Geotechnical Evaluation

1535-1575 Industrial Avenue

San Jose, California

LBA Realty Fund VI, L.P.

3347 Michelson Drive | Irvine, California 92612

December 8, 2020 | Project No. 403870001

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

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Ms. Kathleen Ledbetter

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PROFESSIONAL GEOLOGIST
DAVID C. SEYMOUR
NO. 1574
CERTIFIED
ENGINEERING
GEOLOGIST
STATE OF CALIFORNIA

December 8, 2020

REGISTERED PROFESSIONAL ENGINEER
RANSOM H. HENNEFER
GE 3152
GEOTECHNICAL
STATE OF CALIFORNIA

December 8, 2020

David C. Seymour, PG, CEG
Principal Engineering Geologist

Ransom Hennefer, PE, GE
Senior Engineer

DCS/RH/gvr

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

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1 INTRODUCTION

In accordance with your authorization, we have performed a geotechnical evaluation and geologic hazards assessment for the proposed warehouse development at 1535-1575 Industrial Avenue in San Jose, California (Figure 1). This report presents our understanding of the proposed project, summarizes the scope of our services, presents the findings and conclusions from our geologic hazards assessment, and our geotechnical recommendations for improvements at the site.

2 SCOPE OF SERVICES

The purpose of this investigation was to evaluate the site soil and groundwater conditions, and geologic setting with respect to how they may impact development of the site and to provide recommendations for the design and construction of the project based on the conditions encountered and the results of our engineering analysis of field and laboratory test data.

Our scope of services included the following:

- Site reconnaissance to observe the general site conditions and to mark the planned locations for our subsurface exploration.
- Coordination with Underground Service Alert (USA) to locate underground utilities in the vicinity of our subsurface exploration.
- Performance of underground utility location surveys using a subcontracted locator service to survey the planned boring and cone penetration test locations.
- Obtain a boring permit from Valley Water (formerly the Santa Clara Valley Water District).
- Subsurface exploration consisting of borings and cone penetration tests (CPTs).
- Performance of one percolation test
- Geotechnical laboratory testing on selected soil samples.
- Compilation and engineering analysis of the field and laboratory data, and the findings from our geologic review.
- Preparation of this report presenting our findings and conclusions regarding the potential geologic hazards and geotechnical conditions at the project site, and our geotechnical recommendations for use in design and construction of the proposed improvements.

3 SITE DESCRIPTION

The 1535-1575 Industrial Avenue property consists of two adjoining parcels that occupy about 3.6 acres along the eastern side of Highway 880 just north of the Highway 880 and Highway 101 interchange (Figure 1). The site is bounded by Highway 880 to the west, Industrial Avenue to the northeast, 1605 Industrial Avenue to the northwest, and other heavy industrial properties to the southeast (Figure 2). The property is zoned as Heavy Industrial per the City of San Jose's general plan and is occupied by a specialty trailer supplier and diesel engine repair facilities. There are five main structures and associated outbuildings on the property, and concrete and asphalt paved parking lots and access roads. Underground storage tanks were present on the property and have reportedly been removed. Details regarding the depth of the tank removal excavations and the backfill placement were not provided for our review. The property is relatively flat and lies at an elevation of about 50 feet above mean sea level.

4 PROJECT DESCRIPTION

The proposed project includes design and construction of an approximately 74,650 square-foot concrete tilt-up building with a loading dock and adjacent parking (Figure 2). We understand that the new structure will be built near the existing grade, and cuts and fills will be only a few feet to construct the dock-high building pad and to contour the site for drainage.

5 FIELD EXPLORATION AND LABORATORY TESTING

Our field exploration for this study included a site reconnaissance and subsurface exploration conducted on October 21 and 23, 2020. The subsurface exploration consisted of two CPT soundings, six hollow-stem auger borings, and one hand auger boring. The approximate locations of the exploration locations are shown on Figure 2.

The two CPT soundings were performed on October 21, 2020 to depths of up to approximately 100 feet below the ground surface using a truck-mounted rig with 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, 2009; Robertson & Campanella, 1986, 1989) based on the cone penetration data and sleeve friction. The CPT sounding logs are presented in Appendix A.

The hollow-stem auger borings and hand auger boring were drilled on October 23, 2020. The hollow stem auger borings were advanced to depths of up to 25 feet below the existing grade with a truck-mounted Mobil B-56 drill rig. A representative of Ninyo & Moore logged the subsurface conditions exposed in the borings and collected relatively undisturbed and bulk soil samples from the borings. Visual classification of the soils was made in general accordance with the Unified Soil Classification System (ASTM D2487). Sampling was conducted using a 2.5-inch inside diameter Modified California sampler with stainless steel liners. The sampler was driven into the underlying soil to a depth of 18 inches with a 140-pound hammer falling 30 inches. The hammer for the hollow stem auger borings was raised using a wireline on a hydraulically operated winch. The number of blows required to drive the sampler the last 12 inches of the 18-inch drive are shown as blows per foot on the boring logs. The blows count values on the boring logs have not been corrected for the effects of overburden pressure, sampler size or hammer efficiency.

The collected samples were transported to our geotechnical laboratory for testing. The borings were backfilled with grout after excavation in accordance with the boring permit. Descriptions of the subsurface materials encountered are presented in the following sections. Detailed logs of the borings are presented in Appendix B.

An 8-inch diameter boring was drilled to a depth of two feet below the ground surface for use in percolation testing. A percolation test was performed on October 23, 2020 at the location shown on Figure 2. The percolation test procedure and test results are presented below in Section 8.610. The test data is included in Appendix E. The test hole was backfilled with soil cuttings after testing.

6 LABORATORY TESTING

Laboratory testing of soil samples recovered from the borings included tests to evaluate in-situ soil moisture content and dry density, grain size distribution, Atterberg limits, consolidation characteristics, expansion index, unconfined compressive strength, and R-value. A soil sample was submitted to CERCO Analytical for a corrosivity evaluation. The results of the in-situ moisture content and dry density tests are presented on the boring logs in Appendix A. The results of the other laboratory tests are presented in Appendix C. The results of the corrosivity tests are presented in Appendix D.

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 site is located within Santa Clara Valley, which is a broad alluvial valley situated at the southern end of San Francisco Bay in the Coast Ranges geomorphic province of California. Santa Clara Valley lies between the Santa Cruz Mountains to the west and the Diablo Range to the east. The Coast Ranges are comprised of northwesterly trending 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

According to regional geologic maps covering the subject property, the site is underlain by Holocene age alluvial soils deposited by nearby Guadalupe and Coyote Creeks (Helley et al., 1994; Knudsen et al., 2000; Wesling and Helley, 1989; and Witter et al., 2006). These deposits typically consist of silt and clay interspersed with layers of sand and gravel. The silt and clay deposits can compress under heavy loads and are also expansive.

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.

7.3.1 Pavement and Fill

The majority of the site is covered by asphalt concrete (AC) pavement or aggregate base (AB). The thickness of the AC varied from about 2 to 6 inches with 0 to 6 inches of underlying AB. Fill material consisting of stiff, lean clay with variable amounts of sand and gravel was encountered beneath the pavement. The fill material was encountered to depths from about 1 to 2 feet below the ground surface.

7.3.2 Alluvium

Alluvium was encountered in the borings and CPT soundings from below the fill to the depths explored. Alluvium encountered in the borings generally consisted of moist to wet, stiff to hard, lean clay and sandy lean clay with occasional thin layers of moist to wet, medium dense, clayey sand. Based on the CPT data, layers of silt and clay extend to a depth of about

44 feet and are underlain by dense layers of sand and gravelly sand. More detailed descriptions are presented on the boring logs in Appendix B, and Soil Behavior Type classifications interpreted from CPT data are presented in Appendix A.

7.4 Groundwater

Groundwater was encountered in the borings at depths of approximately 8 to 18 feet. Groundwater levels were also measured in the CPT soundings at depths ranging from 8 to 10 feet. According to regional records, the historical high groundwater level for the site is less than 10 feet (CGS, 2002a).

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, settlement of compressible soil layers from static loading, unsuitable materials, excavation characteristics, soil corrosivity, expansive soils, and infiltration characteristics. 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, and tsunamis and seiche. 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

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) or the City of San Jose (2011) to delineate regions of potential ground surface rupture adjacent to active faults. The projected trace of the Silver Creek Fault lies about 750 feet southwest of the property; however, this fault is not considered active and does not pose a ground rupture hazard to the site (Wentworth et al., 2010).

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.

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 1010.2. Based on the data obtained using a seismic cone in our CPT sounding, the average shear wave velocity for the upper 100 feet of soil (V_{s100}) was 896 feet per second, corresponding to site class D. 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) 7-16 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 0.695g using the SEAOC/OSHPD seismic design tool (SEAOC and OSHPD, 2019) that yielded a mapped MCE_G peak ground acceleration of 0.632g for the site and a site coefficient (F_{PGA}) of 1.1 for Site Class D.

8.1.4 Liquefaction and Strain Softening

The site is located within a liquefaction hazard zone as established by the California Geological Survey (CGS, 2002b) (Figure 5) and the City of San Jose (2011). Regional studies of liquefaction susceptibility by the U.S. Geological Survey (Knudsen et al, 2000; and Witter et al., 2006) indicate that the site has a moderate susceptibility to liquefaction during a moderate to large magnitude earthquake on a nearby fault.

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.

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 8 feet below the ground surface, and considered a seismic event producing a PGA_M of 0.70g resulting from a Magnitude 6.9 earthquake.

Based on a comparison of borings and CPT soundings that were performed in close proximity to one another, material that was identified as having a behavior type index (Ic) of between 2.4 or 2.6 in the soundings generally correlated with lean clays with a plasticity index of greater than 12 and water content of less than 85 percent of the liquid limit. Based on criteria for liquefaction susceptibility for fine grained soils (Bray and Sancio, 2006; Boulanger and Idriss, 2006), materials with these properties are generally not regarded as susceptible to liquefaction. Accordingly, we used a Ic cutoff of 2.4 or less to evaluate the susceptibility to liquefaction and related hazards. The results of our analysis, presented in Appendix F, indicate that sandy soil and non-plastic silt encountered in the depth interval of approximately 44 to 47 below the ground surface will liquefy under the considered ground motion. The potential impacts 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.

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 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 6.9 earthquake producing a PGA of 0.70g and a design groundwater elevation of 8 feet below the ground surface. The results of our analysis, presented in Appendix F, indicate that the site may undergo dynamic settlement on the order of 1 inch. We estimate differential settlement of approximately ½ inch over a lateral distance of about 30 feet could occur. These are free-field settlement estimates. Where liquefaction occurs at relatively shallow depths building settlement, and in particular at the perimeter of building, may be higher. With liquefaction-induced settlement ton the order of 1 inch and occurring at depths on the order of 40 feet, additional settlement of the building beyond that estimated for free field conditions is not likely.

8.1.6 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 evacuation area as shown on the Tsunami Evacuation Planning Map for the County of Santa Clara (State of California, 2009).

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

Based on maps included in the City of San Jose General Plan (2011), the site is located within an inundation path for Anderson Dam (Figure 6), which is located in the nearby East Bay Hills and flows into nearby Coyote Creek. The dam is currently in the process of a seismic retrofit and is operating at below the level required by the Federal Energy Regulatory Commission (FERC) with plans to begin a full drawdown of the reservoir by late 2020 (SCVWD, 2020). Based on the current operating restrictions and long-term seismic retrofit planned for the dam, we do not consider flooding at the site due to catastrophic dam failure to be a design consideration.

The site is located within a FEMA flood zone, designated as Zone D (Figure 7). Zone D is defined as an area with a risk to flooding due to levee failure. The nearest levee is the Coyote Creek levee, which is located about one-half mile northeast from the property.

8.3 Regional Land Subsidence and Sea Level Rise

Regional land subsidence due to groundwater withdrawal occurred in the Santa Clara Valley during the early and later parts of the last century, and over the last 30 years has been controlled by groundwater management practices led by the Santa Clara Valley Water District. Studies by the U.S. Geological Survey (Poland and Ireland, 1988) indicate that as much as 8 feet of land subsidence occurred as of 1982 in the central portion of San Jose. The estimated amount of land subsidence that occurred beneath the site is about 6 feet, according to maps included in the Poland and Ireland (1988) study. Current groundwater management practices have drastically reduced subsidence rates, which are currently monitored by Federal and local agencies.

In addition, the site is not located within an area that is likely to be impacted by future sea level rise (City of San Jose, 2011).

8.4 Corrosive Soil

Corrosivity analysis was performed by CERCO Analytical, Inc. of Concord, California on a sample of the near-surface soil. As reported by CERCO Analytical, the sample was determined to be “corrosive” based on resistivity test results. CERCO Analytical’ s report included the following recommendation: “All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.” Please refer to the CERCO Analytical report included in Appendix D for more information regarding their test results and brief evaluation.

8.5 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 tests were performed in general accordance with the American Society of Testing and Materials

(ASTM) Standard D 4829 (Expansion Index). The results of our laboratory testing to determine Expansion Index of soils indicate that the expansion index of the near-surface soil is about 38, which is consistent with a low expansion characteristic (ASTM D 4829).

The Atterberg limits data from our laboratory tests show that the plastic index for the near surface soil can be as high as 23 (Appendix C). Based on studies by Holtz and Gibbs (1956) and Chen (1988), the expansion potential of the soil in this location can be classified as medium.

To reduce the potential for differential movement and distress to the proposed improvements due to shrink/swell behavior, foundations should be designed for expansive soils. We anticipate that suitable foundation embedment depths and subgrade preparation can be used to mitigate the expansive soil conditions.

8.6 Infiltration Characteristics

Ninyo & Moore performed percolation testing in the southern portion of the site to evaluate the rate of infiltration of the surficial soil for consideration in design of storm water management systems. The percolation test was conducted using the bore hole method. An 8-inch diameter hole was drilled to a depth of two feet below existing grade. The test was initiated immediately after completion of drilling. No presoaking of the bore hole was performed. Water was introduced in the unlined hole (no casing) with a water depth of 6 inches. The water level was allowed to drop over 1/4-hour increments. Additional water was introduced to restore the water level to a depth of 6 inches after each increment of 1/4-hour with a test duration of 3 hours. The test data are presented in Appendix E and the test result is listed in Table 1, below. The test result indicates that the infiltration rate of the near surface soil on site is very slow. The rate measured likely exceeds a steady state infiltration rate that would occur after an extended period or rainfall with wetting of the ground. Due to the variability of subsurface materials encountered during our exploration, variability in subsurface infiltration should be anticipated. The reliability of test results is discussed below.

