Feasibility Level Geotechnical Evaluation

East Whisman

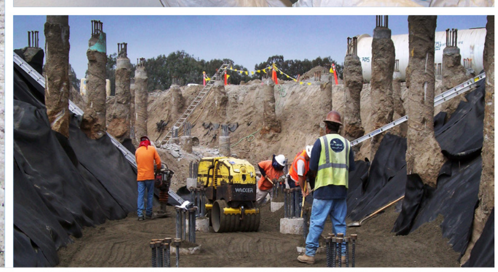
440 Clyde Avenue

Mountain View, California

Google LLC

1600 Amphitheater Parkway | Mountain View, California 94043

April 3, 2020 | Project No. 403253010



Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

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Ms. Lisa Herrera
Google LLC
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PROFESSIONAL GEOLOGIST
DAVID C. SEYMOUR
NO. 1574
CERTIFIED
ENGINEERING
GEOLOGIST
STATE OF CALIFORNIA
April 3, 2020

REGISTERED PROFESSIONAL ENGINEER
RANSOM H. HENNEFER
GE 3152
GEOTECHNICAL
STATE OF CALIFORNIA
April 3, 2020

Ransom H. Hennefer, PE, GE
Senior Project Engineer

David C. Seymour, PG, CEG
Principal Engineering Geologist

REGISTERED PROFESSIONAL ENGINEER
SOUMITRA GUHA
NO. GE 2632
GEOTECHNICAL
STATE OF CALIFORNIA
April 3, 2020

Sam Guha, PhD, PE, GE
PPrincipal Engineer

Terence K. Wang, PE, GE
Principal Engineer

RH/DCS/SG/TKW/gvr

Distribution: (1) Addressee (via e-mail)

REGISTERED PROFESSIONAL ENGINEER
TERENCE K. WANG
No. GE2050
GEOTECHNICAL
STATE OF CALIFORNIA
April 3, 2020

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1 INTRODUCTION

In accordance with your request, we have performed a feasibility level geotechnical evaluation for the 440 Clyde Avenue property in Mountain View, California (Figure 1) as part of the East Whisman project. The purpose of our geotechnical evaluation was to assess the subsurface conditions and geologic hazards at the site, and to provide preliminary conclusions and recommendations for planning purposes for the sites described in Section 4 of this report.

2 SCOPE OF SERVICES

Our scope of services included the following:

- Reviewed readily available geologic literature pertinent to the project area including geologic maps and reports.
- Performed site reconnaissance to observe the general site conditions and to mark the proposed locations for subsurface exploration.
- Coordinated with Underground Service Alert to locate the underground utilities in the vicinity of the proposed exploration locations.
- Performed a private utility survey to further check the exploration locations for underground utility conflicts. Excavation and utility location sketches were submitted to Google per the utility excavation checklist.
- Obtained a boring permit from the Santa Clara Valley Water District.
- Provided the utility checklist for review and approval by Google.
- Performed one (1) Cone Penetration Test (CPT) sounding to a depth of approximately 101 feet below the existing grade to evaluate the subsurface conditions and liquefaction susceptibility. The sounding was backfilled with Portland cement grout in compliance with the Santa Clara Valley Water District drilling permit, and pavement was patched.
- Drilled one (1) boring to a depth of approximately 44½ feet below grade, to evaluate the subsurface conditions. The boring was drilled using a truck-mounted drill rig. A representative of Ninyo & Moore logged the subsurface conditions exposed in the boring, and collected bulk and relatively undisturbed samples for laboratory testing. The boring was backfilled with Portland cement grout.
- Drill spoils were collected and sealed in 55-gallon drums. Analytical testing was performed on a composite soil sample from each soil boring location prior to drum disposal per Google's gSafe soil off-haul guidelines.
- Laboratory testing on selected soil samples to evaluate soil moisture and dry density, soil gradation, Atterberg limits, consolidation, expansion potential, soil corrosivity, and undrained triaxial shear strength 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.

- Prepared this geotechnical feasibility report presenting our preliminary findings, conclusions, and recommendations.

3 SITE DESCRIPTION AND BACKGROUND

The subject property is located at 440 Clyde Avenue in Mountain View, California (Figure 1). The site is currently occupied by a one-story commercial building, paved parking areas, and landscaped areas. The property is bounded to the north, west, and south by commercial properties and to the east by Clyde Avenue. The site is relatively flat with elevations that range from about 51 to 53 feet above mean sea level (Google Earth, 2019).

4 PROJECT DESCRIPTION

As part of the planned development for the East Whisman District, Google has identified twenty-four (24) properties in Mountain View, California for preliminary geotechnical evaluation (Table 1). The details of the proposed new construction are not known at this time, but are anticipated to include the construction of new multi-story commercial or mixed-use buildings.

Table 1 – List of Properties

Ninyo and Moore Report Number	Address
01	450 Clyde Avenue
02	440 Clyde Avenue
03	405 Clyde Avenue
04	891 Maude Avenue
05	433 Clyde Avenue
06	485 Clyde Avenue
07	880 Maude Avenue
08	885 Maude Avenue
09	420 Clyde Avenue
10	495 Clyde Avenue
11	520-526 Clyde Avenue
12	500-506 Clyde Avenue
13	510-516 Clyde Avenue
14	520 Logue Avenue
15	530 Logue Avenue
16	510 Logue Avenue
17	500 Logue Avenue
18	440 Logue Avenue
19	441 Logue Avenue
20	800 Maude Avenue
21	830 Maude Avenue

Table 1 – List of Properties	
Ninyo and Moore Report Number	**Address**
22	840-850 Maude Avenue
23	401 Ellis Street
24	500 East Middlefield Road

5 SUBSURFACE EVALUATION AND LABORATORY TESTING

Our field exploration for this study included a site reconnaissance and subsurface exploration conducted on September 25, 2019, and consisted of one (1) exploratory boring and one (1) CPT sounding. The approximate exploration locations are shown on Figure 2. The CPT sounding was performed on an adjacent parcel, approximately 200 feet north of the subject property. The data from this sounding has been incorporated as part of this feasibility level report and may have reduced applicability in design. Additional soundings should be performed during a design level study.

The CPT sounding was advanced to a depth of approximately 101 feet below the ground surface using a truck-mounted rig with 30-ton reaction capacity. Penetration and pore water pressure data were collected and recorded electronically at intervals of approximately 2 inches while the sounding was being performed. The soil behavior type of the material encountered was assessed using correlations (Robertson, 2009) based on the penetration data. CPT data and the interpreted soil behavior type are presented in Appendix A.

The exploratory boring was advanced with mud-rotary drilling methods to a depth of approximately 44½ feet below the existing grade. A representative of Ninyo & Moore logged the subsurface conditions exposed in the boring and collected relatively undisturbed and bulk soil samples from the boring. The samples were transported to our geotechnical laboratory for testing. The boring was backfilled with grout after sampling and logging were completed. Descriptions of the subsurface materials encountered are presented in the following sections. A detailed log of the boring is presented in Appendix B.

Laboratory testing of soil samples recovered from the boring included tests to evaluate in-situ soil moisture content and dry density, soil gradation, Atterberg limits, consolidation, expansion index, soil corrosivity, and undrained triaxial shear strength. The results of the in-situ moisture content and dry density tests are presented on the boring log in Appendix B. The results of the other laboratory tests are presented in Appendix C.

6 GEOLOGIC AND SUBSURFACE CONDITIONS

6.1 Regional Geologic Setting

The site is located on the south side of San Francisco 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 mountain 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.

6.2 Site Geology

Regional geologic maps by Dibblee (2007) and Witter et al. (2006) indicate that the site is underlain by Holocene age alluvial fan deposits. Soils developed at the surface of the alluvial fan deposits are part of the Hangerone soil series (USDA, 2015). The Hangerone series typically consists of poorly drained, slightly alkaline clay-rich soils that developed on alluvial deposits derived from mixed rock sources.

The alluvial deposits in this area generally consist of silty clay and organic clay that were deposited at the distal edges of alluvial fans emanating from the Santa Cruz Mountains. Lenses of coarser alluvium, consisting of sand and occasional gravel, are present at various depths and are typically elongated in the down valley direction. The alluvial deposits are associated with the alluvial fan that developed along Stevens Creek, which flows into the tidal marsh along the southern shore of San Francisco Bay (Sowers, 2004). According to Sowers (2004), the site was located within a historic grove of willow trees circa 1850. A map of the regional geology is presented as Figure 4 (Dibblee, 2007).

6.3 Subsurface Soil Conditions

The following sections provide a generalized description of the surface materials and geologic units encountered during our subsurface evaluation. More detailed descriptions are presented on the boring log in Appendix B.

6.3.1 Asphalt Pavement

The boring and CPT sounding were advanced through asphalt concrete pavement. The pavement section encountered consisted of approximately 2½ inches of asphalt concrete over approximately 5 inches of aggregate base. Variations in the thickness of the asphalt concrete and aggregate base layers may be encountered due to past maintenance, utility work, or other factors.

6.3.2 Fill

Fill was encountered below the pavement section in the boring and CPT sounding to a depth of approximately 3 feet below the ground surface. The fill generally consisted of moist, firm lean clay.

6.3.3 Alluvium

Alluvium was encountered in the boring and CPT sounding from below the fill to the depths explored. The alluvium generally consisted of moist to wet, firm to very stiff lean clay, and wet, loose to very dense silt, poorly-graded sand with clay, well-graded sand, and poorly graded gravel with silt. More detailed descriptions are presented on the boring log in Appendix B. Soil Behavior Type classifications interpreted from CPT data are presented in Appendix A.

6.4 Groundwater

Groundwater was encountered during our subsurface exploration at a depth of approximately 6 feet below the ground surface in the boring and sounding. The historical high groundwater level for the site is approximately 5 feet below the ground surface (CGS, 2006b).

Fluctuations in the level of groundwater may occur due to variations in ground surface topography, subsurface stratification, rainfall, irrigation practices, groundwater pumping, and other factors which may not have been evident at the time of our field evaluation. In addition, seeps may be encountered at elevations above the groundwater levels encountered due to perched groundwater conditions, leaking pipes, preferential drainage, or other factors not evident at the time of our exploration. Piezometers can be installed to further evaluate the depth to groundwater in the study area and fluctuation in groundwater levels if needed.

7 GEOLOGIC HAZARDS AND GEOTECHNICAL CONSIDERATIONS

This study considered a number of potential issues relevant to the proposed construction on the subject site, including seismic hazards, landsliding, expansive soil, settlement of compressible soil layers, potential of on-site soil to corrode ferrous metals and promote sulfate attack on concrete, and excavation characteristics. These issues are discussed in the following subsections.

7.1 Seismic Hazards

The project site is located within the San Francisco Bay Area, a seismically active region. The seismic hazards considered in this study include the potential for ground surface rupture and ground shaking due to seismic activity, seismically induced liquefaction, dynamic settlement, ground subsidence related to sand boils, lateral spreading, tsunamis, and seiches. These potential hazards are discussed in the following subsections.

7.1.1 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.

7.1.2 Faulting and Ground Surface Rupture

California lies along the boundary between the North American and Pacific tectonic plates. Movement along the plate boundary can generate earthquakes and has created zones of deformation within the Earth's crust. These zones include various types of complex geologic structures and geomorphic features such as folds, faults, sag ponds, shutter ridges, linear valleys, and scarps. During moderate to large magnitude earthquakes, the ground can rupture along well defined zones of deformation where faults intersect the Earth's surface.

In response to hazards associated with ground rupture, or surface displacement, the State of California enacted the Alquist-Priolo Earthquake Fault Zoning Act (AP Act) in 1972, which regulates development of structures for human occupancy in areas within active fault zones. The AP Act requires that the State Geologist delineate zones along active faults where evaluation of the potential for ground rupture is required. As defined by the California Geological Survey (CGS, 2018), active faults are faults that have caused surface displacement within Holocene time, or within approximately the last 11,700 years.

The site is not located within an Alquist-Priolo Earthquake Fault Zone established by the State Geologist (CGS, 2018) to delineate regions of potential ground surface rupture adjacent to active faults. The closest known active fault is the southern segment of the Hayward fault located approximately 9 miles northeast of the site (Santa Clara County [SCC], 2012). The approximate locations of major faults in the region and their geographic relationship to the project vicinity are shown on Figure 3.

