0.24

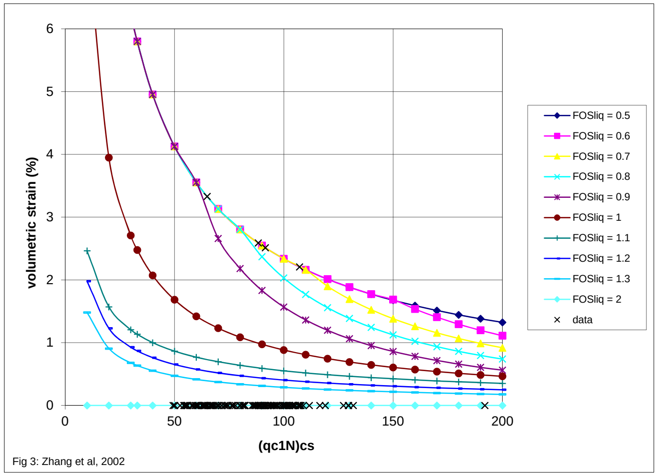
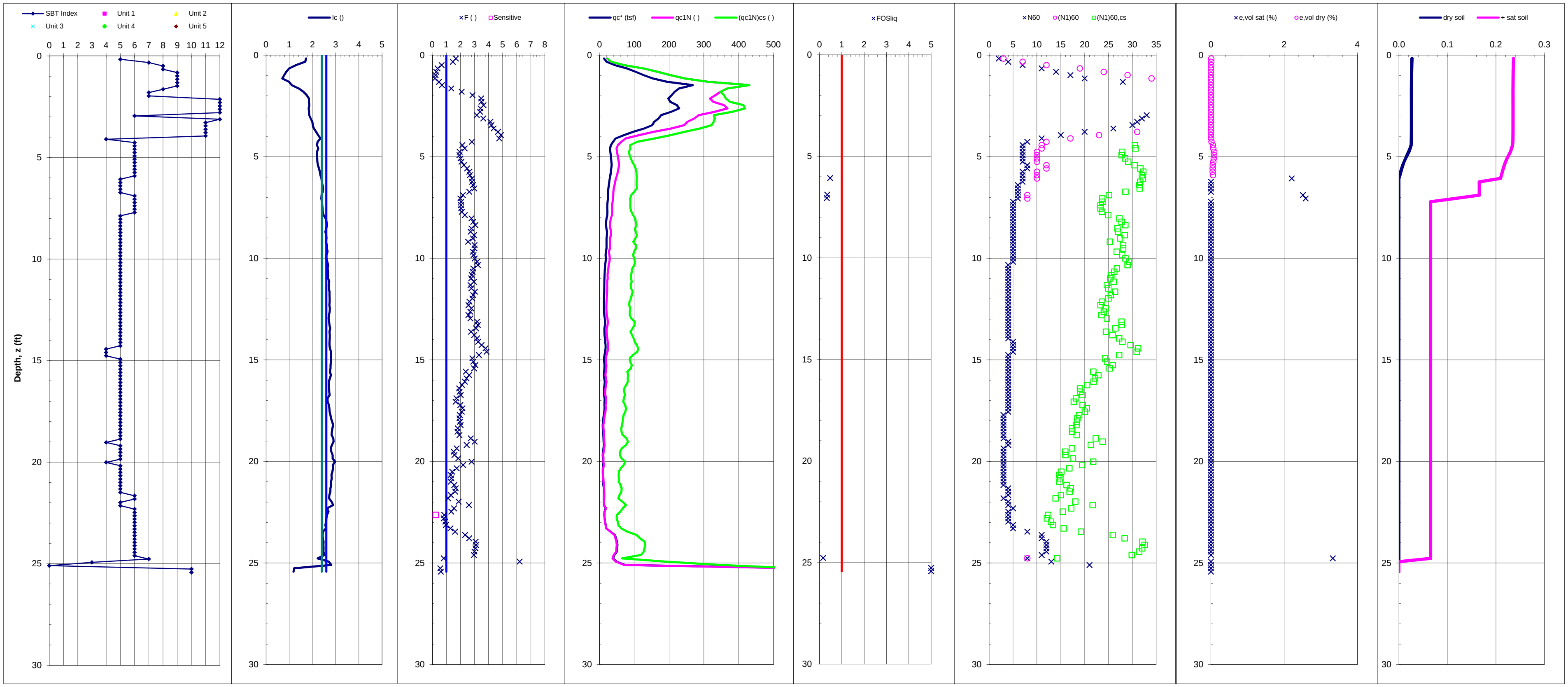


Fig 3: Zhang et al, 2002



LIQUEFACTION AND DYNAMIC SETTLEMENT EVALUATION BY CPT

Project Name:	Solano Community College		
Project Number:	403147001		
Calculation By:	RH	Date:	11/7/2017
Checked By:	TPS	Date:	11/10/2017