Table 1 – Percolation Test Results				
Test (Boring)	Test Depth (ft.)	Subsurface Conditions	Percolation Rate (inch/hour)	Infiltration Rate[1] (inch/hour)
P-1	2	Lean Clay	1.0	0.15

Note:
[1] Infiltration rate is percolation rate adjusted by a reduction factor to exclude percolation through sides of test hole.

There are several methods of conducting percolation or infiltration testing ranging from the simplest, that being the bore hole test, to tests conducted with a double ring infiltrometer. The time of year, ground cover (open or paved), and inclusion or exclusion of presoaking of the test site all influence the test results. The bore hole method is a quick and easy method of measuring water that infiltrates or percolates into the soils both at the base and the sides of the boring at a targeted depth. Lateral infiltration tends to result in an overstated infiltration rate. Infiltration tests conducted in open excavations (test pits) are similar to a bore hole test with some infiltration at the side walls. With the ratio of side wall exposed to water to the pit base in the test pit typically much lower than that of the side wall to base ratio in a bore hole test, slightly better results are obtained in a test pit. The double ring infiltrometer test requires excavation of a test pit and requires considerable more time to conduct. This is considered to be the most accurate test as it was developed to measure vertical infiltration without the influence of lateral infiltration. Based on the configuration of the storm water basin, some lateral infiltration may be appropriate and may be considered by the designer.

Due to the variables in results obtained from the various test methods, the variations in results with variations in performance of each of those tests, and variations in soils, correction factors are commonly applied to measured rates to determine design infiltration rates. We are not aware of a universal correction factor to account for test method. The City of San Francisco PUC recommends correction factors of 0.5 for single ring (simple) and test pit infiltration tests, and correction factor of 0.33 for double ring infiltrometer tests. They recommended more conservative correction factors of 0.10 to 0.15 when using the bore hole test method.

9 CONCLUSIONS

Based on the data collected and the results of our analyses of that data, it is our opinion that development of the site as proposed is feasible from a geotechnical standpoint. Geotechnical considerations include the following:

- Below grade obstructions consisting of foundations and utilities are present on the site. Proper clearing and backfilling of the resulting excavations is important to provide proper support for the new structures.

- Undocumented fill was generally encountered to a depth of about 1 to 2 feet below the ground surface in the borings. Deeper undocumented fill should be anticipated where underground storage tanks were reportedly removed (Figure 2). 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. Recommendations for remedial grading, 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.

- Expansion Index and Atterberg limit testing indicates that the near-surface soil on site has a low to medium 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.

- Groundwater was encountered in the borings at a depth of 8 to 18 feet and measured in the CPT soundings at depths of about 8 to 10 feet. Variation and fluctuation in groundwater levels should be anticipated as discussed in Section 7.4. The presence of shallow groundwater will need to be considered where excavations to remove below grade obstructions or undocumented fill, underground utility trenches and excavations for foundations extend to depths of about 6 feet or more below existing grades.

- 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.

- 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, is left open for an extended period of time, 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.

- 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 analysis, presented in Appendix F, indicate that relatively thin layers of granular soil between depths of approximately 44 to 47 feet will liquefy under the considered ground motion. The potential for bearing capacity reduction due to deeper liquefaction is not a consideration based on the depth of the liquefiable soils.

- The results of our dynamic settlement analysis, presented in Appendix F, indicate that dynamic settlement following the seismic event considered will be relatively minor with approximately 1 inch of total dynamic settlement and a differential of about ½ inch over a horizontal distance of approximately 30 feet.

- We do not regard settlement due to sustained loading as a design consideration for light to moderately loaded structures provided that any loose surficial materials or undocumented fill is mitigated by remedial grading and the improvements are constructed at or below the existing grade.

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

- The site is located in FEMA flood Zone D, which is defined as an area with a risk to flooding due to levee failure. The site is also located within the inundation path for Anderson Dam, however, based on the current operating restrictions and long-term seismic retrofit planned for the dam, we do not consider flooding at the site due to catastrophic dam failure to be a design consideration.

- Our percolation testing at a depth of 2 feet below the existing grade indicates that the infiltration rate of the near-surface soils is very slow.

- Laboratory corrosion testing indicates that the near-surface site soils are considered corrosive. A corrosion engineer should be consulted to provide specific guidance on protective measures to mitigate corrosion.

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

Earthwork should be conducted in accordance with the relevant grading ordinances having jurisdiction and the following recommendations. Ninyo & Moore should observe earthwork operations. Evaluations performed by Ninyo & Moore 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 clearing of the existing above and below grade structures. Existing utilities to be abandoned should be removed or crushed in place. Given the age of the existing developments at the site, the contractor should exercise caution when exposing and removing pipes. ACP pipes were encountered on the adjacent LBA project site. Cleared materials should be disposed of off-site unless a formal on-site recycling process is proposed by the contractor. It may be feasible to recycle existing portland cement and asphalt concrete, as well as existing aggregate base for use as general fill on the site. This would require crushing of the material. Crushed asphalt concrete should not be considered for use as fill below the building.

The site is essentially void of vegetation, except at a few isolated areas at the front of a couple of buildings. Any vegetation and root-laden soils at those locations should be stripped and removed from the site.

Clearing activities should be monitored by a representative of the geotechnical engineer. Excavations and holes that results from the clearing operation should be backfilled with soils placed as engineered fill, which is defined as soil meeting the materials recommendations presented below, that is properly moisture conditioned, placed and compaction as observed and tested by the geotechnical engineer's field representative.

10.1.3 Remedial Grading - Undocumented Fill

Undocumented fill is present, likely over the entire site, with fill depths general ranging from about 1 to 2 feet below the ground surface. Deeper undocumented fill occurs at the sites of previously removed underground storage tanks. There may be other areas of undocumented fill that are not presently known given the presents and age of the existing developments at the site. Undocumented fills exposed through site clearing should be evaluated by Ninyo & Moore to assess the physical properties of the soils and to check for unsuitable soils. Where site grading activities will not remove those fill soils in their entirety through planned cuts, the existing undocumented fill should be removed to within 6 inches of the underlying native soils or in their entirety where found to contain deleterious materials. Undocumented fill that meets the recommendations for general fill presented below may be replaced as engineered fill in accordance with the grading recommendations below. Existing fill soils that do not meet the recommendations for use as fill should be disposed of off-site.

10.1.4 Remedial Grading - Expansive Soils

Laboratory testing indicated that the near-surface soil on site has a low to medium 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 construction fills with low expansion characteristics below building slabs-on-grade, flatwork, and pavement. This may require the removing the existing soil based on finished grade elevations and the need for cut or fill to achieve design grades. The zone of low expansion fill should consist of select, low-expansion import fill conforming with the Material Recommendations section. Alternatively, the on-site soil may be chemically treated by mixing the soil with lime as described in our Chemical Treatment section to reduce the expansion characteristic and create the zone of low-expansion material.

The lateral limits of over-excavations 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 18 inches below building slabs-on-grade, and 12 inches below exterior flatwork or pavement. The limits of the low expansion materials, whether import or chemically stabilized soil, should be included on the grading plans to reduce the potential that these recommendations are overlooked during construction bidding.

10.1.5 Subgrade Preparation

Following completion of site clearing, stripping and the removal of undocumented fills, and prior to placement of fill, the exposed subgrade should be observed by Ninyo & Moore. Materials that are considered unsuitable shall be excavated under the observation of Ninyo & Moore in accordance with the recommendations in this section or supplemental recommendations by Ninyo & Moore. 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.

Exposed subgrade in areas that will receive fill should be scarified to a depth of 12 inches, moisture conditioned to no less than 2 percentage points (2 percent) above the optimum moisture content and should be compacted to no less than 90 percent relative compaction.

Finished subgrade in flatwork and pavement areas should be processed shortly before the planned construction of the concrete flatwork and pavement sections. Finished subgrade should be scarified to a depth of 12 inches, moisture conditioned to no less than 2 percent above the optimum moisture content. Subgrade in concrete flatwork areas that will not be subjected to vehicle loading should be compacted to no less than 90 percent relative compaction. Subgrade in pavement areas, including concrete flatwork areas that will be subjected to vehicle loading, and the warehouse area of the building should be compacted to no less than 95 percent relative compaction. The compacted subgrade should be non-yielding when proof-rolled with a loaded ten-wheel truck, such as a water truck or dump truck, prior to placement of aggregate base or concrete. Subgrade soils should be maintained in a moist and compacted condition until covered with the complete pavement section. 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.

10.1.6 Chemical Treatment

The on-site soil may be chemically treated with high calcium quicklime to reduce the expansion characteristic of the soil as an alternative to importing select fill. The high calcium quicklime should conform with ASTM standard C977. 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 or cement, 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, 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, high calcium quicklime should be added at a dosage that will modify the soil such that the Expansion Index will be less than 50 and the Plasticity Index will be 15 or less. For preliminary cost evaluation a dosage of 4 percent by dry weight of soil should be assumed, with an assumed dry weight of soil of 110 pcf. The actual dosage should be determined through laboratory testing at the time of construction. Testing typically requires about 5 days from receipt of a fresh sample of lime. The contractor should provide a sample of the lime that will be used in construction to Ninyo & Moore about 2 weeks prior to the planned start of lime treatment.

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 minimum mellowing period after the initial mixing. The lime-soil mixture should be compacted within 3 days after initial mixing to achieve 90 percent of the reference density as evaluated by ASTM D1557 on a dry density basis.

The grading contractor should provide assistance to Ninyo & Moore with grade checking to confirm surface elevations and depth of mixing as the lime treatment operation proceeds.

10.1.7 Material Recommendations

Materials used during earthwork operations should comply with the requirements listed in Table 2. Materials should be evaluated by the Civil 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 2 – Recommended Material Requirements

Material and Use	Source	Requirements[1,2]
General Fill[1]	On-site borrow	No additional requirements unless planned for use within upper 18 inches of building pad or 12 inches below concrete flatwork
	Import[2, 3]	As per Select (Low Expansion) Fill
Low Expansion (Select) Fill: - top 18 inches of finished pad below building slabs and top 12 inches of finished subgrade below concrete flatwork	Import	Well-graded with 100 percent passing the 1-inch sieve, 35 percent or more passing No. 4 sieve and either: Plasticity Index of 15 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
Pipe/Conduit Bedding and Pipe Zone Material -material below pipe invert to 12 inches above pipe	Import	75-100 %passing #4 sieve 0-70% passing #50 sieve 0-30% passing #100 sieve 0-10% passing # 200 sieve
Trench Backfill - On-Site - above bedding material	Import or on-site borrow	As per General Fill and Low Expansion Fill excluding rock/lumps retained on 4-inch sieve or 2-inch sieve in top 12 inches
Trench Backfill - Off-Site - above bedding material	Import or on-site borrow	In accordance with City of San Jose requirements
Aggregate Base	Import	Class 2, 3/4-inch max. Should NOT contain recycled asphalt concrete if used below floor slabs. CSS[4] Section 26-1.02
Capillary Break Gravel - below SOG floors other than warehouse space	Import	Open-graded, clean, crushed gravel; nominal size 3/4-inch x 3/8-inch or #4
Controlled Low Strength Material (CLSM) - Utility Trench backfill	Import	CSS[3] (2018) Section 19-3.02G Compressive strength 50-200 psi
Controlled Low Strength Material (CLSM) -below footings	Import	CSS (2018) Section 19-3.02G Compressive Strength Minimum 100 psi

Notes:

In general, fill should be free of rocks or lumps in excess of 4-inches diameter, trash, debris, roots, vegetation or other deleterious material.

In general, import fill should be tested or documented to be non-corrosive as defined by the Corrosion Guidelines (Caltrans, 2018b) and free from hazardous materials in concentrations above levels of concern.

CSS is California Standard Specifications

10.1.8 Fill Placement and Compaction

Fill and backfill should be compacted in horizontal lifts in conformance with the recommendations presented in Table 3. 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 for large grading equipment and 4 inches in loose thickness for manually operated equipment such as jacking jack or vibratory plate compactors.

Table 3 – Compaction Recommendations

Material	Location	Min. Relative Compaction[1]	Moisture Content[2]
Rough Subgrade	In areas to receive fill	90 percent	Min. 2 percent above opt.
General Fill	All locations during rough grading	90 percent	Min. 2 percent above opt.
Select Fill	18-inch fill section at building pad	95 percent	Min. 2 percent above opt.
	12-inch fill section at concrete flatwork	90 percent	Min. 2 percent above opt.
Lime-Treated Soils	All locations	95 percent	Min. 3 percent above opt.
Finished Subgrade	In pavement areas and concrete flatwork areas that will be subject to vehicle loading - top 12 inches	95 percent	Min. 2 percent above opt.
Bedding and Pipe Zone Fill	Material below invert to 12 inches above pipe or conduit	90 percent	Near Optimum
Trench Backfill	From Bedding to 12 inches below finished subgrade	90 percent	Min. 2 percent above opt.
	To finished subgrade in concrete flatwork areas	90 percent	Min. 2 percent above opt.
	Top 12 inches of subgrade in pavement areas and concrete flatwork areas subject to vehicle loading	95 percent	Min. 2 percent above opt.
Aggregate Base	Pavement section or below hardscape	95 percent	Near Optimum

Notes:

1 Expressed as percent relative compaction or ratio of field density to reference density on a dry density basis for soil and aggregate. 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 4 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 4 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 4 – 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.

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, over-excavating to a suitable depth and replacing the wet material with suitable fill, compacting a layer of crushed rock fill into the subgrade, using geogrid to stabilize additional fill, and in the case of utility trenches, constructed a layer of geotextile wrapped crushed gravel. Specific recommendations for excavation stabilization will be influenced by the nature of the excavation and the conditions encountered during construction. Ninyo & Moore should be consulted at the time of grading to assist in determining appropriate methods of subgrade stabilization if needed.

10.1.10 Construction Dewatering

Groundwater was encountered during our subsurface exploration in the borings and CPTs at depths of about 8 to 18 feet below the ground surface. This is consistent with regional maps which indicate that the historic high groundwater level in the site vicinity is less than approximately 10 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.

10.1.11 Utility Trenches

Excavations should conform to applicable State and Federal industrial safety requirements. The responsibility for the safety of open excavations should be borne by the contractor. The walls of trenches extending into the clayey soils will likely stand in vertical cuts in the upper four to five feet with appropriate shoring, provided proper moisture content in the soils is maintained and that the trench walls are not subjected to vibration or surcharge loads above the excavation. Where weaker soils or granular soils are encountered in the upper 4 to 5 feet of the site or trenches will extend deeper than 5 feet, trench sidewalls should be shored or should be sloped. Trenches constructed for the installation of underground utilities should be stabilized in accordance with our recommendations in our Excavation Stabilization section.