Based on our review of the referenced geologic maps, the project site is not underlain by known active faults (i.e., faults that exhibit evidence of surface displacement in the last 11,700 years). Therefore, the potential for ground surface rupture because of faulting at the site is considered low. Lurching or cracking of the ground surface as a result of nearby seismic events is possible.

7.1.3 Strong Ground Motion

Based on historic activity, the potential for future strong ground motion at the site is considered significant. 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, 2016) 7-16 Standard and the 2019 California Building Code (CBC). The MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) was calculated as 0.722g using the Structural Engineer Association of California (SEAOC) and California's Office of Statewide Health Planning and Development (OSHPD) seismic design map tool (SEAOC & OSHPD, 2019). The PGA_M is based on a mapped MCE_G peak ground acceleration of 0.601g for the site and a site coefficient (F_{PGA}) of 1.2 for Site Class D - Default. A site-specific ground motion hazard analysis was not part of our feasibility level study and should be performed during the design level investigation.

7.1.4 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 located within a liquefaction hazard zone established by the California Geological Survey (CGS, 2006a) and by Santa Clara County (SCC, 2012). The seismic hazard zones for the site vicinity are presented on Figure 5. Regional studies of liquefaction susceptibility (Witter et al., 2006) indicate that the liquefaction susceptibility at the site is high.

We encountered deposits of sand and fine-grained soil of low plasticity below the historic high groundwater level during our subsurface exploration. We evaluated the potential for liquefaction in accordance with the methods presented by Boulanger and Idriss (2014) using the CPT data collected during our subsurface exploration and the computer program CLiq (GeoLogismiki, 2018). Our analysis assumed a design groundwater elevation of 5 feet below the ground surface, and considered a seismic event producing a PGA of 0.722g resulting from a Magnitude 7.3 earthquake. Soil with a behavior type index (Ic) of 2.6 or less was evaluated for susceptibility to liquefaction and related hazards. The results of our analysis, presented in Appendix D, indicate that thin layers of sandy soil and fine-grained soil of low plasticity below the assumed groundwater level will liquefy under the considered ground motion. The potential for reduction in foundation bearing capacity due to liquefaction will be a consideration for significant structures. Other consequences of liquefaction, including dynamic settlement, sand-boil-induced ground subsidence, 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.

7.1.5 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 due to liquefaction of saturated soil using the computer program CLiq (GeoLogismiki, 2018) to evaluate the CPT data collected during our field investigation with the methodology of Boulanger and Idriss (2014). Our analysis considered a Magnitude 7.3 earthquake producing a PGA of 0.722g and a design groundwater elevation of 5 feet below the ground surface. The results of our analysis, presented in Appendix D, indicate that the free-field total dynamic settlement following the considered seismic event will be approximately 1¾ inches with negligible dry sand settlement. Differential dynamic settlement is estimated to be about 1 inch over a horizontal distance of approximately 30 feet.

During our preliminary investigations for other sites within the East Whisman area, dynamic settlement values ranging from 1¼ inch to 3½ inches were calculated using the input parameters listed above. Additional CPT soundings performed as part of a design level evaluation may help reduce the calculated variability in dynamic settlement across the proposed development.

Deep foundations or ground improvement can mitigate dynamic settlement concerns for structures. Additional sampling and laboratory testing performed as part of a design level evaluation may provide justification for a reduced I_C cutoff for liquefaction and dynamic settlement analysis which would reduce the estimated dynamic settlement.

7.1.6 Ground Subsidence

Sand boils that occur when liquefied, near-surface soil escapes to the ground surface, can result in ground subsidence due to loss of material that is in addition to dynamic settlement. The Liquefaction Potential Index (LPI) described by Iwasaki et al. (1978) was computed from the results of our liquefaction analysis with the CPT data to evaluate the potential for surface manifestation of liquefaction such as sand boils. The computed values of the LPI, presented in Appendix D, indicate that the potential for surface manifestation of liquefaction or sand boils is moderate. We also evaluated the potential for ground subsidence using the case study data presented by Ishihara (1985). Based on the design PGA and the relative density, thickness and depth of the saturated, loose granular soil encountered during our subsurface exploration; we do not anticipate that sand boils and resulting ground subsidence at the ground surface will occur following a significant seismic event.

7.1.7 Lateral Spread

In addition to vertical displacements, seismic ground shaking can induce horizontal displacements as surficial soil deposits spread laterally by floating atop liquefied subsurface layers. Lateral spread can occur on sloping ground or on flat ground adjacent to an exposed face. Based on the level ground conditions on site and the relatively thin and discontinuous nature of the liquefiable soil encountered, we do not regard lateral spreading as a design consideration.

7.1.8 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 location is not within a tsunami inundation area as shown on the Tsunami Inundation Map for Emergency Planning Map (State of California, 2009). Seiches are waves generated in a large enclosed body of water. Based on the inland location of the site and considering that there are no large enclosed bodies of water nearby, the potential for damage due to tsunamis or seiches is not a design consideration.

7.2 Landsliding and Slope Stability

Based on our background review, the site is not within a mapped landslide or landslide hazard zone (CGS, 2006a). The site and surrounding areas are relatively flat and the proposed improvements do not include grading significant slopes. As such, we do not regard landsliding or slope stability as a design consideration.

7.3 Expansive Soil

Some clay minerals undergo volume changes upon wetting or drying. Unsaturated soil 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 sample of the near-surface soil to evaluate the expansion index. The test was performed in accordance with the American Society of Testing and Materials (ASTM, 2019) Standard D 4829 (Expansion Index). The results of our laboratory test indicate that the expansion index of the sample tested was 65. This result is an indicative of a medium expansion characteristic. The results of the expansion index test are presented in Appendix C.

Based upon this result and the results of testing at nearby sites, it is our opinion that special mitigation measures for expansive soil may be needed for near-surface improvements such as parking lots, hardscape, and minor structures. Mitigation measures may include chemical treatment (lime and/or cement) of the soil or import of non-expansive select fill.

7.4 Static Settlement

Although building loads were not available at the time of this report, based on the results of our subsurface evaluation and laboratory testing, static settlement due to sustained loading is a design consideration. We conducted preliminary engineering analysis of the settlement potential using the consolidation and other laboratory test results, from the boring performed near this parcel and others in the campus, to estimate the column loading that would result in about 1 inch of static settlement, which is a value generally provided as acceptable by structural engineers. For planning purposes, we anticipate that column loads of up to about 400 kips, presuming a bearing pressure of about 2,500 pounds per square feet (psf), will result in a static settlement on the order of 1 inch with a differential of about ½ inch over a lateral span of 30 feet. Deep foundations or ground improvement can mitigate static settlement concerns for structures as needed. A design level subsurface evaluation, laboratory testing, and engineering analysis should be performed once the final building location, configuration of the structure, design column loads and structural design tolerances are known.

7.5 Corrosivity

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 where the soil contains chloride concentration of 500 ppm or greater, soluble sulfate concentration of 1,500 ppm or greater, electrical resistivity of 1,100 ohm-centimeters or less, and a pH of 5.5 or less (Caltrans, 2018).

Based on these criteria, the near-surface soils at the site meet the definition of a corrosive environment. A corrosion engineer can be consulted to further assess the potential for corrosion and provide recommendations for supplementary measures as needed. Based on the criteria used to evaluate the deleterious nature of soil on concrete and recommendations from the American Concrete Institute (ACI, 2014) for sulfate exposure classes, the soil on site is defined as Exposure Class S0.

7.6 Excavation Characteristics

We anticipate that the proposed project will involve excavations of up to 10 feet in depth for installation of utilities, grading, and shallow foundation construction. The soil encountered during our subsurface exploration generally consisted of moist to wet, firm to very stiff lean clay, and wet, loose to very dense silt, poorly-graded sand with clay, well-graded sand, and poorly graded gravel with silt. Near-vertical cuts in the soil should not be considered stable due to a shallow groundwater level. Groundwater was encountered at a depth of approximately 6 feet below existing ground surface, but could rise to shallower depths. Regional studies (CGS, 2006b) indicate that the depth to historic high groundwater is about 5 feet below the ground surface.

Excavations extending near or below groundwater may be unstable without dewatering to depress the water level. Excavations in the fill may encounter debris, rubble, oversize material, buried objects, or other potential obstructions.

We anticipate excavations of up to approximately 90 feet for deep foundation construction. Difficult drilling conditions for continuous flight augers or drilled displacement equipment may be encountered in dense sand or gravelly alluvial material. Excavations for deep foundation construction should not be considered stable due to a shallow groundwater level and the presence of granular soil layers. Drilled excavations should be cased or stabilized with slurry during foundation construction.

8 CONCLUSIONS

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

- Our subsurface exploration, consisting of a limited number of subsurface exploration points including information from adjacent parcels, encountered undocumented fill and alluvium. Fill was encountered to a depth of about 3 feet. The fill generally consisted of moist, firm lean clay. The alluvium generally consisted of moist to wet, firm to very stiff lean clay, and wet, loose to very dense silt, poorly-graded sand with clay, well-graded sand, and poorly graded gravel with silt.

- Groundwater was encountered at a depth of approximately 6 feet below the existing grade during our subsurface exploration. Variations in the groundwater level across the site and over time should be anticipated. Regional mapping indicates that the historic high groundwater level is approximately 5 feet below the existing grade.

- The earth materials underlying the site should be excavatable with conventional earth moving equipment in good working condition. Caving conditions could occur, particularly in granular material below groundwater. Casing should be anticipated for drilling of piles.

- The site will experience a relatively large degree of ground shaking during a significant earthquake on a nearby fault.

- The results of our liquefaction analysis, presented in Appendix D, indicate that layers of sandy soil and fine-grained soil of low plasticity below the assumed groundwater level will liquefy under the considered ground motion. The potential for reduction in foundation bearing capacity due to liquefaction will be a consideration for significant structures.

- The results of our dynamic settlement analysis, presented in Appendix D, indicate that the total dynamic settlement following the considered seismic event will be approximately 1¾ inches with a differential dynamic settlement of about 1 inch over a horizontal distance of approximately 30 feet. Deep foundations or ground improvement can mitigate dynamic settlement concerns for structures. The results are based on a limited number of exploration points and the results may vary in a design level study.

- Lateral spreading is not a design consideration based on the subsurface conditions encountered.
- 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 are exposed to water, extend below groundwater, or encounter granular soil may be unstable and prone to sloughing.
- Excavations in the fill may encounter debris, rubble, oversize material, buried objects, or other potential obstructions. Difficult drilling conditions for continuous flight augers or drilled displacement equipment may be encountered in dense sand or gravelly alluvial material.
- Expansion Index testing indicates that the near-surface soil on site has a medium expansion characteristic. Based upon this result and results from nearby sites, special mitigation measures for expansive soil may be needed for near-surface improvements. This result is based on a limited subsurface exploration and could vary across the site.
- Our limited laboratory corrosion testing indicates that the near-surface site soils should be considered corrosive based on California Department of Transportation (Caltrans, 2018) corrosion guidelines. This result is based on a limited subsurface exploration and could vary across the site.

As previously discussed, the findings and preliminary recommendations provided in this report are based on a limited subsurface evaluation. For a design level geotechnical evaluation, Ninyo & Moore should perform additional subsurface exploration, laboratory testing, and engineering analysis to prepare recommendations for the design and construction of the project prior to the preparation of design documents.

9 PRELIMINARY RECOMMENDATIONS

The following recommendations are preliminary and are intended for planning purposes. The results are based on a limited geotechnical evaluation. A complete geotechnical evaluation should be performed once development details are available.

9.1 Earthwork

Preliminary earthwork recommendations are presented below. Evaluations performed by the geotechnical consultant during the design phase evaluation and over the course of operations may result in new recommendations, which could supersede the recommendations in this section.

Expansive soils were encountered during our preliminary investigations for sites within the East Whisman area. Additional borings and laboratory testing performed as part of a design level evaluation can be used to further evaluate the potential for expansive soils across the proposed development.

In general, we anticipate that the on-site soils will be suitable for use as general fill provided the material is free of rocks or lumps in excess of 6 inches in diameter, trash, debris, roots, vegetation or other deleterious material. On-site materials may need to be dried out before re-use as fill.

Recommendations for placement and compaction of engineered fill will be included as part of a design level geotechnical evaluation once details of the proposed construction are known.

9.2 Construction Dewatering

Groundwater was encountered during our subsurface exploration at a depth of approximately 6 feet. Regional maps indicate that the historic high groundwater level in the site vicinity is around 5 feet below the ground surface. Variations in groundwater levels across the site and over time should be anticipated. Water intrusion into the excavations may occur as a result of groundwater intrusion 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.