CPT Sounding:	CPT-4
Location:	CPT-4

Depth to GWT during CPT evaluation (ft)	8
Design Depth to GWT (ft)	6
Atmospheric Pressure (tsf)	1.0581
Design EQ Peak Ground Acceleration, amax (g)	0.809
Design Earthquake Moment Magnitude, Mw	6.8
Magnitude Scaling Factor, MSF	1.28
At-Rest Coefficient Lateral EP, Ko	0.5
Number of Strain Cycles, Nc	9.34

Estimated dry soil dynamic settlement (in)	0.00
Estimated saturated soil dynamic settlement (in)	0.09
Total estimated dynamic settlement (in)	0.09

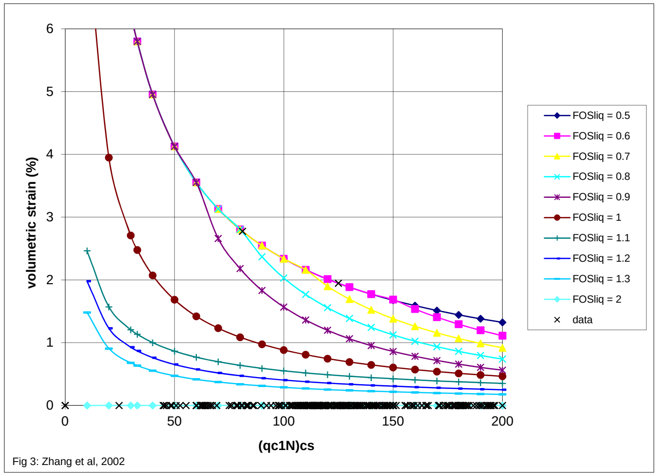
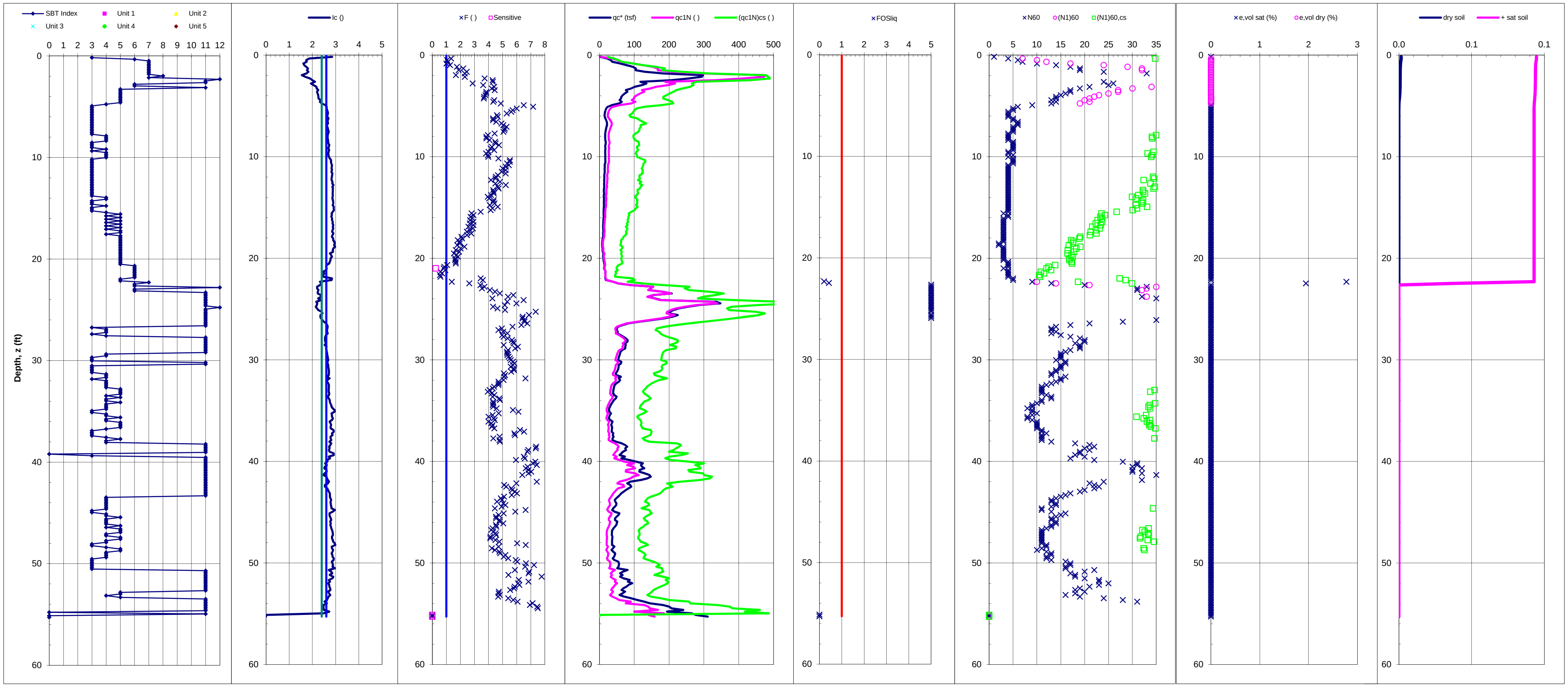


Fig 3: Zhang et al, 2002



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