Trench Location Relative to Structures

To maintain the desired support for the foundations, underground utilities that are not entering structures and that will be oriented roughly parallel to the structures should be located away from the building perimeters. Trenches should be located such that the bottom of the trench closest to the foundation is located above a projected plane extending down and away from the base of the foundation at an inclination of 1.5 Horizontal: 1 Vertical (1.5H:1V).

Backfill

Utility trenches should be backfilled with materials that conform to our recommendations in our Material Recommendations section. Trench backfill, bedding, and pipe zone fill should be compacted in accordance with our Fill Placement and Compaction section 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. Backfill materials should be placed in level lifts about 4 to 12 inches in loose thickness, moisture conditioned and mechanically compacted. Lift thickness will be a function of the type of compaction equipment in use. Thinner lifts (4- to 6-inch lifts) will be required for manually operated equipment, such as wackers or vibratory plates, and thicker lifts possible where a sheepsfoot wheel is used on the stick of an excavator. Densification of trench backfill by flooding or jetting should not be permitted.

Seepage Cut-Off

To reduce potential for moisture intrusion into a building envelope, we recommend plugging utility trenches at locations where the trench excavations cross under a 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. 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.

Where soils are too wet to achieve a well-compacted, stable subgrade as a result of the impacts of rainfall, chemical stabilization using high calcium lime may be needed. Lime and cement treatment can also be used as a preventative treatment to stabilize the site allow construction to continue through period of rainfall with significantly less impact on the soils.

10.2 Seismic Design Criteria

Table 5 presents the Risk-Targeted, Maximum Considered Earthquake (MCE_R) spectral response accelerations consistent with the 2019 California Building Code and corresponding site-adjusted and design level spectral response accelerations based on the USGS seismic design maps (SEAOC/OSHPD, 2020). The values provided in the table may be used for structures with a fundamental period of 0.75 seconds or less presuming that the seismic response coefficient is calculated from equation 12.8-2 of ASCE Standard 7-16 in accordance with Exception 2 in Section 11.4.8 of ASCE Standard 7-16.

Table 5 – 2019 California Building Code Seismic Design Criteria

Seismic Design Parameter Evaluated for 37.3706° North Latitude, 121.9001° West Longitude	Section 11.4 ASCE 7-16
Site Class	D[1]
Site Coefficient, F_a	1.0
Site Coefficient, F_v	-
Mapped MCE_R Spectral Response Acceleration at 0.2-second period, S_S	1.500g
Mapped MCE_R Spectral Response Acceleration at 1.0-second period, S_1	0.600g
Site-Adjusted M_{CER} Spectral Acceleration at 0.2-second period, S_{MS}	1.500g
Site-Adjusted M_{CER} Spectral Acceleration at 1.0-second period, S_{M1}	-
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.000g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	-
Seismic Design Category for Risk Category I, II, or III	-

Note:
[1] Based on the data obtained using a seismic cone in our CPT sounding, the average shear wave velocity for the upper 100 feet of soil (Vs100) was 896 feet per second, corresponding to site class D.

10.3 Foundation Recommendations

The planned building may be supported on spread footings. 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. Footings bearing on alluvium or new engineered fill with subgrade prepared in accordance with the recommendations in Section 10.1.5 may be designed using the criteria listed in Table 6. Ninyo & Moore should observe the footing excavations to evaluate bearing materials and subgrade condition before the exposed subgrade is covered.

Table 6 – Recommended Bearing Design Parameters for Footings					
Footing	Sustained Loads	Footing Width[1]	Bearing Depth[2]	Allowable Bearing Capacity[3]	Static Settlement[4]
Wall Footing	5 kips/foot or less	1½ feet or more	3 feet or more	2,000 psf	1-inch total ½ inch differential over 30 feet
Column Footing	40 kips or less	2 feet or more	3 feet or more	2,500 psf	¾ inch total ½ inch differential over 30 feet
	90 kips	6 feet or more	2 feet or more	2,500 psf	1 inch total ½ inch differential over 30 feet
	140 kips	8 feet or more	2 feet or more	2,500 psf	1¼ inch total ⅔ inch differential over 30 feet

Notes:

1 Assumes square footing shape.

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 load of 65 kips, footing width should be 4 feet or more, bearing depth 3 feet or more, allowable bearing pressure should be 2,500 psf, and static settlement should be anticipated to be 1 inch total.

Structures supported on footings consistent with these recommendations should be designed for the total and differential settlements listed in Table 6 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 7. The designer may interpolate between the values in the table for intermediate footing widths.

Table 7 – Footing Modulus of Subgrade Reaction					
Footing[1]	Footing Width				
	1½ feet	2 feet	3 feet	4 feet	5 feet
Wall Footing	75 pci	53 pci	33 pci	24 pci	18 pci
Column Footing[2]	---	103 pci	63 pci	45 pci	35 pci

Notes:
[1] Assumes bearing depth of 36 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 1.5H:1V (horizontal to vertical) angle. 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.

10.4 Dock-High Retaining Walls

Retaining walls that are incapable of deflection or walls that are fully constrained against deflection may be designed for an equivalent fluid at-rest pressure of 65 pounds per square foot per foot per depth. This design pressure does not include surcharge pressures associated with loads placed behind and above the walls, hydrostatic pressure or pressure associated with expansive soils. The surcharge effect from loads placed above the walls should be included in the wall design. The surcharge load for restrained walls should be based on one-half of the applied load placed above and within four feet of the back of the wall. The surcharge load should be distributed over the full height of the wall. The stated lateral earth pressures do not include the effects of hydrostatic pressures generated by infiltrating surface water that may accumulate behind the retaining walls. With the walls designed to retain fill that will be fully enclosed and within the interior of the building, it is very unlikely that that hydrostatic pressures will develop. However, if this is a concern, the Structural Engineer should consider a hydrostatic component, or a drain should be constructed behind the wall.

Retaining walls should be backfilled with granular or low expansion materials for a horizontal distance of not less than 2 feet. Backfill should be compacted to between 90 and 95 percent relative compaction up to the top one foot with the top one foot compacted to 95 percent. Over-compaction should be avoided because increased compactive effort can result in lateral pressures significantly higher than those recommended above. During grading and backfilling operations adjacent to any walls, heavy equipment should not be allowed to operate within a lateral distance of 4 feet from the wall to avoid developing excessive lateral pressures. Only hand operated equipment (rammers, vibratory plates, or trench roller compactors) should be used to compact the backfill soils within this zone.

10.5 Concrete Floor Slabs-On-Grade & Exterior Flatwork

10.5.1 Warehouse Concrete Floor Slabs

Warehouse concrete floor slabs should be supported by at least 6 inches of Class 2 aggregate base over 18 inches of low expansion potential fill soils (PI of 15 or less) or lime-treated clay soils. The Class 2 aggregate base and top 18 inches of building pad subgrade soils should be compacted to a minimum of 95 percent relative compaction. Where the Class 2 aggregate base is supported on silty to clayey sand or sandy clay, the slabs may be designed using a modulus of subgrade reaction, k, of 75 pounds per cubic inch (pci). The actual soils placed for the top two feet of the dock-high fill should be verified at the time of grading to allow for verification of the modulus of subgrade reaction as recommended. An allowable bearing value of 2,500 pounds per square foot (psf) due to dead plus live loads may be used in thickness design for pallet rack column loads.

With relatively shallow groundwater at the site, placement of a vapor retarder below the warehouse floor slab should be considered. See Section 10.5.3 for vapor retarding system recommendations. To further reduce the potential for soil moisture to migrate through the slabs-on-grade as a vapor, and to reduce concrete shrinkage, consideration should be given to the use of a low water/cement ratio concrete mix (w/c 0.45 or less). Consolidation of the concrete will also reduce the degree of vapor that can pass through the slab.

Warehouse concrete floor slabs should be designed for both static and rolling loads. The designers should consider punching shear at point loads where pallet racks will be installed and flexural stresses associated with moving loads. Forklift traffic results in highly concentrated loads and stress reversals in the slab as the forklift moves across the slabs. Warehouse floor slab failure frequently occurs at joints. Joint design should incorporate keyed and reinforced joints, doweled joints or thickened concrete sections at panel edges. Joint spacing should be developed with consideration of concrete shrinkage and slab reinforcement (which may consist of fiber-reinforced concrete).

10.5.2 Office Space Concrete Floor Slabs

Interior concrete slab-on-grade floors should be underlain by a layer of crushed, open-graded gravel to act as a capillary break. A vapor retarder should be provided between the concrete slab and the capillary break. Concrete should be placed directly on the vapor retarder; we recommend that sand not be placed over the vapor retarder. See Section 10.5.3 for vapor retarding system recommendations.

10.5.3 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. Placement of a vapor retarder should also be considered below the warehouse floor slab. For areas outside of the warehouse space 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 plastic membrane should conform to the requirements in the latest version of ASTM Standard E 1745 for a Class A membrane. The capillary break should be constructed of clean, compacted, open-graded crushed rock or angular gravel of 3/4-inch x 3/8-inch nominal size.

The vapor retarder should be placed over the capillary break gravel section or Class 2 aggregate base if placed in the warehouse space. Sand should not be placed over the vapor retarder. 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.

10.5.4 Slab Crack Control Joints

Joints consistent with ACI guidelines (ACI, 2016) should be constructed to reduce the potential for random cracking of the slabs. Slab control joints should be spaced in accordance with the recommendations presented in the ACI Manual of Concrete Practice. In the event that control or contraction joints are to be constructed by saw cutting of the slabs, saw cuts should be made by soff-cut sawing within 4 to 12 hours after the initial hardening (not curing) of the concrete, as required by atmospheric conditions. The contractor should be responsible for monitoring of the concrete during initial set or hardening for determination of the optimal timing for cutting of the slabs.

10.5.5 Non-Structural Exterior Concrete Flatwork

With the exception of slabs subject to vehicular loads, it is our opinion that, from a geotechnical engineering standpoint, exterior concrete slabs-on-grade, such as sidewalks, can be placed directly on the prepared subgrade consisting of low expansion soils. The use of aggregate base as support for concrete flatwork should be avoided except in traffic areas where it may be required as part of a structural section. Where subgrade soils consist of clay soils having a moderate expansion potential, it is important that these soils be properly moisture conditioned and compacted, and that the moisture content is maintained until the concrete has been constructed. The moisture content of the subgrade soils should be checked several days prior to the placement of concrete. Where moderately expansive soils are present and the soil moisture content is less than 5 percent above optimum, the subgrade should be presoaked to at least 5 percent over optimum moisture content prior to placing concrete. Even with proper site preparation there will be some effects of soil moisture change on concrete flatwork.

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.

10.6 Pavements

Recommendations for asphalt concrete pavement (flexible pavement) and portland cement concrete pavement (rigid pavement) are presented in the following sections. The design R-value used for evaluate the pavement sections was selected based on R-value testing performed on a sample collected during our subsurface exploration. The pavement subgrade should be observed by Ninyo & Moore 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 10.1.5.

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.

10.6.1 Asphalt Pavement

Ninyo & Moore conducted an analysis to evaluate appropriate asphalt pavement structural sections following the methodology presented in the Highway Design Manual (Caltrans, 2019). 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 flexible pavement sections are presented in Table 8.

Table 8 – Asphalt Concrete Pavement Sections			
Traffic Index	R-value	Alternative 1	Alternative 2
4.5	8	2½ inches AC 8½ inches AB	3 inches AC 5 inches AB 12 inches L/CTS
5.0	8	3 inches AC 9½ inches AB	3 inches AC 6 inches AB 12 inches L/CTS
6.0	8	3½ inches AC 12 inches AB	3½ inches AC 8 inches AB 12 inches L/CTS
6.5	8	4 inches AC 13 inches AB	4 inches AC 8 inches AB 12 inches L/CTS
7.0	8	4 inches AC 15 inches AB	4 inches AC 10 inches AB 12 inches L/CTS
7.5	8	4½ inches AC 16 inches AB	4½ inches AC 10 inches AB 12 inches L/CTS
8.0	8	5 inches AC 17 inches AB	5 inches AC 10 inches AB 15 inches L/CTS
10.5	8	6½ inches AC 23½ inches AB	6½ inches AC 14 inches AB 15 inches L/CTS

Notes:
[1] AC is Type A, Dense-Graded 3/4-inch Hot Mix Asphalt complying with Caltrans Standard Specification 39.
[2] AB is Class II Aggregate Base complying with Caltrans Standard Specification 26-1.02.
[3] L/CTS is chemically stabilized soil using lime or lime +cement to stabilize the subgrade soils.

Aggregate base for pavement should be placed in lifts of no more than 8 inches in loose thickness and compacted per Section 10.1.9. Asphalt concrete should be placed and compacted in accordance with Caltrans Standard Specification and Construction Manual; asphalt concrete should be compacted to between 92 and 96 percent of the theoretical maximum specific gravity and density (Rice gravity - ASTM D 2041) of the material. 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.

10.6.2 Concrete Pavement

Concrete pavement sections based on methodologies developed by the Portland Cement Associate (PCA) are presented in Table 9 for a 20-year design period with appropriate periodic maintenance.

Table 9 – Portland Cement Concrete Pavement Sections

Loading Condition[1]	Equivalent Traffic Index	Subgrade Modulus[2]	Concrete Pavement Section 5,000 psi	Concrete Pavement Section 4,000 psi	Concrete Pavement Section 3,500 psi
7,300 Annual Vehicles including 48 annual 40-kip trucks	6	50 pci	7 inches PCC[3] 12 inches AB[4]	8 inches PCC[3] 12 inches AB[4]	8½ inches PCC[3] 12 inches AB[4]
18,200 Annual Vehicles including 365 annual 40-kip trucks	7	50 pci	8 inches PCC[3] 12 inches AB[4]	9 inches PCC[3] 12 inches AB[4]	9½ inches PCC[3] 12 inches AB[4]
36,500 Annual Vehicles including 3,650 annual 40-kip trucks	8	50 pci	8½ inches PCC[3] 12 inches AB[4]	9½ inches PCC[3] 12 inches AB[4]	10 inches PCC[3] 12 inches AB[4]

Notes:

1 Assumes 4-kip passenger vehicles and box truck with 8-kip single axle and 32-kip dual 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 (2018a).
4 AB is Class II Aggregate Base complying with Caltrans Standard Specification Section 26 (2018a).

The recommended section presumes that the concrete will have compressive strength of 5,000, 4,000, and 3,500 psi at 28 days. Aggregate base for pavement should be placed in lifts of no more than 8 inches in loose thickness and compacted per recommendations in this report.