9.3 Foundations

Presented below are suitable foundation types for planning purposes based on our feasibility level study. Details of the proposed construction, including site layout and anticipated column loads, were not available at the time of preparation of this report. A design level geotechnical report will be prepared once the final building location, configuration of the structure, design column loads and structural design tolerances are known. Based on the results of this feasibility level geotechnical evaluation and our experience in the area, we anticipate that buildings of one or two stories in height may be supported on spread footings with slab-on-grade floors. If the estimated liquefaction induced dynamic settlement listed in section 7.1.5 is considered to exceed the structural tolerance of the building, ground improvement may be performed below the building foundations or deep foundations can be used. Ground improvement considerations are provided in Section 9.3.3. Additional sampling and laboratory testing performed as part of a design level evaluation may provide justification for a reduced I_C cutoff for liquefaction and dynamic settlement analysis which would reduce the estimated dynamic settlement.

Foundations 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.

9.3.1 Shallow Foundations and Slabs-on-Grade

We anticipate that new structures of one to two stories may be supported on shallow foundations consisting of spread footings with slab-on-grade floors provided that the estimated dynamic settlement values presented in Section 7.1.5 and the estimated static settlement values presented in Section 7.4 are tolerable for the structural system. A design level geotechnical investigation should be performed to evaluate the preliminary estimated settlement values once construction plans are available.

9.3.2 Deep Foundations

Deep foundations can mitigate concerns related to liquefaction and settlement under foundation loads. We anticipate that deep foundations could include driven concrete piles or auger-cast piles. Based on our liquefaction and dynamic settlement analysis, we anticipate that the pile tips will extend to a depth of 60 feet, or more.

Pile driving may induce vibrations in adjacent structures, and may also heave adjacent structures or previously driven piles. Pre-drilling portions of the pile embedment depth may reduce the vibration and heave on adjacent structures and previously driven piles. Consideration should be given to implementing instrumentation and monitoring programs to document existing conditions and to monitor movements and vibrations during construction, particularly where construction activities will be close to improvements that are sensitive to ground deformation or construction vibration.

Auger cast piles are cast-in-place foundations that are generally constructed by drilling a shaft in one pass with a hollow-stem auger, injecting cement grout through the hollow stem to fill the shaft as the auger is withdrawn from the excavation, then lowering a cage of reinforcing steel into the grout-filled shaft. Methods for constructing auger cast piles include utilizing continuous flight augers (CFA) and drilled displacement (DD) techniques.

CFA piles are constructed using an auger with continuous flight and a consistent shaft diameter. The auger is advanced and rotated in a controlled fashion so that the cuttings are not transported up the auger but remain on the flights to stabilize the borehole during auger advancement. Once the tip elevation is achieved, the auger is pulled out of the hole with no rotation to remove the cuttings while the grout is injected into the hole.

DD piles are constructed utilizing an auger with a shaft diameter that increases with distance above the cutting head. The increasing shaft diameter displaces the excavated soil laterally as the auger is advanced to increase the density of the soil around the excavation and reduce the quantity of drill cuttings produced. DD piles that utilize an auger with a shaft diameter that increases to meet the flighting diameter, can be considered “full displacement” piles. DD piles may be constructed as full or partial displacement piles with continuous or limited flight. Augers with limited flight generally include a section with reversed flights above the displacement body to gather and displace sloughed soil as the auger is rotated out of the hole.

9.3.3 Ground Improvement

Static settlement and liquefaction and dynamic settlement concerns can be mitigated through ground improvement. Detailed design of the soil improvement, including construction procedures, equipment, and the size and spacing of the improvement should be prepared by a specialty contractor to meet the project objectives. In general, we anticipate that ground improvement methods could include vibro stone columns, rammed aggregate piers, drilled displacement columns, or deep soil mixing. Based on our liquefaction and dynamic settlement analysis, we anticipate the ground improvement will extend to depths of 47 feet, or more.

Vibro stone columns involves the insertion of crushed stone in a grid pattern with a vibratory probe. The strength of the soil mass is increased due to the reinforcement of crushed stone and densification of surrounding soils. In addition, the potential for liquefaction of the subsurface soils is reduced with the improved drainage provided by the stone columns.

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

Drilled displacement columns consist of a grid of a grout columns installed beneath the building footprint. They are constructed with similar methods as drilled displacement auger-cast piles but typically do not include steel reinforcement and are not structurally connected to the building foundation. An aggregate cushion is typically constructed between the top of the grout columns and the foundation.

Deep soil mixing consists of mechanically mixing subsurface soils with a cementitious binder to create a grid of soil-cement columns. Deep soil mixing utilizes specialty auger drills to shear the soil and inject cementitious slurry which improves soil properties and reduces dynamic and static settlement potential. The slurry can be injected during the penetration and withdrawal phases. Upon curing, a conventional shallow foundation can be constructed.

10 LIMITATIONS

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

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

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

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

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

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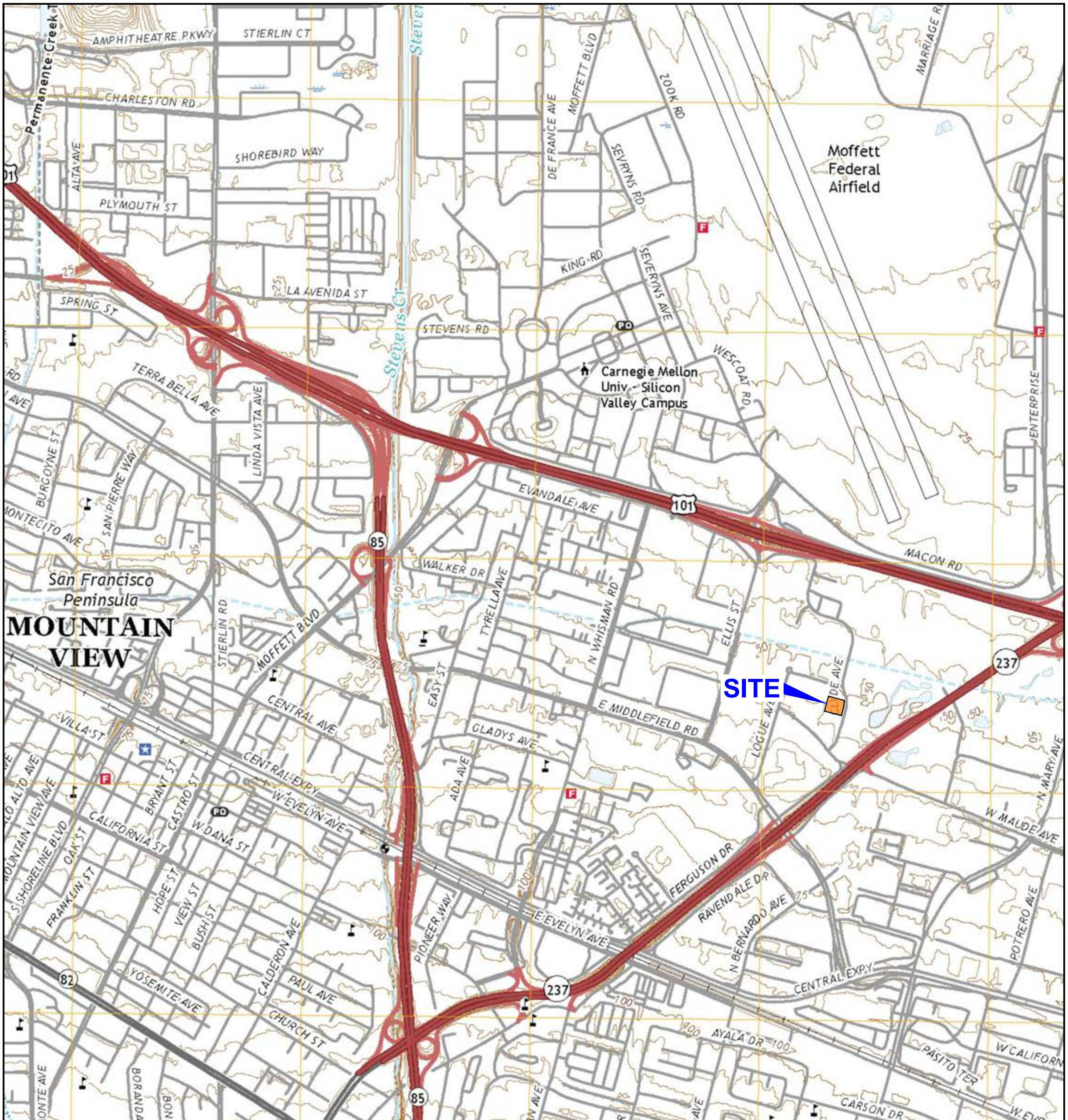
FIGURES



NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE I REFERENCE: USGS, 2018

N

SCALE (FEET)

0 2,000 4,000

403253010_SL.dwg 10/16/2019 AEK

FIGURE 1

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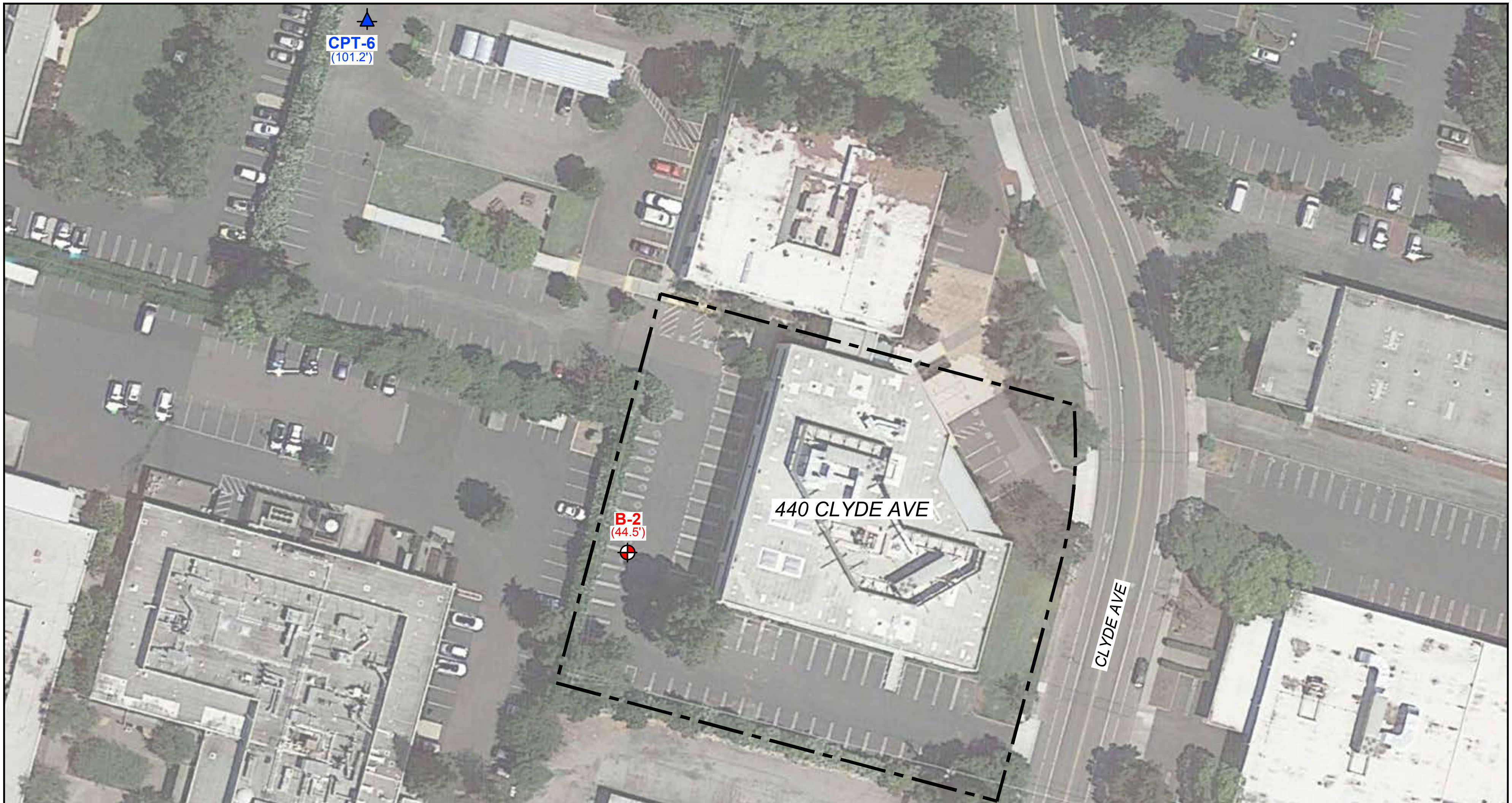
SITE LOCATION

EAST WHISMAN
440 CLYDE AVENUE
MOUNTAIN VIEW, CALIFORNIA

403253010 I 04/20



LEGEND

B-2 (44.5') BORING LOCATION TOTAL DEPTH, IN FEET

CPT-6 (101.2') CPT LOCATION TOTAL DEPTH, IN FEET

SITE BOUNDARY

N

SCALE (FEET)

0 80 160

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE I REFERENCE: GOOGLE EARTH, 2019

403253010_EL.dwg 10/28/2019 AEK

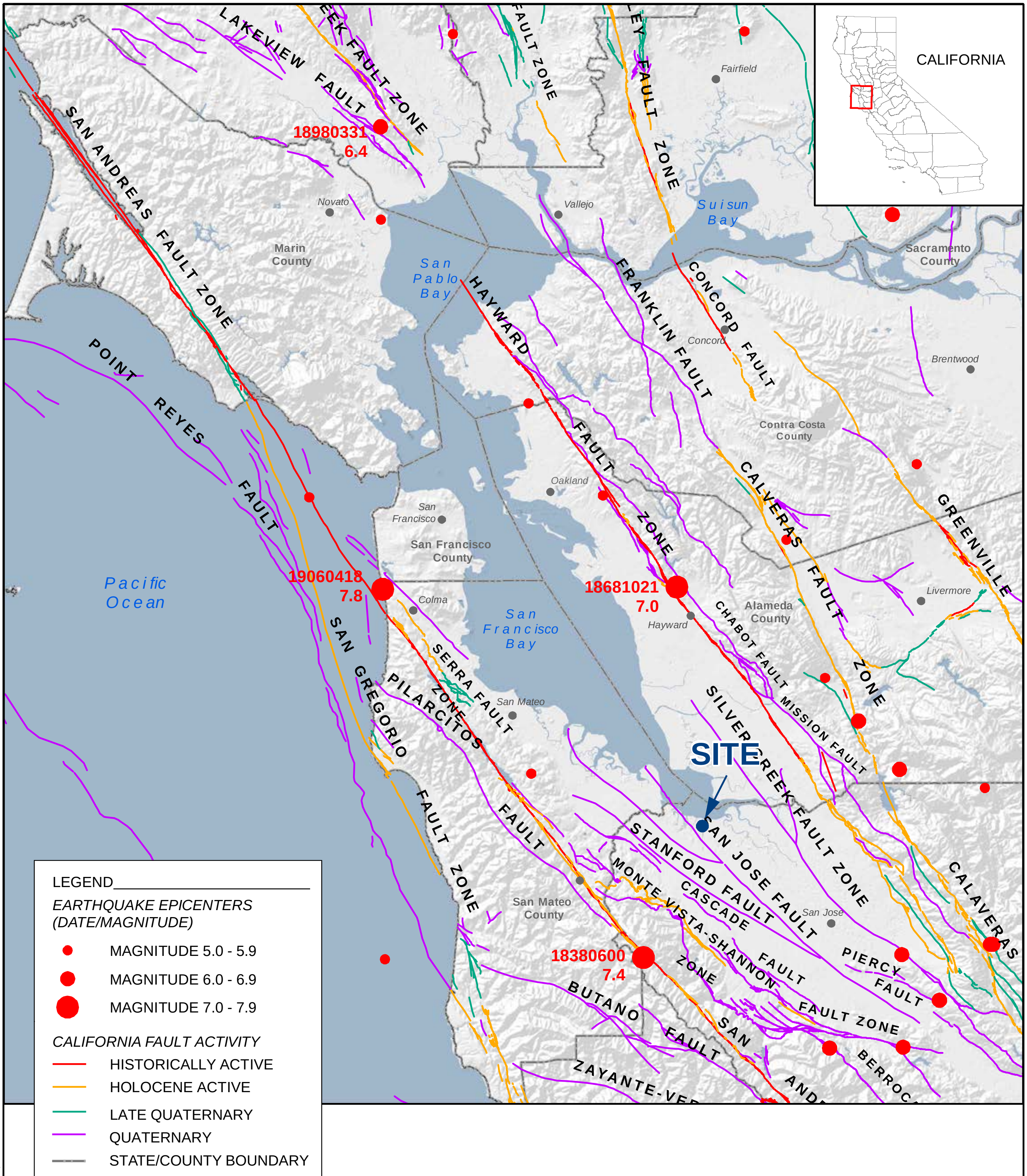
FIGURE 2

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

EXPLORATION LOCATIONS

EAST WHISMAN
440 CLYDE AVENUE
MOUNTAIN VIEW, CALIFORNIA
403253010 I 04/20



NOTE: DIRECTIONS, DIMENSIONS, AND LOCATIONS ARE APPROXIMATE

SOURCES: CALIFORNIA GEOLOGICAL SURVEY, 2010, FAULT ACTIVITY MAP OF CALIFORNIA
CALIFORNIA GEOLOGICAL SURVEY, 2000, MAP SHEET MS 49

N

SCALE (MILES)
0 10 20

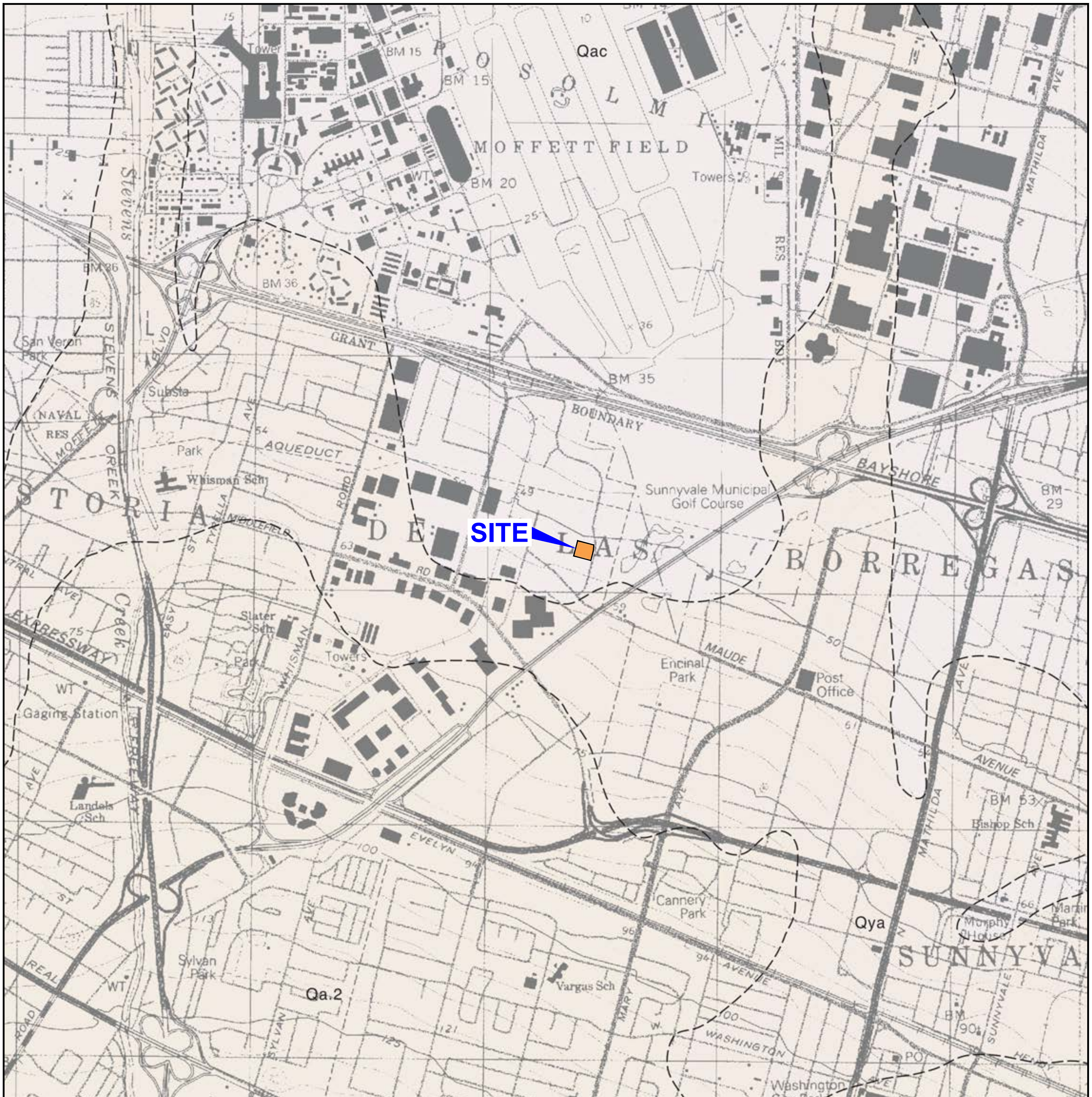
403253010_FL.pdf 10/07/2019 AEK

FIGURE 3

FAULT LOCATIONS AND EARTHQUAKE EPICENTERS

EAST WHISMAN
440 CLYDE AVENUE
MOUNTAIN VIEW, CALIFORNIA
403253010 | 04/20

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



LEGEND

Qa.2	ALLUVIAL GRAVEL, SAND, SILT, AND CLAY (HOLOCENE)
Qya	ALLUVIAL SAND (HOLOCENE)
Qac	SILTY & ORGANIC CLAY (HOLOCENE)

N

SCALE (FEET)

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE I REFERENCE: DIBBLEE, 2007

403253010_RG.dwg 10/18/2019 AEK

FIGURE 4

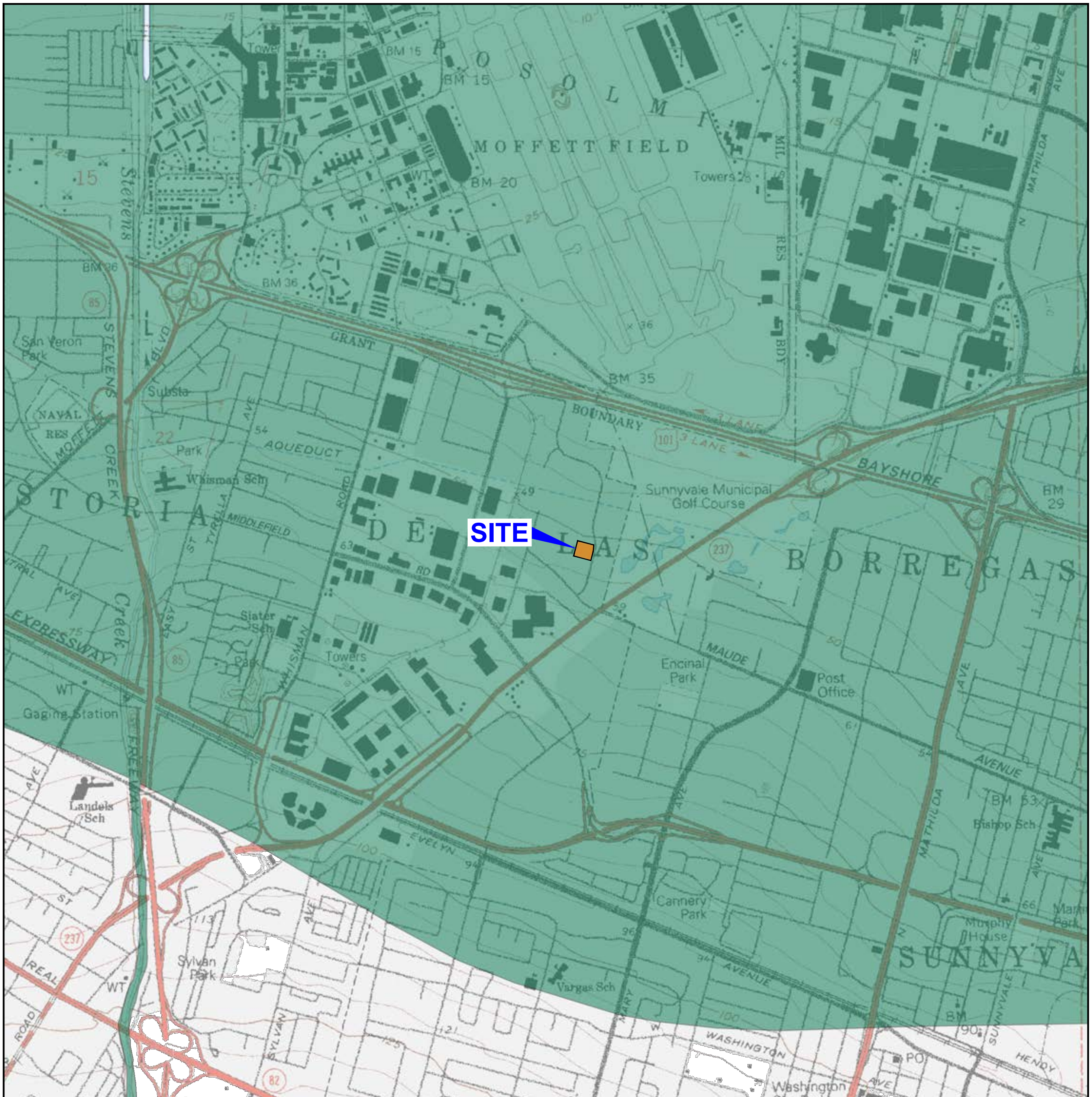
Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