Appropriate jointing of concrete pavement can reduce the potential for crack development between joints. Joints should be laid out in a consistent square pattern. Contraction, construction, and isolation joints should be detailed and constructed in accordance with the guidelines of the ACI Committee 302 (ACI, 2015). Contraction joints formed by premolded inserts, grooving plastic concrete, or saw-cutting at initial hardening, should extend to a depth equivalent to 25 percent of the slab thickness and 1 inch or more for thin slabs. Contraction joints should be reinforced with smooth dowels placed across the joint at mid-slab height. Construction joints subject to traffic loading should be reinforced with smooth dowels as for contraction joints. Construction joints within the middle third of the typical joint spacing pattern should be reinforced with tie bars. Recommendations for contraction joint spacing, dowel dimensions, dowel spacing, tie bar dimensions, and tie bar spacing are provided in Table 10. Isolation joints should consist of full-depth premolded joint filler placed where the pavement abuts structures or other fixed objects. At isolation joints where the edge of the pavement will be subjected to traffic loading, the thickness of the slab should be increased by 20 percent at the edge of the pavement with a 40:1 taper (horizontal to vertical) to the nominal slab thickness.

Table 10 – Concrete Pavement Joints and Reinforcement

Slab Thickness	Contraction Joint Spacing	Dowels	Tie bars at 10 feet to free edge	Tie bars at 25 feet to free edge	Distributed Steel
7 inches	14 feet or less	7/8 x 14 at 12 inches	½ x 24 at 30 inches	½ x 24 at 20 inches	#5 at 18 inches both ways
8 inches	16 feet or less	1 x 14 at 12 inches	½ x 24 at 30 inches	½ x 24 at 17 inches	#5 at 18 inches both ways
9 inches	18 feet or less	1 1/8 x 14 at 12 inches	½ x 24 at 30 inches	½ x 24 at 14 inches	#5 at 18 inches both ways
10 inches	20 feet or less	1 1/4 x 14 at 12 inches	½ x 24 at 30 inches	½ x 24 at 12 inches	#5 at 18 inches both ways

Note:
[1] Dowels and Tie bars specified in nominal diameter x length at spacing along joint in inches. The designer may interpolate between the values provide for an intermediate distance to the free edge of pavement.

The recommended sections presume that the pavement is laterally restrained by curbs, structures, driveway aprons, or other pavements. The thickness of the recommended concrete sections should be increased by 1 inch for pavements that are unrestrained and joints that are parallel and adjacent to pavement edges should be reinforced with deformed steel tie bars, instead of dowels, placed across the joint at mid-slab height as recommended in Table 10.

Distributed reinforcing steel consisting of deformed steel bars may be placed to reduce the potential for differential slab movement, should cracking occur between joints. The spacing between contraction joints may be increased where distributed reinforcing steel is utilized. Pavements reinforced with distributed steel consistent with the recommendations in Table 10, may be designed for a contraction joint spacing of up to 70 feet. 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 concrete cover over the steel. The distributed steel should be terminated about 6 inches from contraction or isolation joints.

The pavement surface and subgrade should be sloped to provide positive drainage toward suitable drainage devices. 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 saw cut 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.

10.6.3 Subgrade – Chemically Stabilized

Chemical stabilization consists of the addition of cement and/or lime to the subgrade soils to increase the strength of the soil, thereby allowing for its use as a partial replacement for the required aggregate base section. Lime is the preferred stabilization agent for cohesive soils and works best on moderate to high plasticity clays. Cement is the preferred agent for granular soils and low plasticity clay. We recommend that lime be used as the initial/primary stabilizing agent to stabilize the moderately expansive subgrade soils at the subject site. Cement may need to be added as a secondary stabilizing agent to achieve the desired unconfined compressive strength of the stabilized and compacted soil.

The required dosage of the stabilizing agent(s) will need to be determined through laboratory testing. The treated soils should have an unconfined compressive strength of no less than 400 pounds per square inch. For preliminary budgeting a lime dosage of 5 percent high calcium quick lime based on a soil dry unit weight of 110 pounds per cubic foot should be assumed. Preliminary budgeting should also consider lime dosage of 3.5 percent high calcium quick lime on the initial spread and mix, followed by the addition of 3.5 percent cement based on a soil dry unit weight of 110 pounds per cubic foot. The in-place densities of the untreated soils should be measured for use in determining actual cement spread rates.

Prior to the start of chemical stabilization, the mass grading of the site to achieve design grades should be completed. The area to be treated should be brought to design grades with a tolerance of +0.00 to -0.05 feet. The area should be proof-rolled to confirm that there are no soft areas that extend into the soils below the section to be treated. If soft soils are present below the soils to be cement-treated the required relative compaction of the cement-treated spoils will not be achievable, reducing the performance and life expectancy of the completed pavement. Excessively wet soils within the depth of treatment should also be removed or additional lime may be required. Grades should be checked and confirmed prior to the initiation of lime treatment. Where significant variations from design grade are not addressed prior to lime treatment, the final section may be less than required or filling with additional aggregate base may be needed.

The contractor should exercise caution in controlling the moisture content of the soil-cement blend such that there is sufficient moisture for adequate cement hydration, without there being excessive moisture preventing proper compaction. The limits of the lime or lime+cement should extend not less than 2 feet beyond the edge of pavements.

Most specialty contractors providing soil stabilization services are equipped to treat and compact soils to a depth of 12 inches, with some equipped to treat and compact a full 18-inch section in one lift. Where the depth of treated soil will exceed 12 inches and where the contractor is not equipped to treat and compact a section greater than 12 inches, the treatment of the subgrade will require two lifts. The lower lift should be treated and compacted, followed by placement of the required soils to achieve pad grade. The second lift of soil may be treated after it has been placed to establish the graded grades or may be treated on adjacent mixing tables.

Lime-treated soils should be allowed to mellow for 36 hours before final mixing or the spreading and mixing of cement. The gradation of the mixed material should be 95 to 100 percent passing the 1” sieve, with 60 to 100 percent passing the No. 4 sieve.

Compaction of all fills, and stabilization and compaction of lime- or lime+cement treated soils should extend no less than 2 feet back of all curbs or beyond pavement edges. Lime -treated soils should be compacted within 48 hours after completion of the final remix. Cement treated soils should be compacted on the same day the cement is spread. Stabilized soils should be compacted to not less than 95 percent relative compaction.

The completed section of lime+cement-treated soils should be protected from drying during the initial curing period of 7 days. The area can be covered with plastic or may be sealed with an asphalt emulsion such as SS-1. A spray rate of not less than 0.10 gal/square yard (0% added water) is recommended. The lime+cement-treated soils should be allowed to cure for not less than 7 days before placement of aggregate base.

10.7 Concrete and Soil Corrosivity Considerations

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).

10.8 Bioretention Areas

Bioretention swales and basins located in close proximity to roadways, parking lots and exterior concrete flatwork can cause settlement of these structures as well as cracking associated with lateral extension of these structures with lateral movement of the supporting soils. Loss of lateral support for adjacent pavement sections may occur if the bioretention area is placed too close to the pavement. Where located immediately adjacent to structural pavements, the load carrying capacity of the pavement may be reduced with complete pavement failure a possibility in the area along the edge of pavement. Where lateral support is reduced or eliminated, curb and curb and gutter sections can move laterally away from the pavement and will likely settle.

Excavations for bioretention facilities should be located away from structural pavements. Where a vertical excavation will be made to allow for placement of drainage and bio media backfill, the excavation should be located five feet or more away from pavements and concrete flatwork. Where the excavation will be made with sloped sidewalls at an inclination of 1 horizontal to 1 vertical (1H:1V), the top of the excavation should be located a minimum of 3 feet away from pavement or concrete flatwork. If these conditions cannot be met due to site constraints, bioretention areas should be constructed with structural side walls (retaining walls) capable of withstanding the loads from the adjacent improvements.

Bioretention areas located within five feet of pavements should be lined with impermeable liners. A perforated drain pipe should be provided within the swale or basin when a liner is installed or where the site soils have a low permeability rate and infiltration capacity (i.e. the clay soils at the subject site). The perforated pipe should lead to a solid-wall pipe to convey accumulated water to a suitable point of discharge.

10.9 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. Where possible, drainage gradients should be 2 percent or more a distance of 5 feet or more from the structure for impervious surfaces. This may be reduced to the maximum allowable of 1½ percent under ADA regulations where necessary. A drainage gradient of 5 percent or more a distance of at least 10 feet from the structure is recommended for pervious surfaces. Roof drainage should be collected and diverted to suitable discharge areas away from structures

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.

10.10 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.

10.11 Construction Observation and Testing

The recommendations provided in this report are based on subsurface conditions encountered in relatively widely spaced exploratory borings. During construction, Ninyo & Moore or his representative in the field should be allowed to check the exposed subsurface conditions. During construction, Ninyo & Moore or the 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, aggregate base, and asphalt concrete.
- 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.

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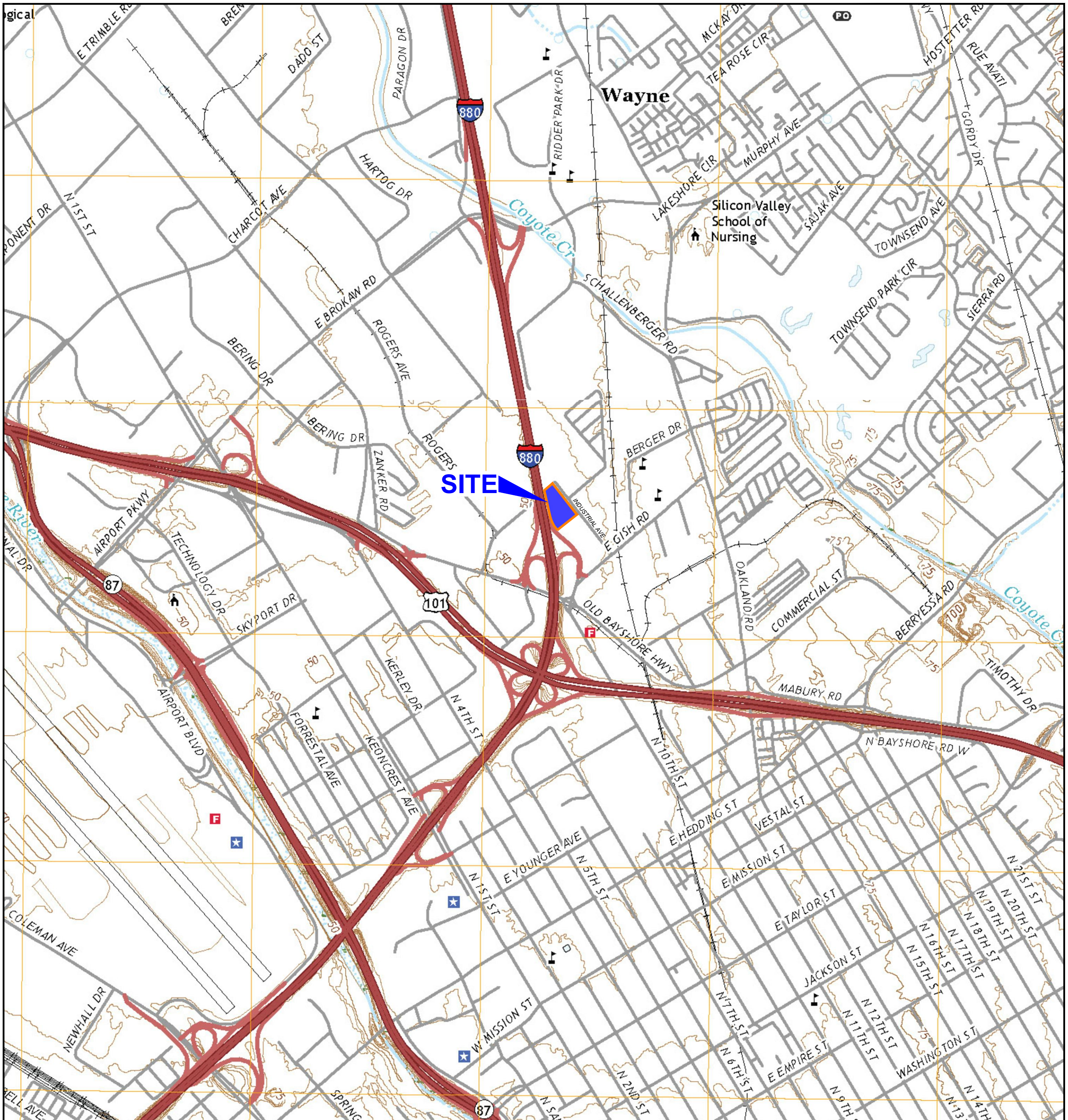
FIGURES



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

N

SCALE (FEET)

0 2,000 4,000

403870001.dwg 12/02/2020 AEK

FIGURE 1

SITE LOCATION

1535-1575 INDUSTRIAL AVENUE
SAN JOSE, CALIFORNIA

403780001 I 12/20

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

403870001.dwg 12/02/2020 AEK

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE | REFERENCE: GOOGLE EARTH, 2020

FIGURE 2

EXPLORATION LOCATIONS

1535-1575 INDUSTRIAL AVENUE
SAN JOSE, CALIFORNIA
403780001 | 12/20

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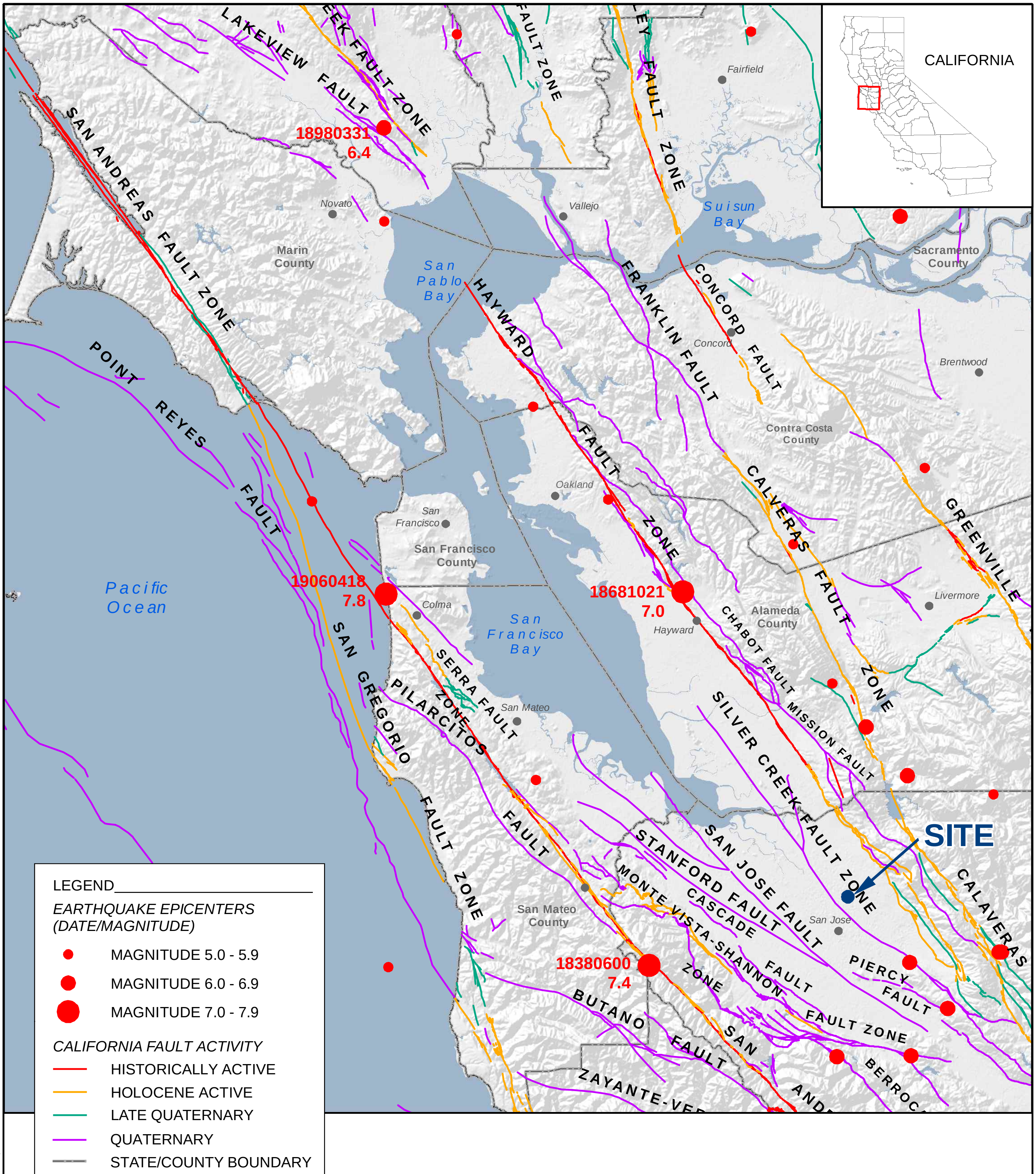
Geotechnical & Environmental Sciences Consultants



NOTE: DIRECTIONS, DIMENSIONS, AND LOCATIONS ARE APPROXIMATE

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

N

SCALE (MILES)
0 10 20

403870001-Fig3-FL.pdf 12/08/2020 AEK

FIGURE 3

FAULT LOCATIONS AND EARTHQUAKE EPICENTERS

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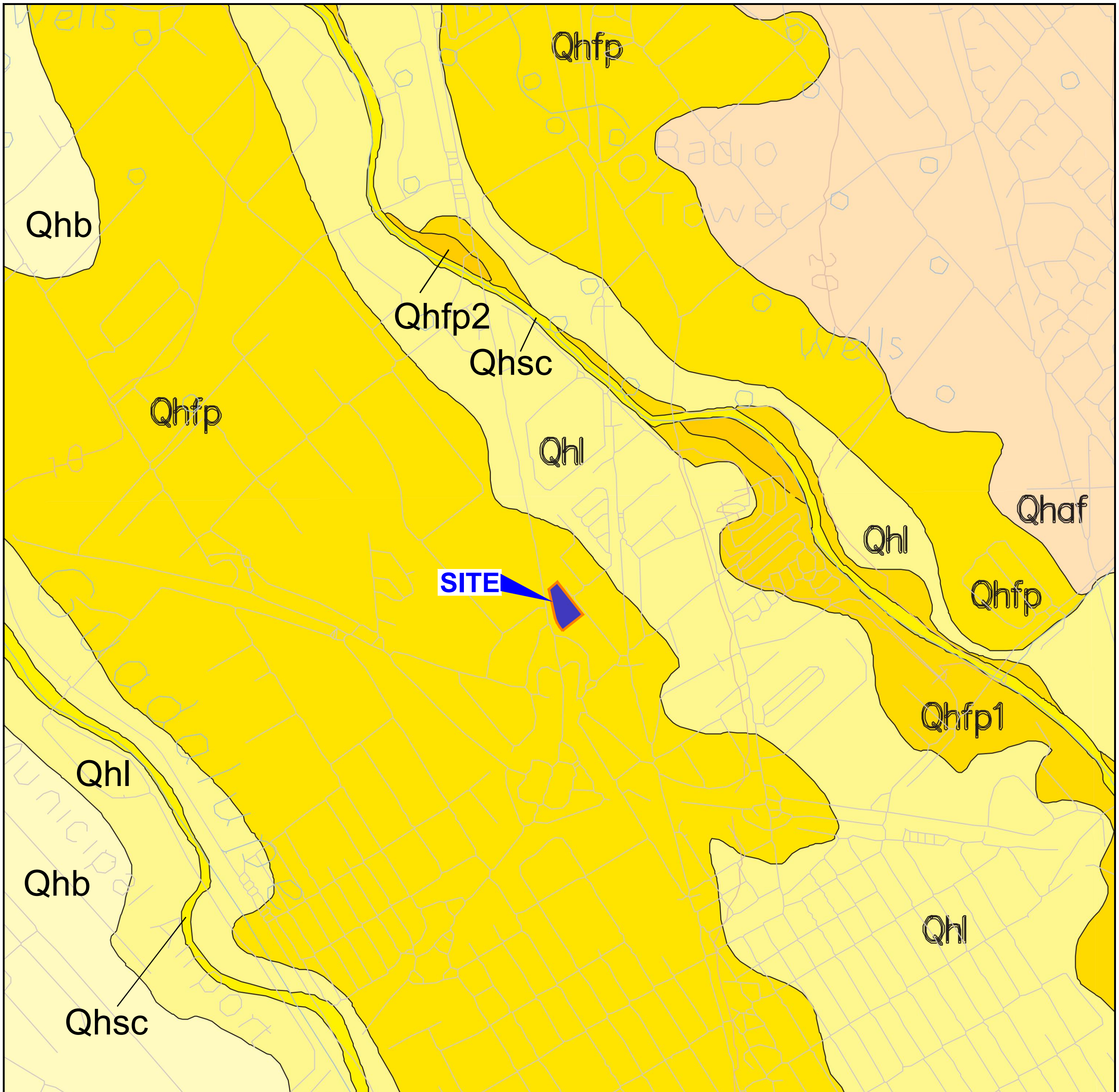
1535-1575 INDUSTRIAL AVENUE
SAN JOSE, CALIFORNIA
403870001 | 12/20



LEGEND

Qhsc — STREAM CHANNEL DEPOSITS (HOLOCENE)	Qhfp2 — SECOND ALLUVIAL TERRACE DEPOSITS (HOLOCENE)	Qhaf — ALLUVIAL FAN DEPOSITS (HOLOCENE)
Qhl — NATURAL LEVEE DEPOSITS (HOLOCENE)	Qhfp — FLOODPLAIN DEPOSITS (HOLOCENE)	
Qhfp1 — FIRST ALLUVIAL TERRACE DEPOSITS (HOLOCENE)	Qhb — FLOODBASIN DEPOSITS (HOLOCENE)	

FAULT
GEOLOGIC CONTACT

N

SCALE (FEET)
0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE I REFERENCE: USGS, OFR 94-231,1994

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FIGURE 4

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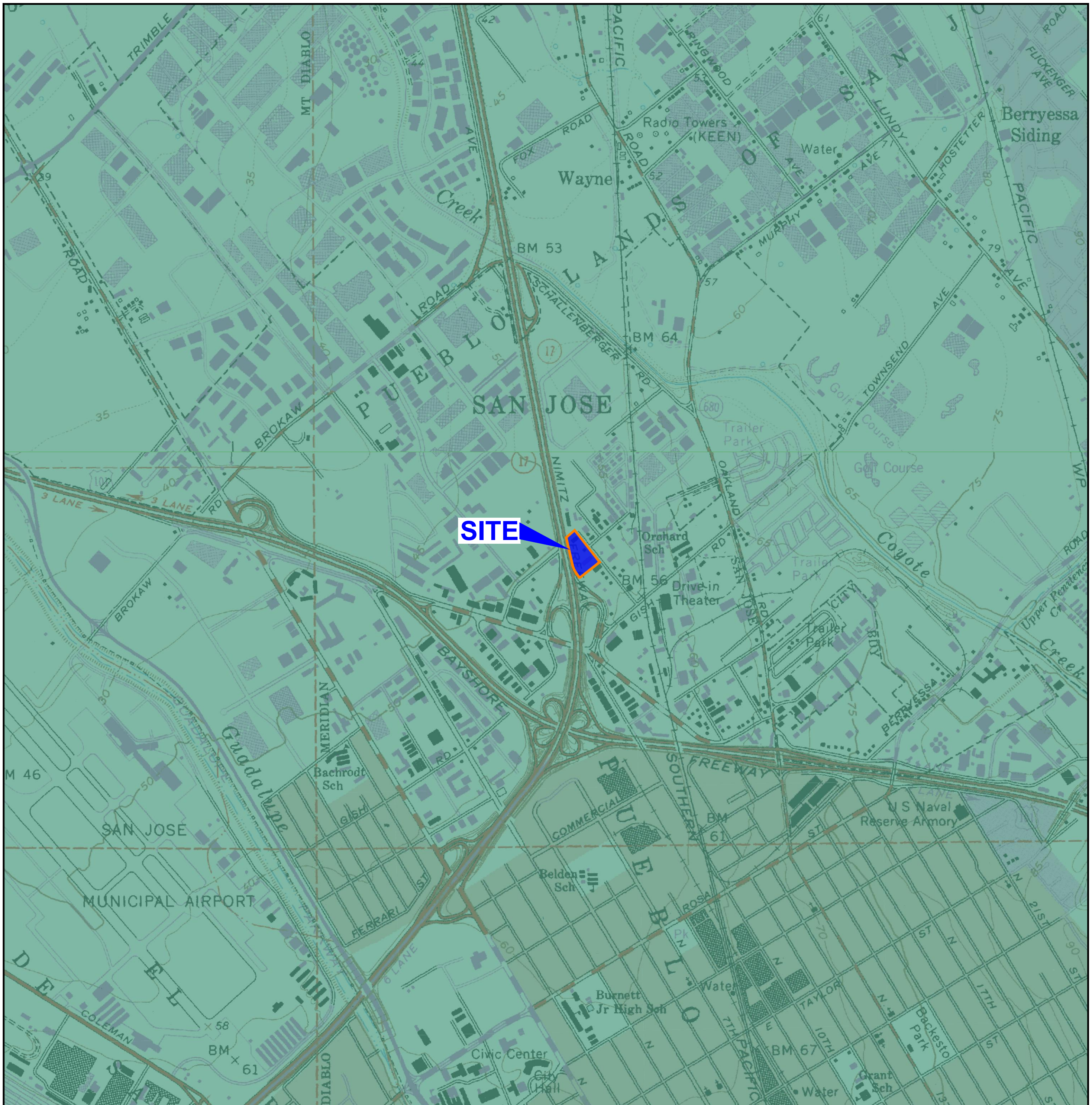
Geotechnical & Environmental Sciences Consultants

REGIONAL GEOLOGY

1535-1575 INDUSTRIAL AVENUE
SAN JOSE, CALIFORNIA

403780001 I 12/20



LEGEND

LIQUEFACTION ZONES:
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; 2002, 2004

N

SCALE (FEET)
0 2,000 4,000

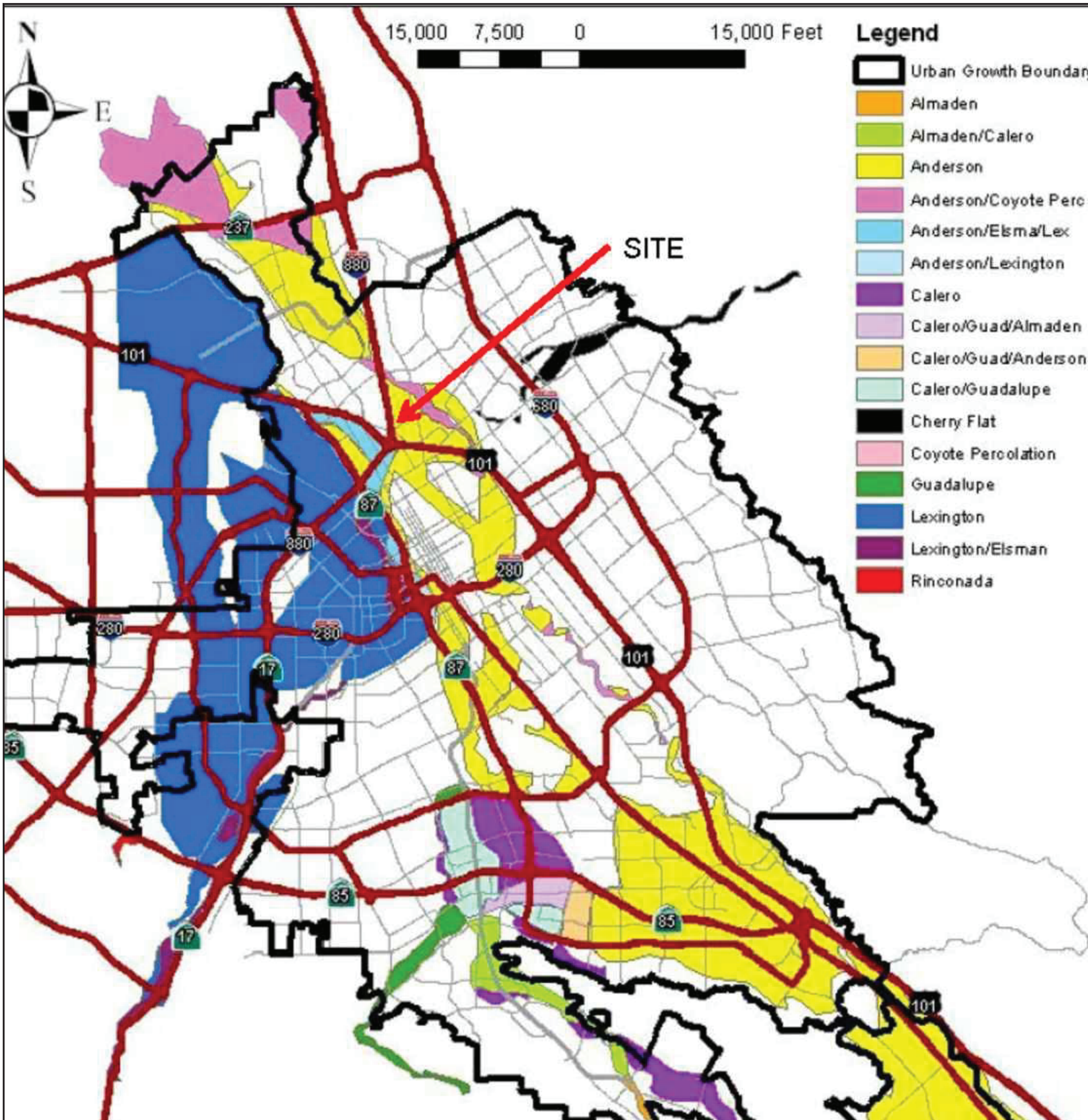
403870001.dwg 12/02/2020 AEK

FIGURE 5

SEISMIC HAZARD ZONES

1535-1575 INDUSTRIAL AVENUE
SAN JOSE, CALIFORNIA
403780001 I 12/20

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE
REFERENCE: ABAG DAM FAILURE, APPENDIX F OF SAN JOSE GENERAL PLAN, 2011