REGIONAL GEOLOGY

EAST WHISMAN
440 CLYDE AVENUE
MOUNTAIN VIEW, CALIFORNIA

403253010 I 04/20



LEGEND

LIQUEFACTION:

Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE | REFERENCE: CGS, 2006

N

SCALE (FEET)

0 2,000 4,000

403253010_SHZ.dwg 10/18/2019 AEK

FIGURE 5

SEISMIC HAZARD ZONES

EAST WHISMAN
440 CLYDE AVENUE
MOUNTAIN VIEW, CALIFORNIA

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APPENDIX A

Cone Penetration Testing

APPENDIX A

CONE PENETRATION TESTING

<u>Field Procedure for Cone Penetration Testing</u>

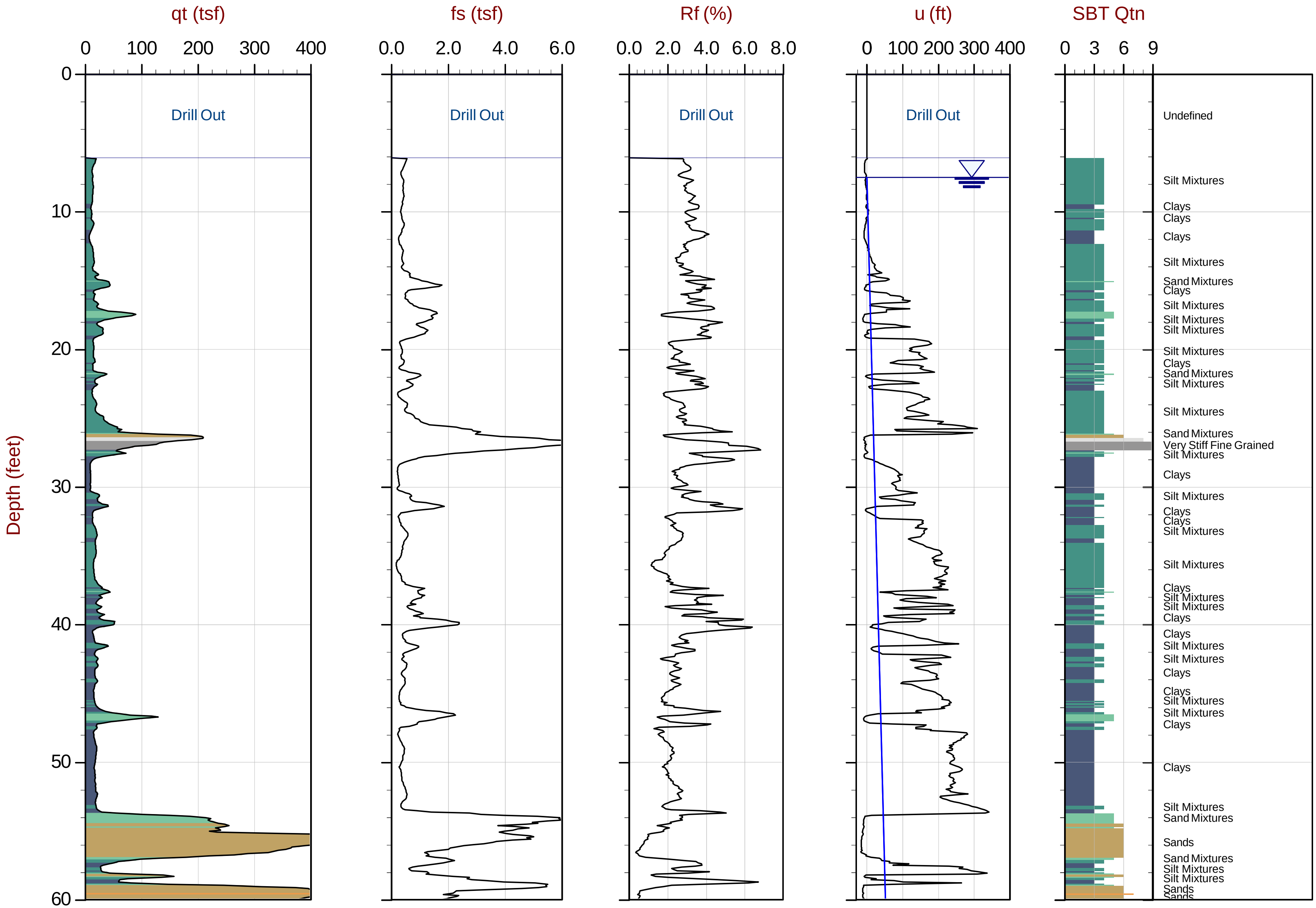
A penetrometer with a conical tip having an apex angle of 60 degrees and a cone base area of 15 square centimeters was hydraulically pushed through the soil using the reaction mass of a 30-ton rig at a constant rate of about 20 millimeters per second in accordance with ASTM D 5778. The penetrometer was instrumented to measure, by electronic methods, the water pressure acting on a transducer near the cone tip, the force on the conical point required to penetrate the soil, and the force on a friction sleeve behind the cone tip as the penetrometer was advanced. Penetration and pore water pressure data (P_w) was collected and recorded electronically at intervals of approximately 1 inch. Cone resistance (Q_t) 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 (R_f) was calculated as the ratio of the tip resistance to the sleeve friction (Q_t/F_s). A graph of the computed values of cone resistance (Qt), friction ratio (Fs/Qt), and pore water pressure (U) 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 and Campanella (1986). Equivalent SPT blowcounts at a 60 percent energy ratio with overburden correction ($N_{1(60)}$ values) were calculated from the tip resistance and friction ratio. A graph of the equivalent $N_{1(60)}$ values and the encountered soil types are also presented on the logs in the following pages.

CONETEC

Ninyo & Moore

Job No: 19-56150
Date: 2019-09-25 08:22
Site: East Whisman

Sounding: CPT-06
Cone: 446:T1500F15U500



Max Depth: 30.850 m / 101.21 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56150_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139459m E: 584235m

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

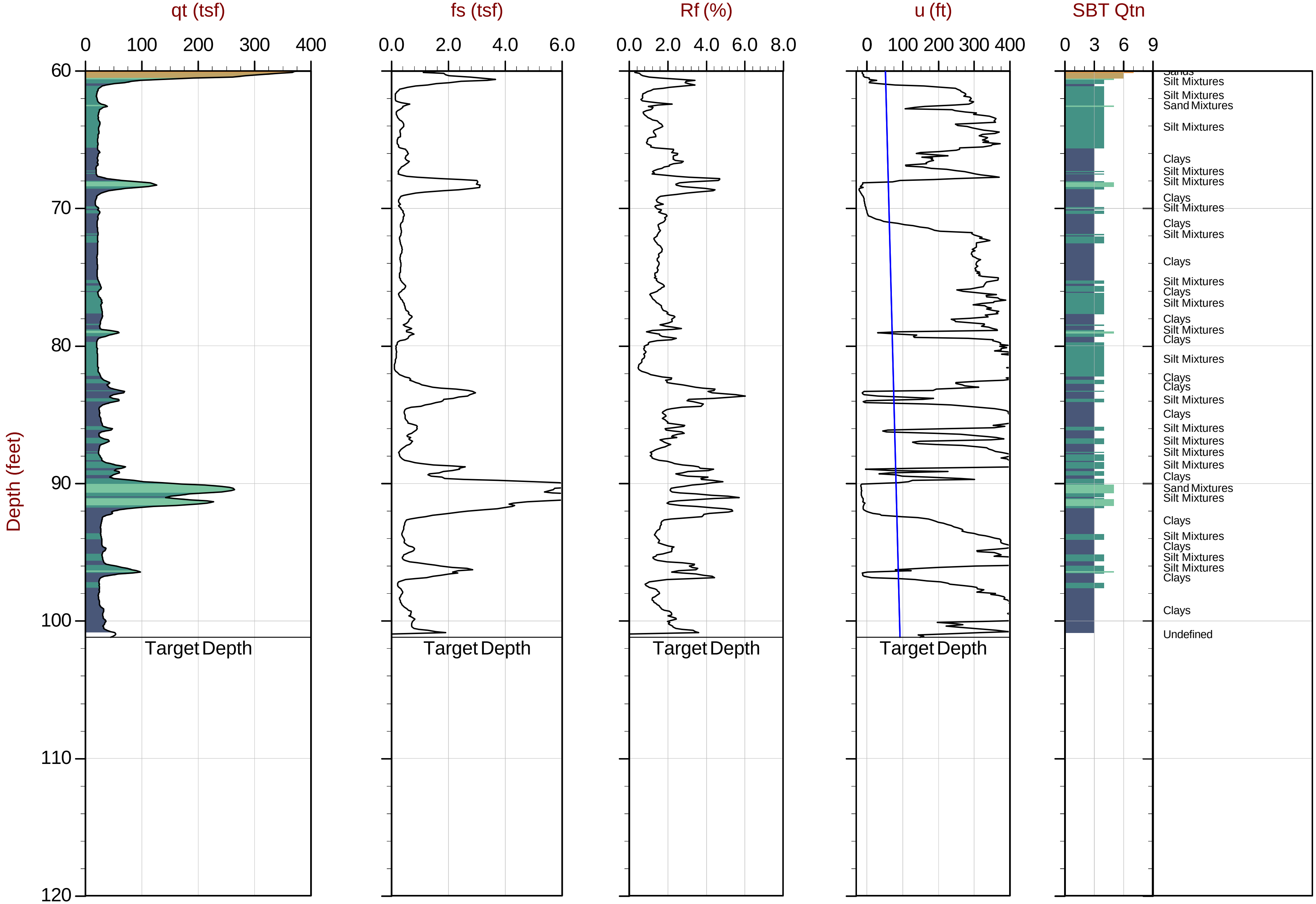
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Ninyo & Moore

Job No: 19-56150
Date: 2019-09-25 08:22
Site: East Whisman

Sounding: CPT-06
Cone: 446:T1500F15U500



Max Depth: 30.850 m / 101.21 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56150_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139459m E: 584235m