N

SCALE (MILES)
0 2.5 5

403870001.dwg 10/19/2020 AEK

FIGURE 6

DAM INUNDATION

1535-1575 INDUSTRIAL AVENUE
SAN JOSE, CALIFORNIA
403780001 | 12/20

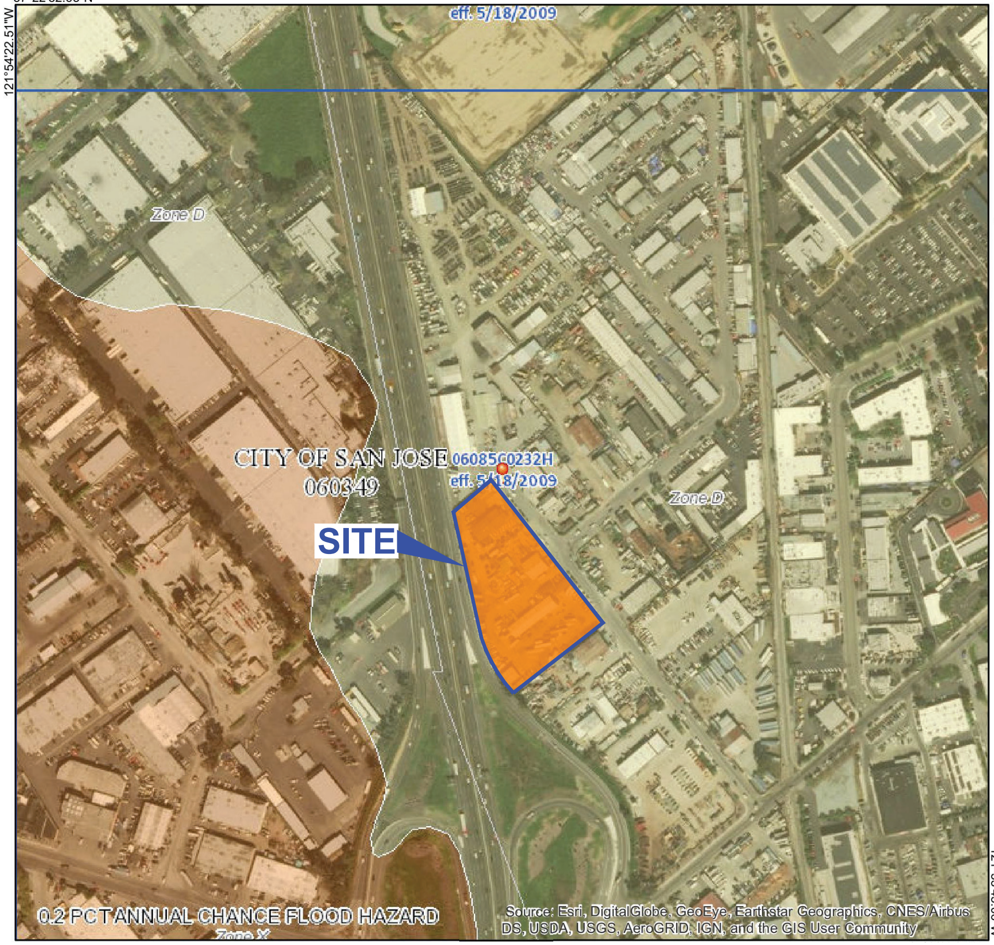
Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

National Flood Hazard Layer FIRMette

FEMA

37°22'32.63"N

121°54'22.51"W



121°53'45.06"W

Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS
- Without Base Flood Elevation (BFE) *Zone A, V, A99*
- With BFE or Depth
- Regulatory Floodway *Zone AE, AO, AH, VE, AR*

OTHER AREAS OF FLOOD HAZARD
- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
- Area with Flood Risk due to Levee *Zone D*

OTHER AREAS
- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Effective LOMRs
- Area of Undetermined Flood Hazard *Zone D*

GENERAL STRUCTURES
- Channel, Culvert, or Storm Sewer
- Levee, Dike, or Floodwall

OTHER FEATURES
- 20.2 / 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
- Coastal Transect
- 513 Base Flood Elevation Line (BFE)
- Limit of Study
- Jurisdiction Boundary
- Coastal Transect Baseline
- Profile Baseline
- Hydrographic Feature

MAP PANELS
- Digital Data Available
- No Digital Data Available
- Unmapped

N

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The base map shown complies with FEMA's base map accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 3/20/2018 at 4:22:32 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

This map image is void if the one or more of the following map elements do not appear: base map imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

N

SCALE (FEET)

0 600 1,200

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE | REFERENCE: FEMA, ESRI; 2018

403870001.dwg 10/19/2020 AEK

FIGURE 7

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

FEMA FLOOD ZONE

1535-1575 INDUSTRIAL AVENUE
SAN JOSE, CALIFORNIA
403780001 | 12/20

APPENDIX A

Cone Penetration Testing

APPENDIX A

CONE PENETRATION TESTING

Field Procedure for Cone Penetration Testing

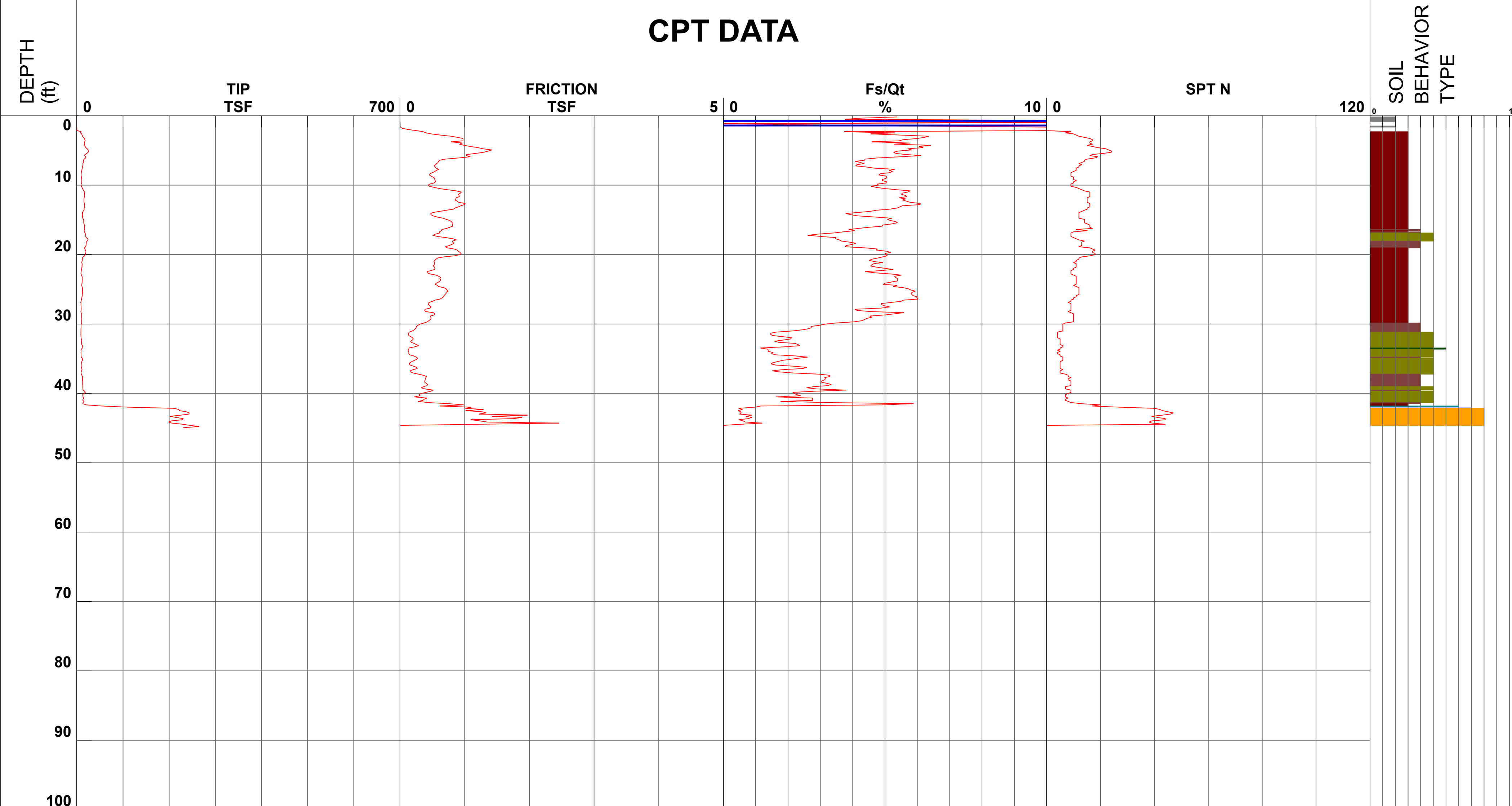
A penetrometer with a conical tip having an apex angle of 60 degrees and a cone base area of 15 square centimeters was hydraulically pushed through the soil using the reaction mass of a 20-ton rig at a constant rate of about 20 millimeters per second in accordance with ASTM D 5778. The penetrometer was 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 2 inches. 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 sleeve resistance (F_s), friction ratio (Fs/Qt) 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.

Ninyo & Moore

Middle Earth Geo Testing Inc.

Project	LBA 1535 Industrial Avenue	Operator	JM-ZG	Filename	SDF(176).cpt
Job Number	403870001	Cone Number	DDG1530	GPS	
Hole Number	CPT-01	Date and Time	10/21/2020 3:30:58 PM	Maximum Depth	44.95 ft
EST GW Depth During Test	8.00 ft				

Net Area Ratio .8



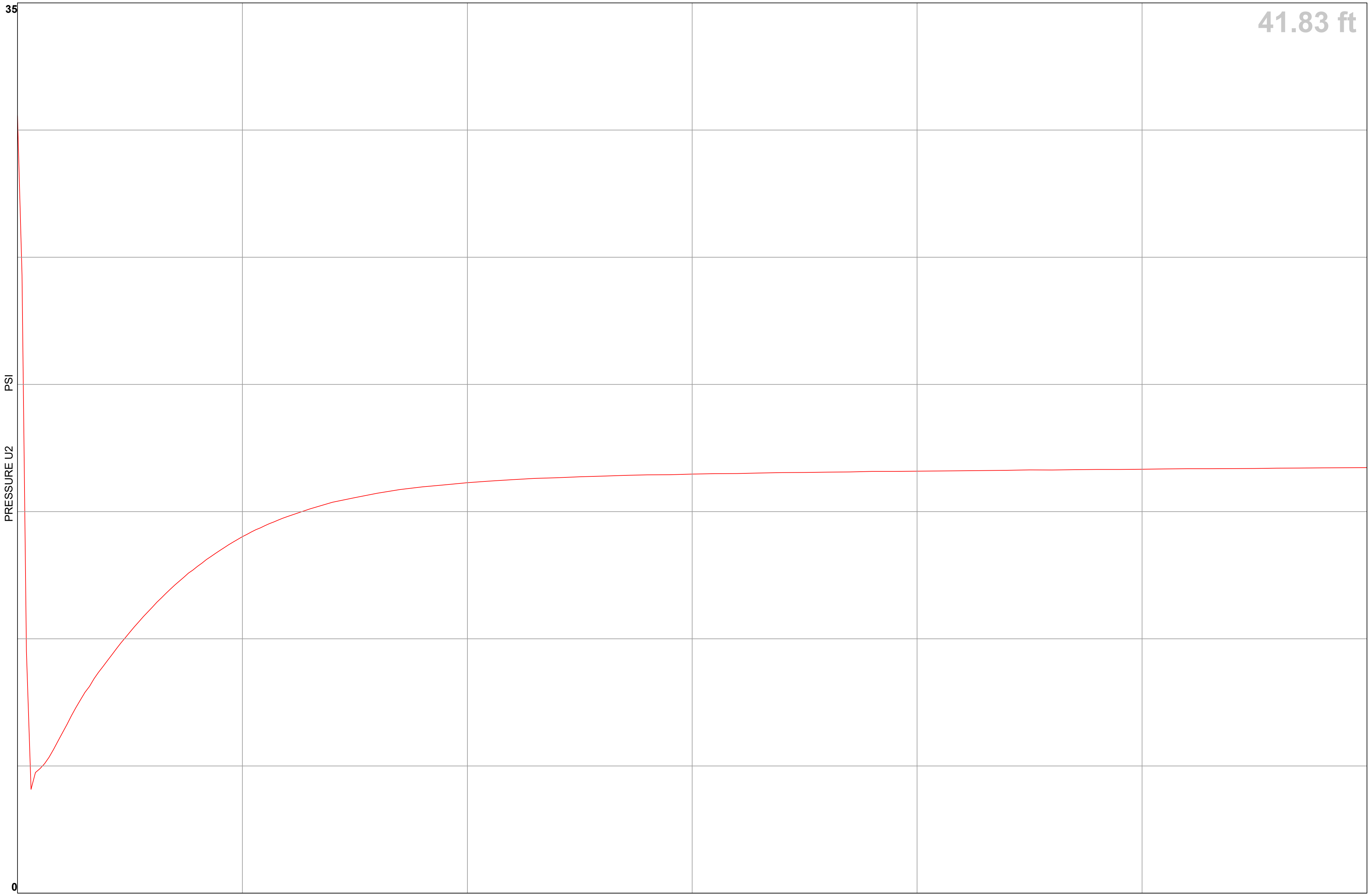
- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey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