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

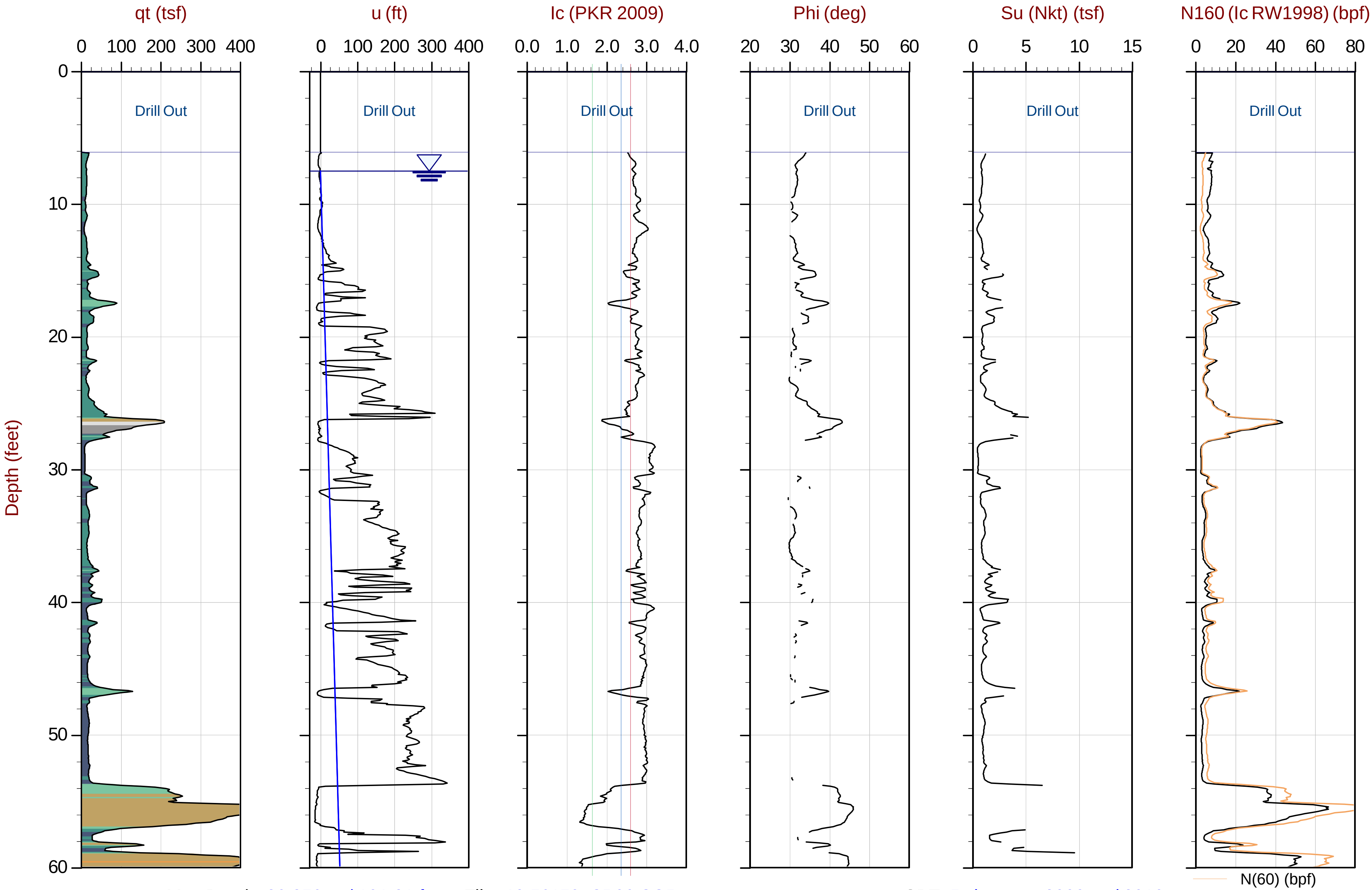
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Ninyo & Moore

Job No: 19-56150
Date: 2019-09-25 08:22
Site: East Whisman

Sounding: CPT-06
Cone: 446:T1500F15U500



Max Depth: 30.850 m / 101.21 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56150_CP06.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

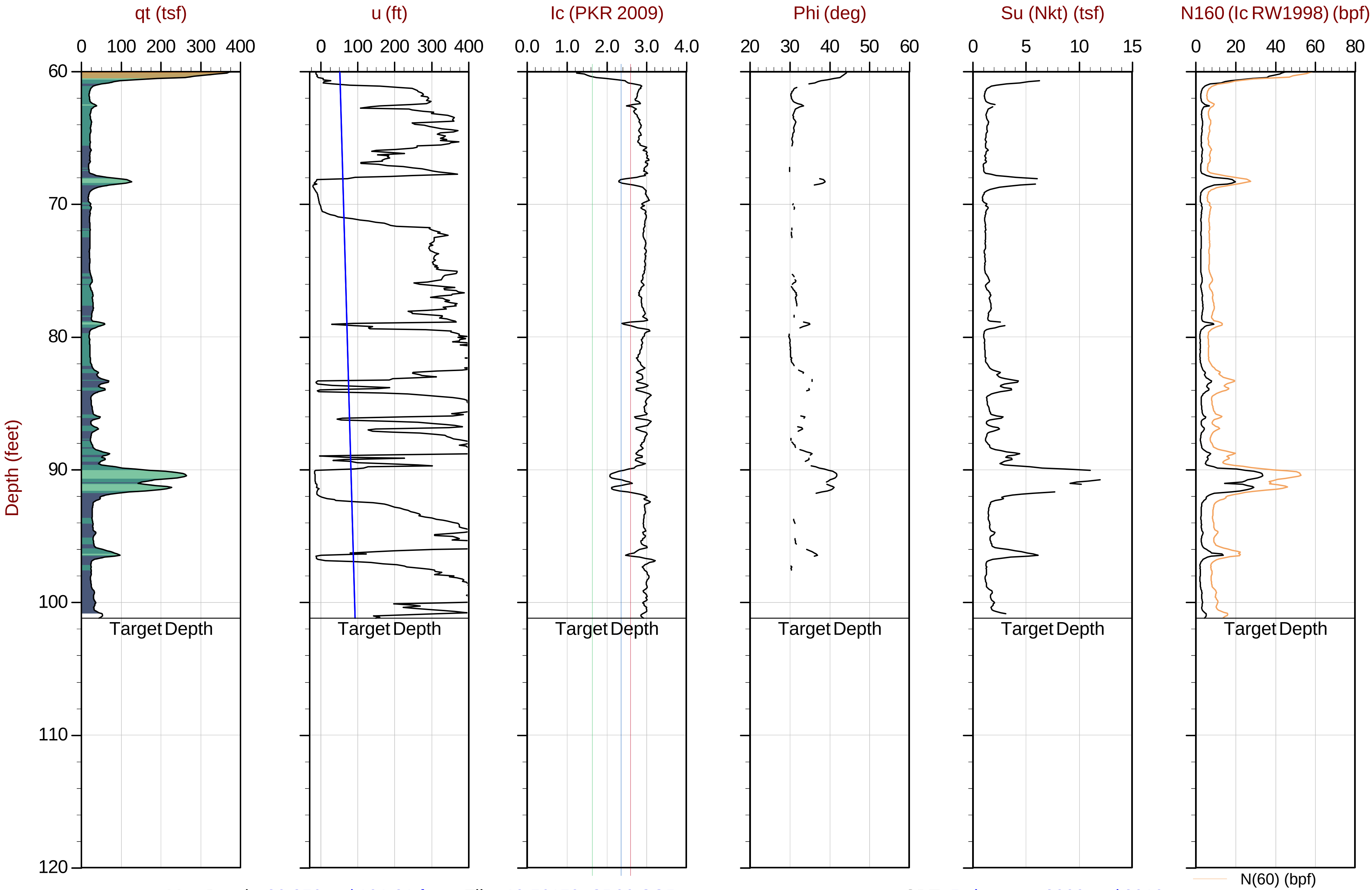
SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139459m E: 584235m

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

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

CONETEC

Ninyo & Moore	Job No: 19-56150	Sounding: CPT-06
	Date: 2019-09-25 08:22	Cone: 446:T1500F15U500
	Site: East Whisman	



Max Depth: 30.850 m / 101.21 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56150_CP06.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139459m E: 584235m

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

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

APPENDIX B

Boring Log

APPENDIX B

BORING LOG

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 boring. 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 falling freely from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples
Relatively undisturbed soil samples were obtained in the field using the following method.

Modified Split-Barrel Drive Sampler
Relatively undisturbed soil samples were obtained in the field using a 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 logs 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.

The Shelby Tube Sampler
The Shelby tube sampler is a seamless, thin-walled, steel tube having an external diameter of 3.0 inches and a length of 30 inches. The tube was connected to the drill rod and pushed into an undisturbed soil mass to obtain a relatively undisturbed sample of cohesive soil in general accordance with ASTM D 1587. When the tube was almost full (to avoid overpenetration), it was withdrawn from the boring, removed from the drill rod or hand tool, sealed at both ends, and transported to the laboratory for testing.

DATE DRILLED 9/25/2019 BORING NO. B-2

GROUND ELEVATION 52'±(MSL) SHEET 1 OF 2

METHOD OF DRILLING 4" Mud Rotary, PD Failing 1500 (Pitcher), 3" HA top 6'

DRIVE WEIGHT 140 lbs (automatic trip hammer) DROP 30 inches

SAMPLED BY KCC LOGGED BY KCC REVIEWED BY PCC

DEPTH (feet)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0					ASPHALT CONCRETE: Approximately 2.5 inches thick
	Qc=30			CL	AGGREGRATE BASE: Approximately 5 inches thick.
	Qc=20				
	Qc=21				
	12/12	21.3	94.8	CL	FILL: Dark gray, moist, firm, sandy lean CLAY; trace gravel.
	Qc=20				ALLUVIUM: Brown to light gray, moist, firm, lean CLAY.
	30/20				Light gray; stiff.
10	5				Light gray with orange and black staining; wet; firm.
	30/30	24.4	99.7		Brown; stiff; trace sand.
20	19	19.4	108.3		Gray; very stiff; increase in sand content.
				ML	Gray, wet, medium dense, sandy SILT.
				SP-SC	Gray, wet, very dense, poorly graded SAND with clay and gravel.
	39				
				CL	Gray, wet, stiff, sandy lean CLAY.
30	11	8.2	92.9	GP-GM	Gray, wet, loose, poorly graded GRAVEL with silt.
				CL	Light gray, wet, stiff, sandy lean CLAY.
	13	21.6	102.3		
				SW	Gray, wet, medium dense, well-graded SAND; trace gravel.
40					

FIGURE B- 1

EAST WHISMAN
MOUNTAIN VIEW, CALIFORNIA
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DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 9/25/2019 BORING NO. B-2 GROUND ELEVATION 52'±(MSL) SHEET 2 OF 2 METHOD OF DRILLING 4" Mud Rotary, PD Failing 1500 (Pitcher), 3" HA top 6' DRIVE WEIGHT 140 lbs (automatic trip hammer) DROP 30 inches SAMPLED BY KCC LOGGED BY KCC REVIEWED BY PCC **DESCRIPTION/INTERPRETATION**
40			12				CL	ALLUVIUM: (continued) Gray, wet, stiff, lean CLAY.
			21					Very stiff.
								Total depth = 44.5 feet. Backfilled with cement grout on 9/25/2019. Notes: Depth to groundwater obscured by method of drilling. 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.
50								
60								
70								
80								

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 log of the exploratory boring in Appendix B.

In-Place Moisture and Density Tests
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory boring were evaluated in accordance with ASTM D 2937. The test results are presented on the log of the exploratory boring in Appendix B.

Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in accordance with ASTM D 422. The grain-size distribution curves are shown on Figures C-1 through C-3. The test results were utilized in evaluating the soil classifications in accordance with the USCS.

Atterberg Limits
Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in 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-4.

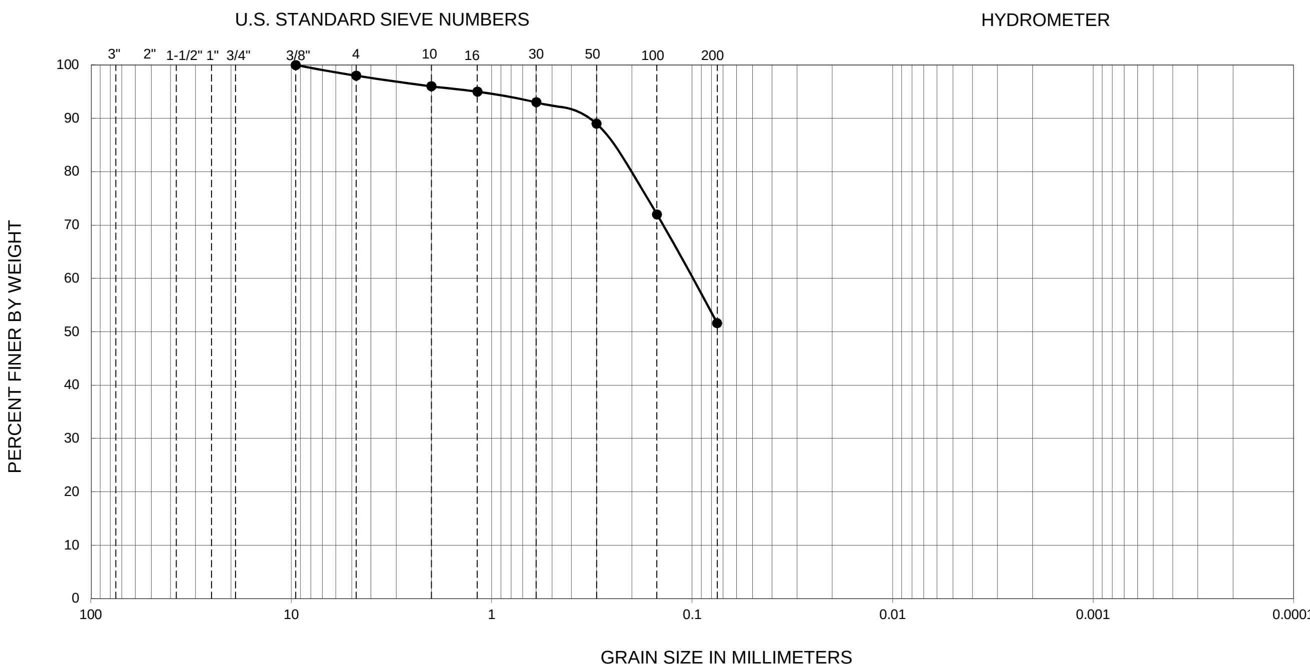
Consolidation Test
A consolidation test was performed on a selected relatively undisturbed soil sample in accordance with ASTM D 2435. The sample was inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The test results are presented on Figure C-5.

Expansion Index Test
The expansion index of a selected material was evaluated in 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-6.

Soil Corrosivity Tests
Soil pH, and resistivity tests were performed on a representative sample in accordance with California Test (CT) 643. The soluble sulfate and chloride content of the selected sample was evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure C-7.

Unconsolidated Undrained Triaxial Compression Test
Triaxial compression tests were performed on selected relatively undisturbed samples in accordance with ASTM D 2850. The test results are shown on Figure C-8.

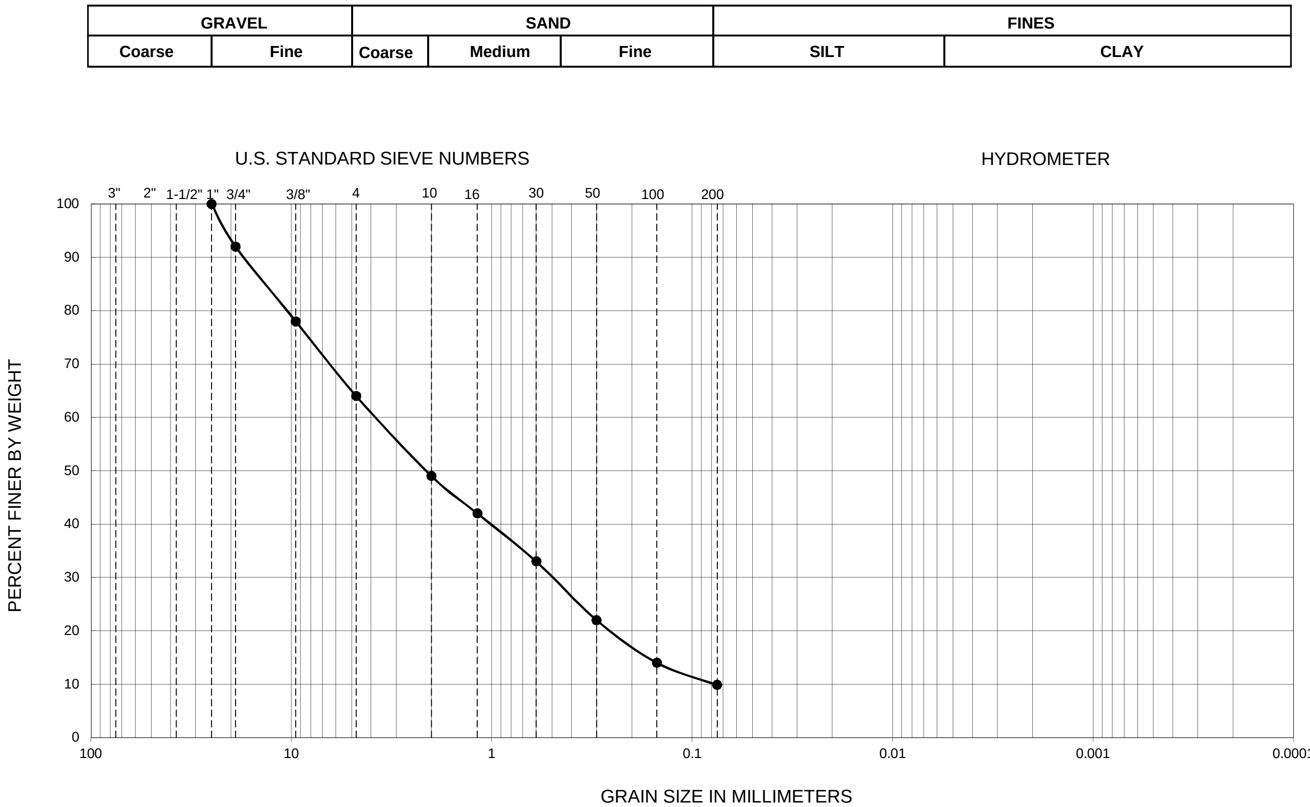
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



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	21.0-21.5	--	--	NP	--	--	--	--	--	52	ML

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

NP - INDICATES NON-PLASTIC



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	25.0-26.5	--	--	--	0.08	0.50	3.80	50.7	0.9	10	SP-SC

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

FIGURE C-2

GRADATION TEST RESULTS

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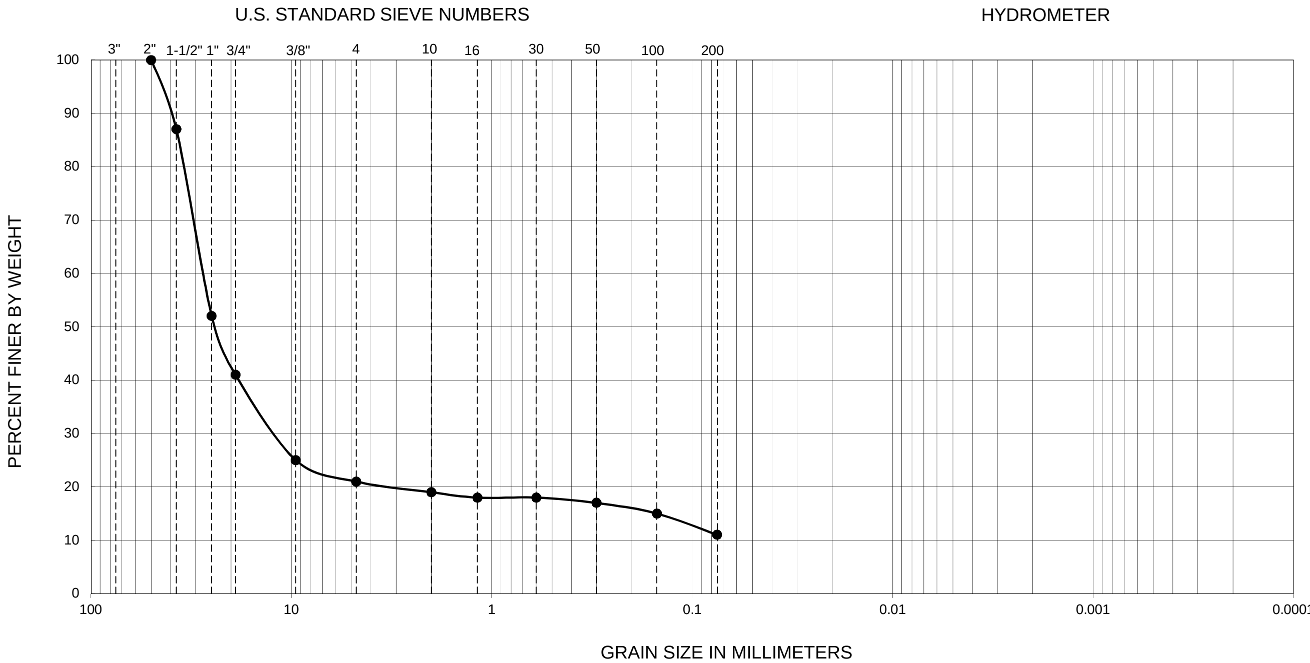
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



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	31.0-31.5	--	--	--	0.075	12.5	27.9	372.0	74.7	11	GP-GM

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FIGURE C-3

GRADATION TEST RESULTS

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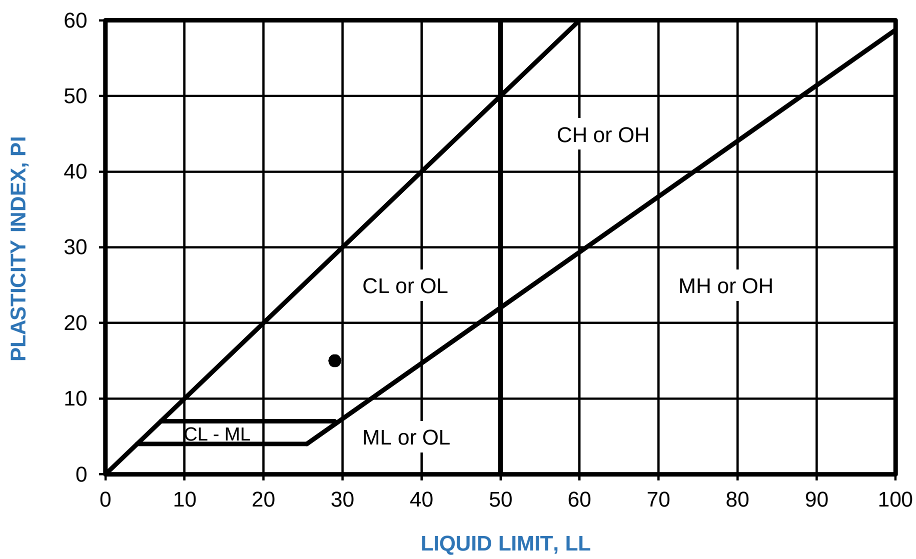
SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-2	10.0-11.5	29	14	15	CL	CL
■	B-2	21.0-21.5	--	--	NP	ML	ML

NP - INDICATES NON-PLASTIC



PERFORMED IN ACCORDANCE WITH ASTM D 4318

FIGURE C-4

ATTERBERG LIMITS TEST RESULTS

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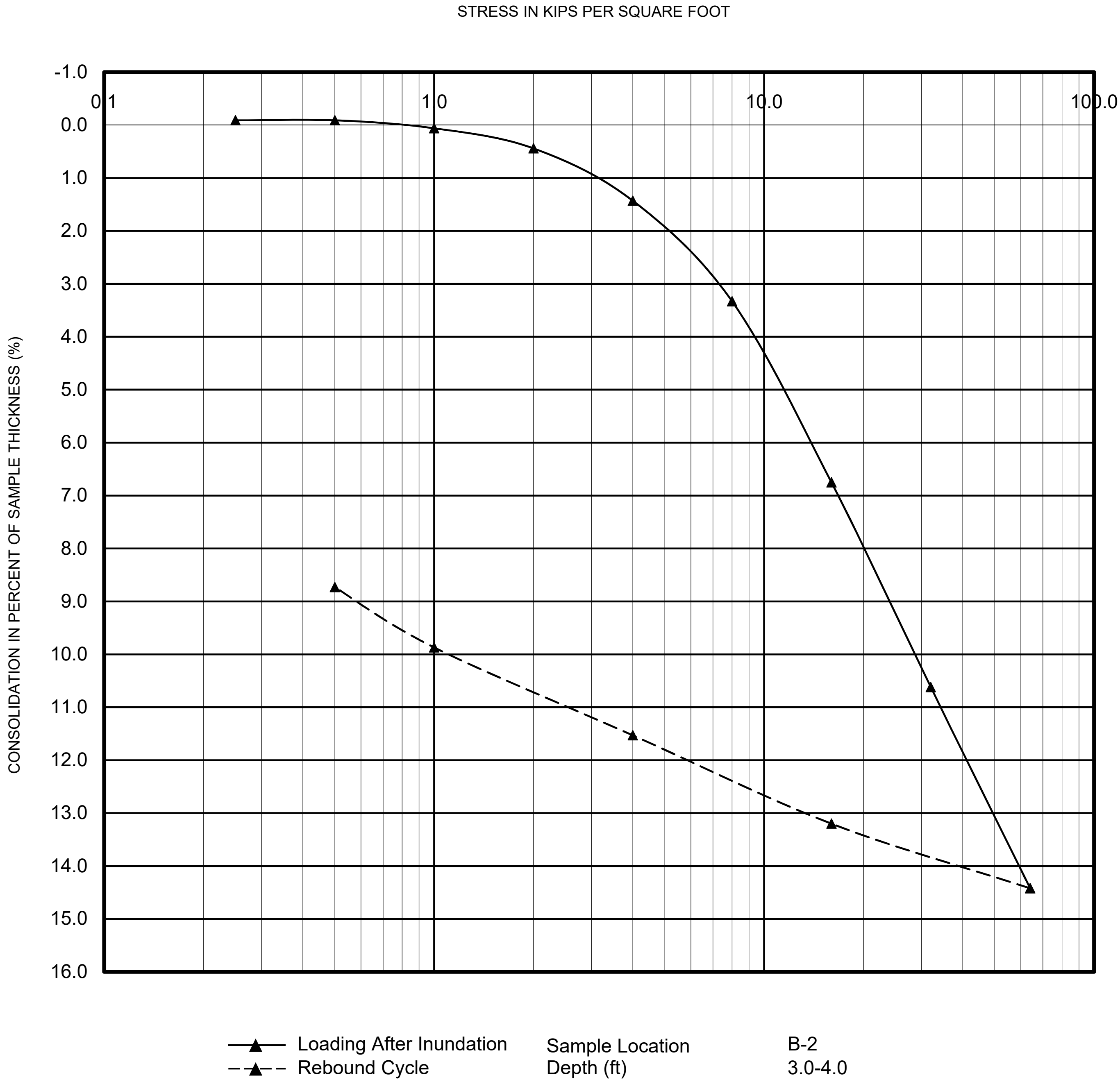