Cone Size 15cm squared

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

Ninyo & Moore

Location	LBA 1535 Industrial Avenue	Operator	JM-ZG		
Job Number	403870001	Cone Number	DDG1530	GPS	
Hole Number	CPT-01	Date and Time	10/21/2020 3:30:58 PM		
Equilized Pressure	16.5	EST GW Depth During Test	3.5		

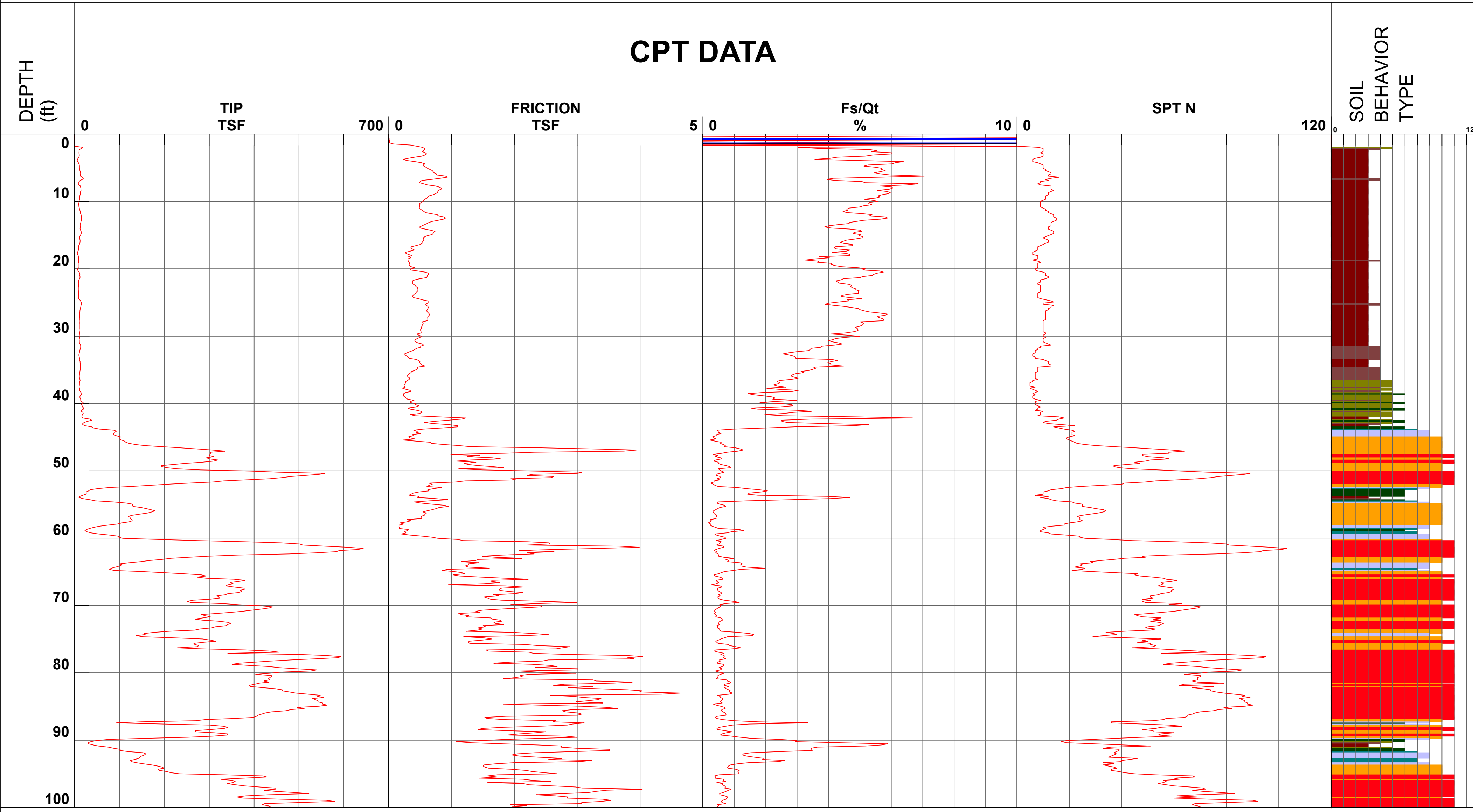


Ninyo & Moore

Project	LBA 1535 Industrial Avenue	Operator	JM-ZG	Filename	SDF(175).cpt
Job Number	403870001	Cone Number	DDG1530	GPS	
Hole Number	CPT-02	Date and Time	10/21/2020 12:42:26 PM	Maximum Depth	100.72 ft
EST GW Depth During Test		10.00 ft			

Net Area Ratio .8

CPT DATA



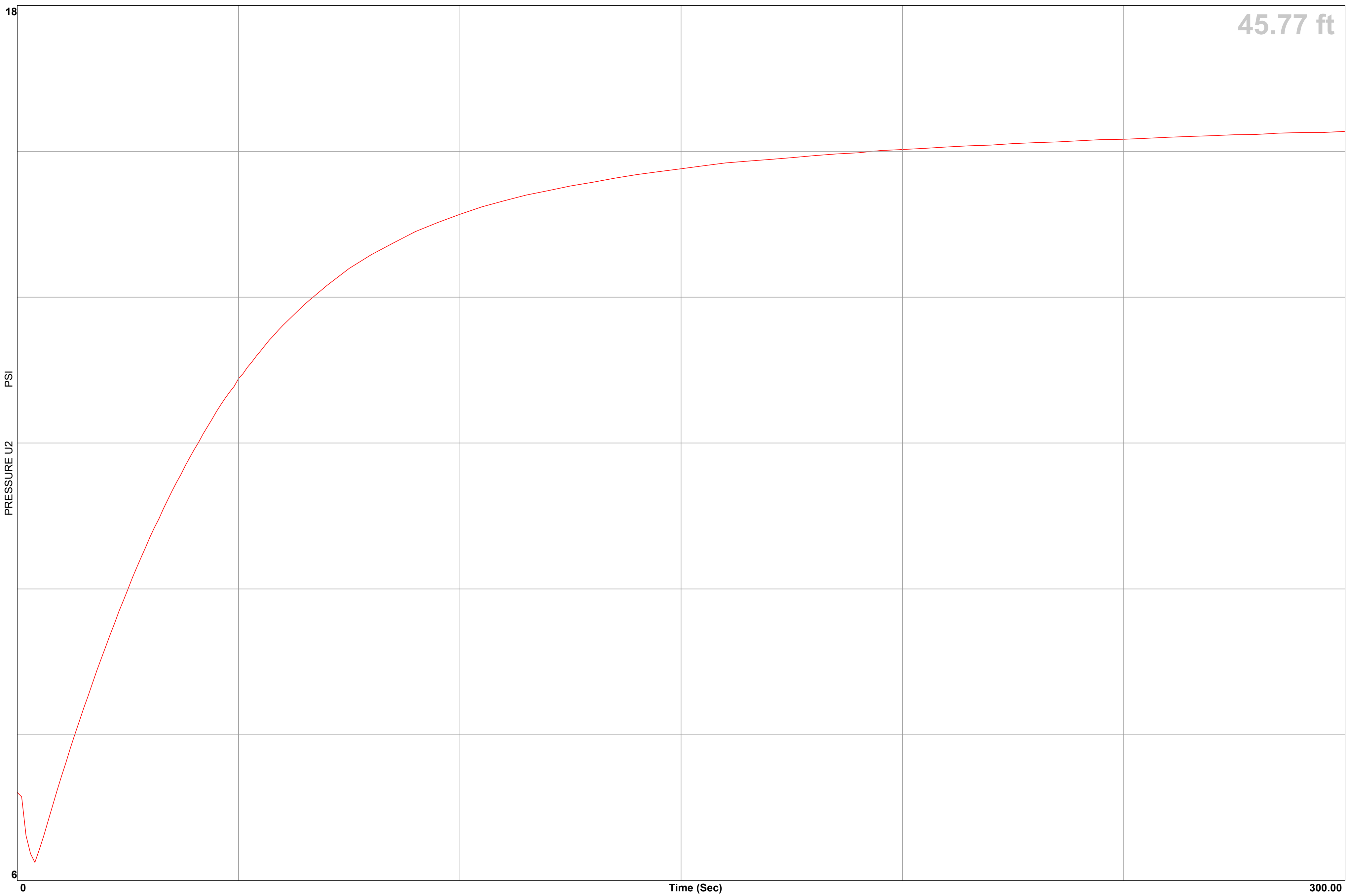
1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

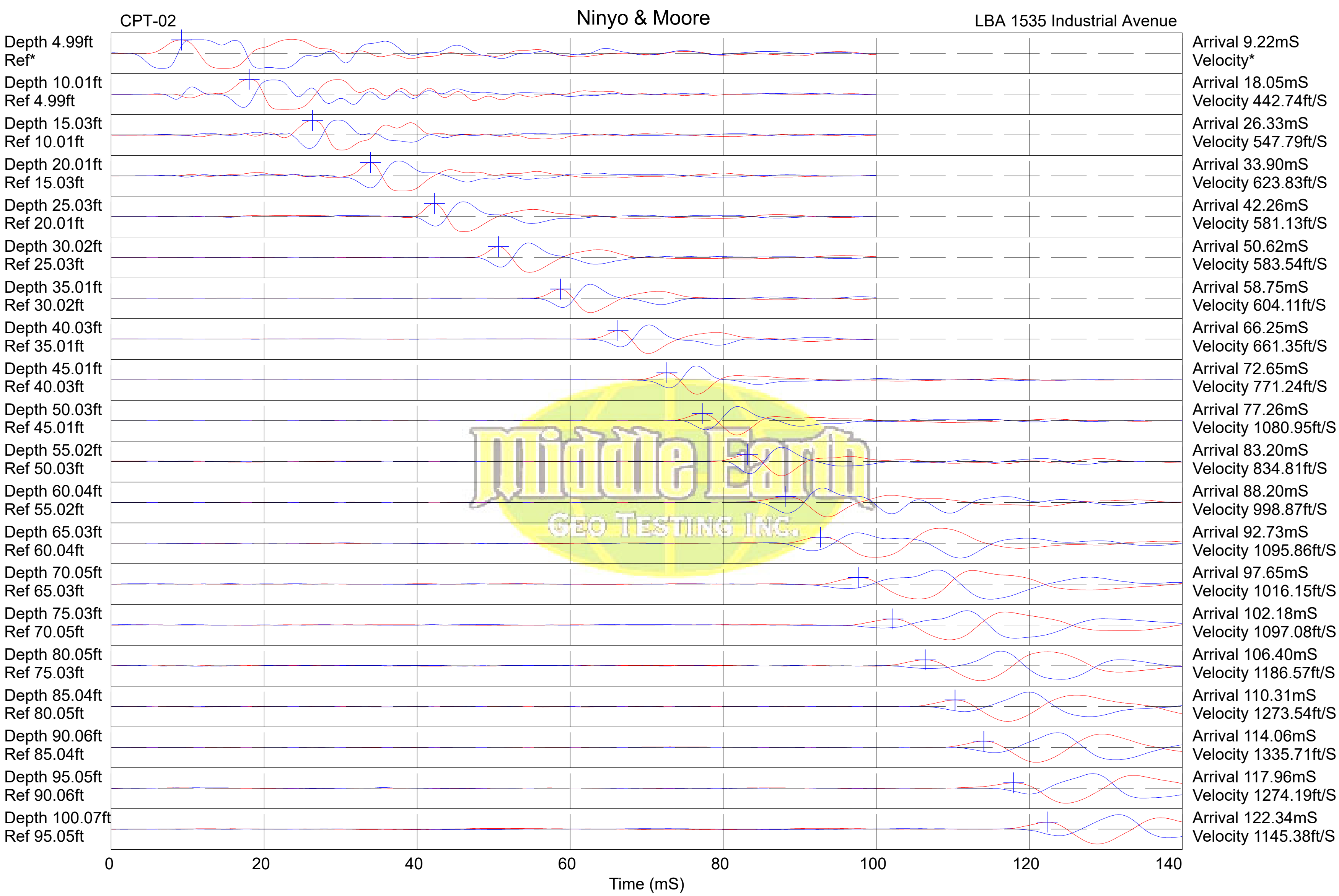
Cone Size 15cm squared

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

Ninyo & Moore

Location	LBA 1535 Industrial Avenue	Operator	JM-ZG		
Job Number	403870001	Cone Number	DDG1530	GPS	
Hole Number	CPT-02	Date and Time	10/21/2020 12:42:26 PM		
Equilized Pressure	16.2	EST GW Depth During Test	8.2		





Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

APPENDIX B

Boring Logs

APPENDIX B

BORING LOGS

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

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

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

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

BORING LOG EXPLANATION SHEET

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

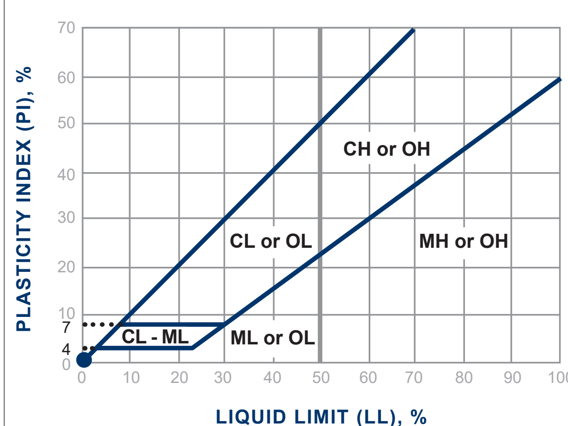
Soil Classification Chart Per ASTM D 2488

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

Consistency - Fine-Grained Soil

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

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

USCS METHOD OF SOIL CLASSIFICATION

DATE DRILLED	10/23/2020
BORING NO.	B-1
GROUND ELEVATION	42 ± (MSL)
SHEET	1 OF 1
METHOD OF DRILLING	8" HSA, B-56 Truck Mounted (Exploration Geo.) Top 2' HA
DRIVE WEIGHT	140 LBS (wireline)
DROP	30 INCHES
SAMPLED BY	JJC
LOGGED BY	JJC
REVIEWED BY	RH

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								AGGREGATE BASE: Approximately 6 inches thick.
							CL	Brown, moist, stiff, lean CLAY; trace gravel.
			14					Dark brown.
			17	21.3	100.4			Brown; increase in sand content.
			16	30.2	90.7			
10							CH	Grayish brown, moist, stiff, fat CLAY.
			16					
							CL	Brown, wet, very stiff, sandy lean CLAY. Grayish brown.
			25					
			28	21.3	103.3			Orange staining, decrease sand content.
20								
			15	35.4	83.8			Gray.
30								
40								

Total Depth = 25.0 feet

Backfilled the borehole with drill cuttings and patched with concrete shortly after drilling on 10/23/2020.