Sample Location	B-2
Depth (ft)	3.0-4.0
Soil Type	CL

PERFORMED IN ACCORDANCE WITH ASTM D 2435

FIGURE C-5

CONSOLIDATION TEST RESULTS

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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	0.6-3.0	9.4	112.0	21.8	0.065	65	Medium

PERFORMED IN ACCORDANCE WITH ASTM D 4829

EXPANSION INDEX TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-2	0.6-3.0	6.6	750	1320	0.132	540

[1] PERFORMED IN ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN ACCORDANCE WITH CALIFORNIA TEST METHOD 422

CORROSIVITY TEST RESULTS

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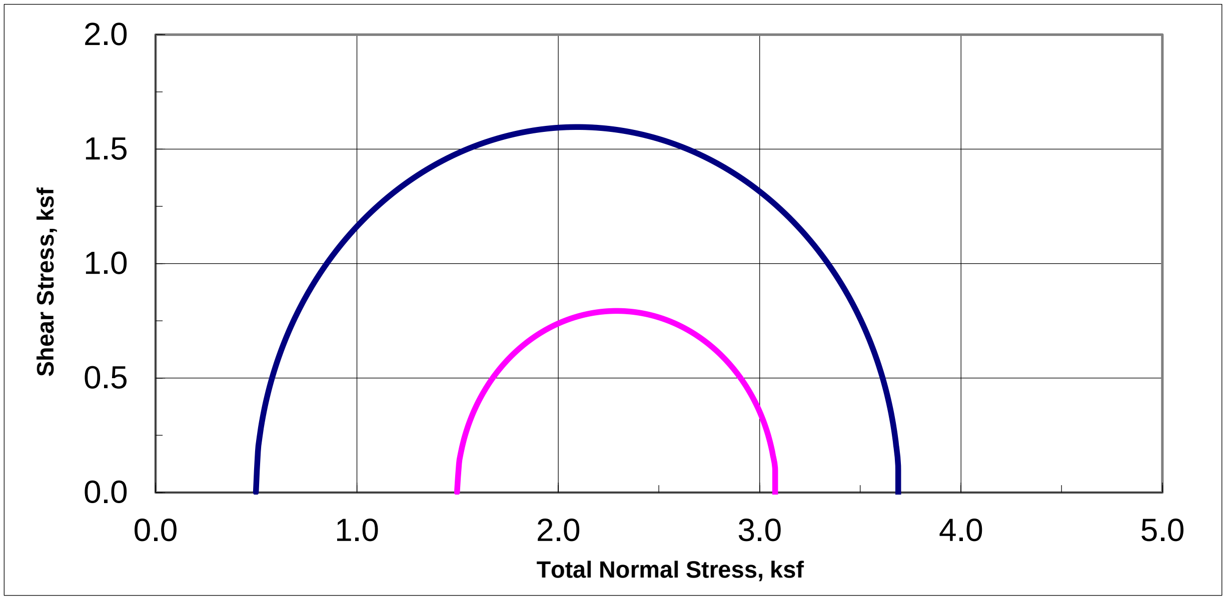

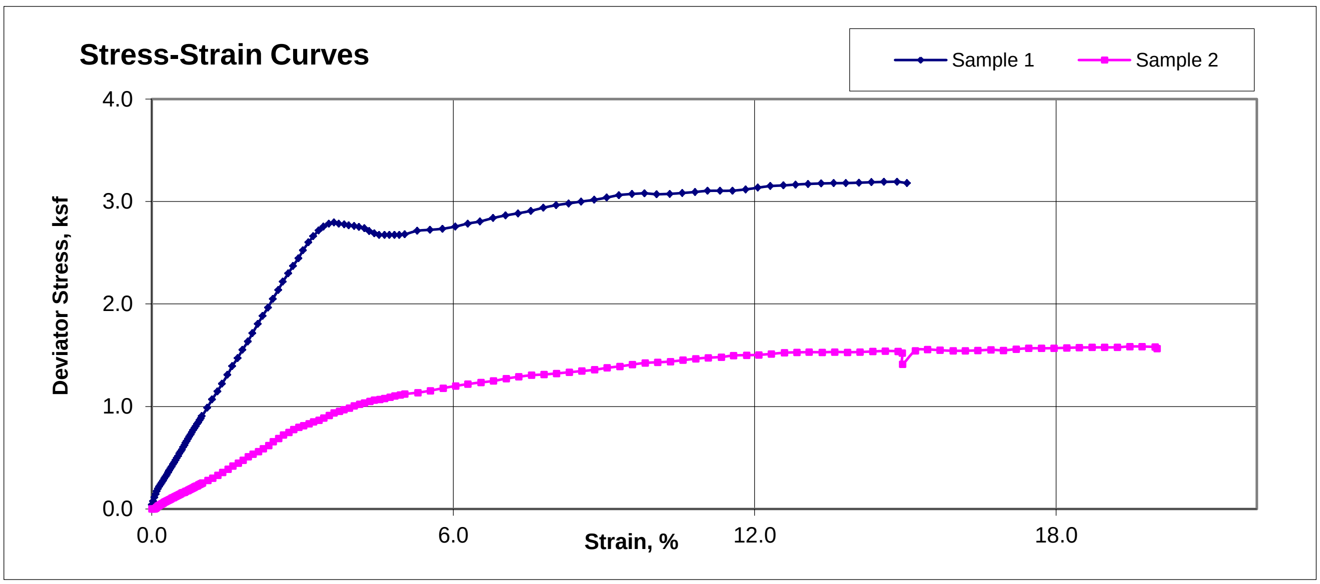

SYMBOL	DESCRIPTION	SOIL TYPE	SAMPLE LOCATION	SAMPLE DEPTH (ft.)	MOISTURE CONTENT w, (%)	DRY DENSITY γ_d, (pcf)	CELL PRESSURE (ksf)	UNDRAINED SHEAR STRENGTH (ksf)
◆	Dark Gray Sandy Lean CLAY	CL	B-2	3.0-4.0	21.3	94.8	0.50	1.60
■	Brown Lean CLAY	CL	B-2	15.0-17.0	24.4	99.7	1.50	0.80

PERFORMED IN ACCORDANCE WITH ASTM D 2850

STRAIN RATE: 1.0%/MIN

FIGURE C-8

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UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST

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APPENDIX D

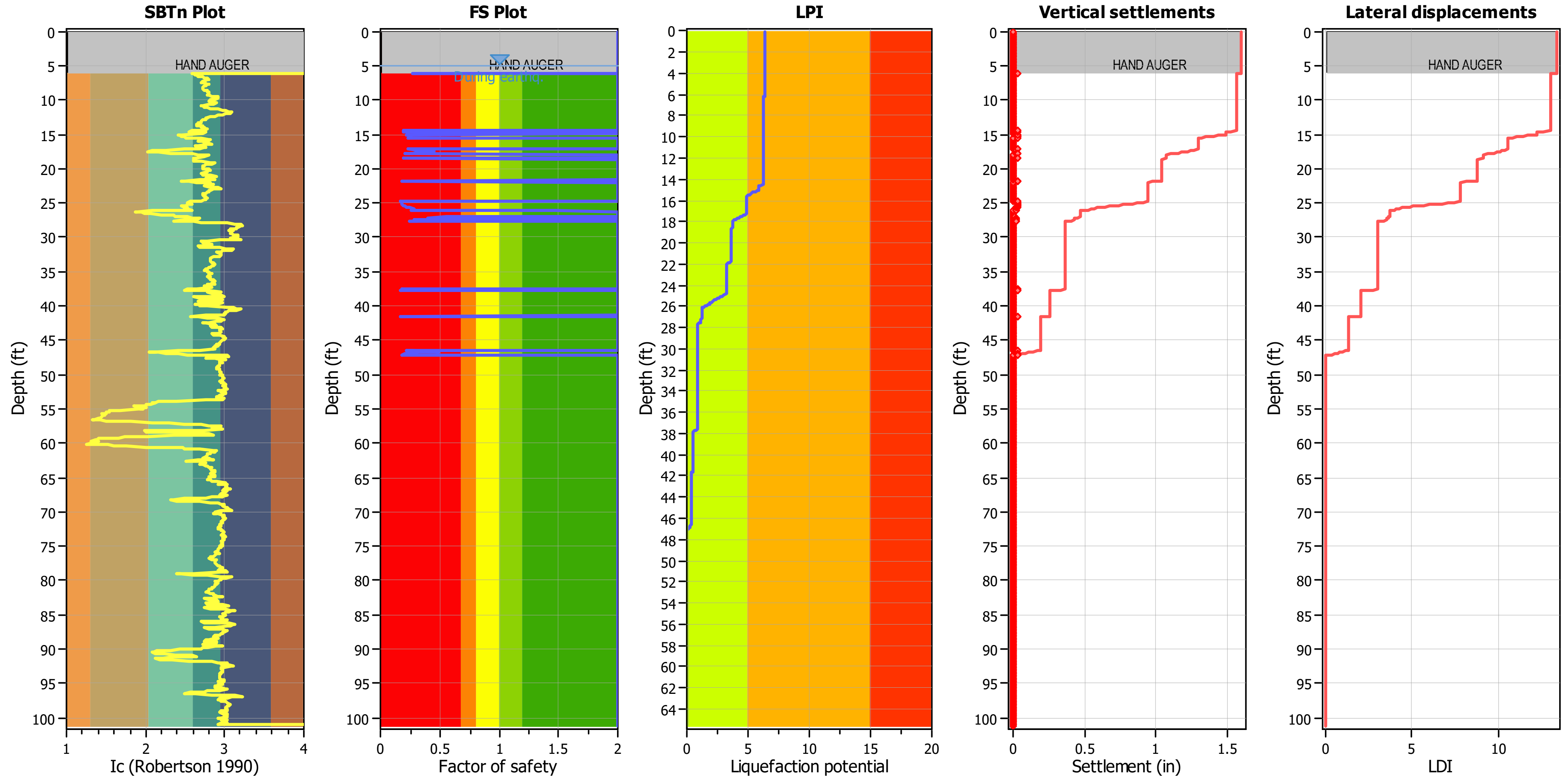
Calculations

Estimated Liquefaction-Induced Vertical and Lateral Soil Displacements
Based on CPT Data

2149 O'Toole Avenue, Suite 30
San Jose, CA 95131
Ph: (408) 435-9000
http://www.ninyoandmoore.com/

Project: 403253010 - East Whisman
Location: Mountain View, California

CPT: CPT-6
Total depth: 101.21 ft



Analysis method:	B&I (2014)	G.W.T. (in-situ):	7.30 ft	Use fill:	No	Clay like behavior applied:	. Yes
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.72	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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