Notes:
Groundwater was encountered at 13 feet in the borehole 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 (Google, 2020).

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 10/23/2020 BORING NO. B-2 GROUND ELEVATION 42 ± (MSL) SHEET 1 OF 1 METHOD OF DRILLING 8" HSA, B-56 Truck Mounted (Exploration Geo.) Top 2' HA DRIVE WEIGHT 140 LBS (wireline) DROP 30 INCHES SAMPLED BY JJC LOGGED BY JJC REVIEWED BY RH **DESCRIPTION/INTERPRETATION**
0								ASPHALT CONCRETE: Approximately 6-inches thick.
								AGGREGATE BASE: Approximately 6-inches thick.
							CL	Brown, moist, stiff, lean CLAY; trace gravel. Dark brown
			16					
			14					
							CH	Grayish brown, moist, stiff, fat CLAY.
10			12					
			15	29.2	91.6			
							CL	Brown, moist, stiff, lean CLAY.
				▽				Wet; increase in sand content.
20			14					
			32	32.0	88.3			Grayish brown, very stiff.
								Total Depth = 25.0 feet Backfilled the borehole with drill cuttings and patched with concrete shortly after drilling on 10/23/2020. Notes: Groundwater was encountered at 18 feet in the borehole 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 (Google, 2020).
30								
40								

DATE DRILLED 10/23/2020 BORING NO. B-3

GROUND ELEVATION 45 ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" HSA, B-56 Truck Mounted (Exploration Geo.) Top 2' HA

DRIVE WEIGHT 140 LBS (wireline) DROP 30 INCHES

SAMPLED BY JJC LOGGED BY JJC REVIEWED BY RH

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 4-inches thick.
								AGGREGATE BASE: Approximately 6-inches thick.
	X	X	15	26.4	94.1		CL	Brown, moist, stiff, lean CLAY. Dark brown.
		X	17					
10		X	13					Brown.
		X	29	28.0	95.6		CH	Grayish brown, moist, very stiff, fat CLAY.
20		X	15	23.7	99.8		CL	Grayish brown with orange staining, wet, stiff, sandy lean CLAY.
		X	25					Gray, very stiff.
30								Total Depth = 25.0 feet Backfilled the borehole with drill cuttings and patched with concrete shortly after drilling on 10/23/2020. Notes: Groundwater was encountered at 18 feet in the borehole 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 (Google, 2020).
40								

DATE DRILLED 10/23/2020 BORING NO. B-4

GROUND ELEVATION 43 ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" HSA, B-56 Truck Mounted (Exploration Geo.) Top 2' HA

DRIVE WEIGHT 140 LBS (wireline) DROP 30 INCHES

SAMPLED BY JJC LOGGED BY JJC REVIEWED BY RH

DEPTH (feet)	SAMPLES: Bulk	SAMPLES: Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 4-inches thick.
	X						CL	Brown, moist, firm, lean CLAY; trace sand and gravel. Dark brown.
		X	9	22.3	98.8			
		X	16					
10		X	14	28.1	93.1		CH	Grayish brown, moist, stiff, fat CLAY.
		X	15	▽ 29.0	93.4			Wet.
20								Total Depth = 15.0 feet Backfilled the borehole with drill cuttings and patched with concrete shortly after drilling on 10/23/2020. Notes: Groundwater was encountered at 13.5 feet in the borehole 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 (Google, 2020).
30								
40								

DATE DRILLED: 10/23/2020 | BORING NO. B-5

GROUND ELEVATION: 42 ± (MSL) | SHEET 1 OF 1

METHOD OF DRILLING: 8" HSA, B-56 Truck Mounted (Exploration Geo.) Top 2' HA

DRIVE WEIGHT: 140 LBS (wireline) | DROP: 30 INCHES

SAMPLED BY: JJC | LOGGED BY: JJC | REVIEWED BY: RH

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	X						CL	ASPHALT CONCRETE: Approximately 2-inches thick. Brown, moist, stiff, lean CLAY; trace sand. Dark brown.
		X	11	21.6	90.8			
		X	11					
10		X	13	28.4	92.9		CH	Grayish brown, moist, stiff, fat CLAY.
		X	15	29.8 (▽)	90.4			Wet.
20								
30								
40								

Total Depth = 15.0 feet

Backfilled the borehole with drill cuttings and patched with concrete shortly after drilling on 10/23/2020.

Notes:
Groundwater was encountered at 13.5 feet in the borehole 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 (Google, 2020).

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

DATE DRILLED 10/23/2020 BORING NO. B-6

GROUND ELEVATION 42 ±(MSL) SHEET 1 OF 1

METHOD OF DRILLING 3-Inch Hand Auger

DRIVE WEIGHT N/A DROP N/A

SAMPLED BY JJC LOGGED BY JJC REVIEWED BY RH

Depth (feet)	Bulk	Driven	Blows/Foot	Moisture (%)	Dry Density (PCF)	Symbol	Classification U.S.C.S.	Description/Interpretation
0								ASPHALT CONCRETE: Approximately 4-inches thick.
							CL	Brown, moist, stiff, lean CLAY; trace sand and gravel.
								Total Depth = 5.0 feet Backfilled the borehole with drill cuttings and patched with concrete shortly after drilling on 10/23/2020. Notes: Groundwater was not encountered in the borehole 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 (Google, 2020).
10								
20								
30								
40								

DEPTH (feet)	SAMPLES: Bulk	SAMPLES: Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							CL	ASPHALT CONCRETE: Approximately 2-inches thick. Brown, moist, stiff, lean CLAY; trace sand and gravel. Dark brown to light brown.
			12					Dark brown
			16	22.6	92.9			
10			13	29.7	89.9		CH	Grayish brown with orange staining, wet, stiff, fat CLAY.
			16	28.9	93.9			
20			20				CL	Grayish brown, wet, stiff, sandy lean CLAY.
			26	36.0	82.4			Gray, very stiff.
30								Total Depth = 25.0 feet
40								

DATE DRILLED 10/23/2020 BORING NO. B-7

GROUND ELEVATION 43 ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" HSA, B-56 Truck Mounted (Exploration Geo.) Top 2' HA

DRIVE WEIGHT 140 LBS (wireline) DROP 30 INCHES

SAMPLED BY JJC LOGGED BY JJC REVIEWED BY RH

Total Depth = 25.0 feet

Backfilled the borehole with drill cuttings and patched with concrete shortly after drilling on 10/23/2020.

Notes:
Groundwater was encountered at 8.0 feet in the borehole 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 (Google, 2020).

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 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 accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings 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 and C-2. 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 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-3.

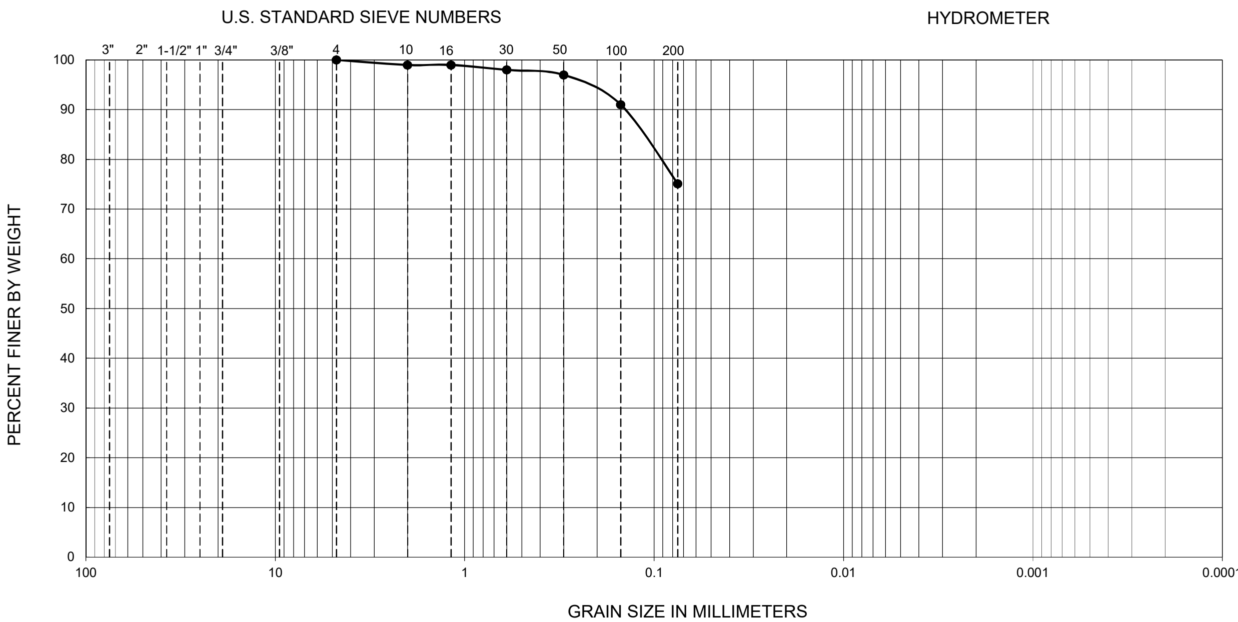
Consolidation Test
A consolidation test was performed on a selected relatively undisturbed soil sample in accordance with ASTM D 2435 04. The sample was inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are summarized on Figure C-4.

Expansion Index Test
The expansion index of a selected material was evaluated in accordance with ASTM D 48210. 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-5.

Unconfined Compression Test
Unconfined compression tests were performed on relatively undisturbed samples in accordance with ASTM D 2166. The test results are shown on Figure C-6.

R Value
The resistance value, or R value, for site soils was evaluated in accordance with California Test (CT) 301. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are shown on Figure C-7.

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-1	17.5-18.0	--	--	--	--	--	--	--	--	75	CL

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

FIGURE C-1

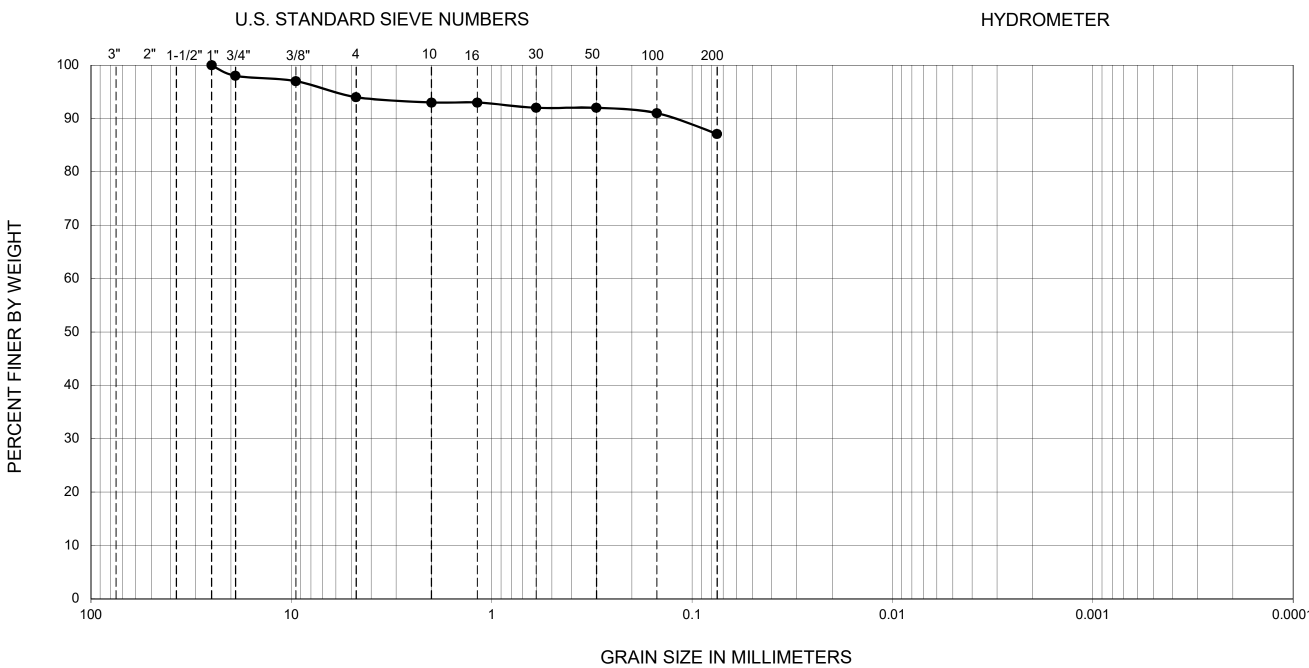
GRADATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

1535-1575 INDUSTRIAL AVENUE
SAN JOSE, CALIFORNIA

403870001 | 11/20

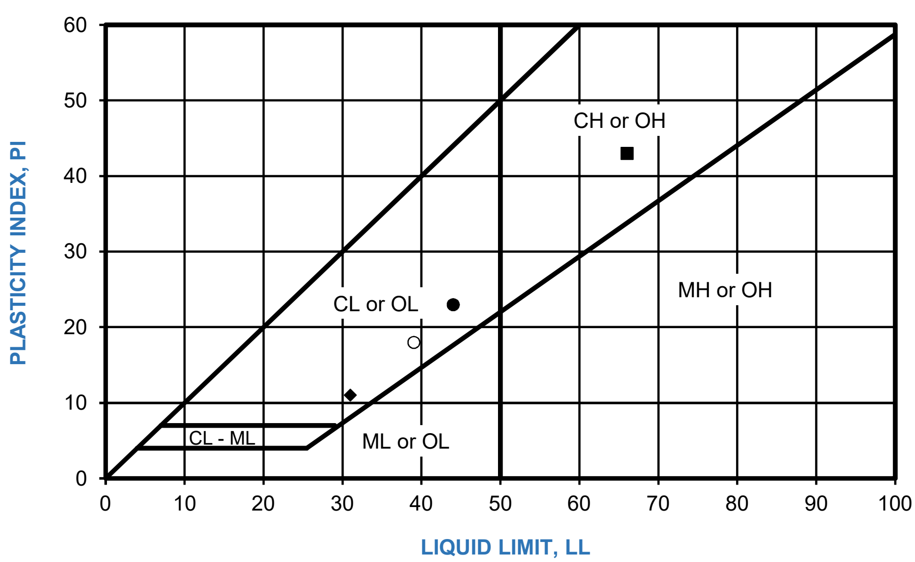
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



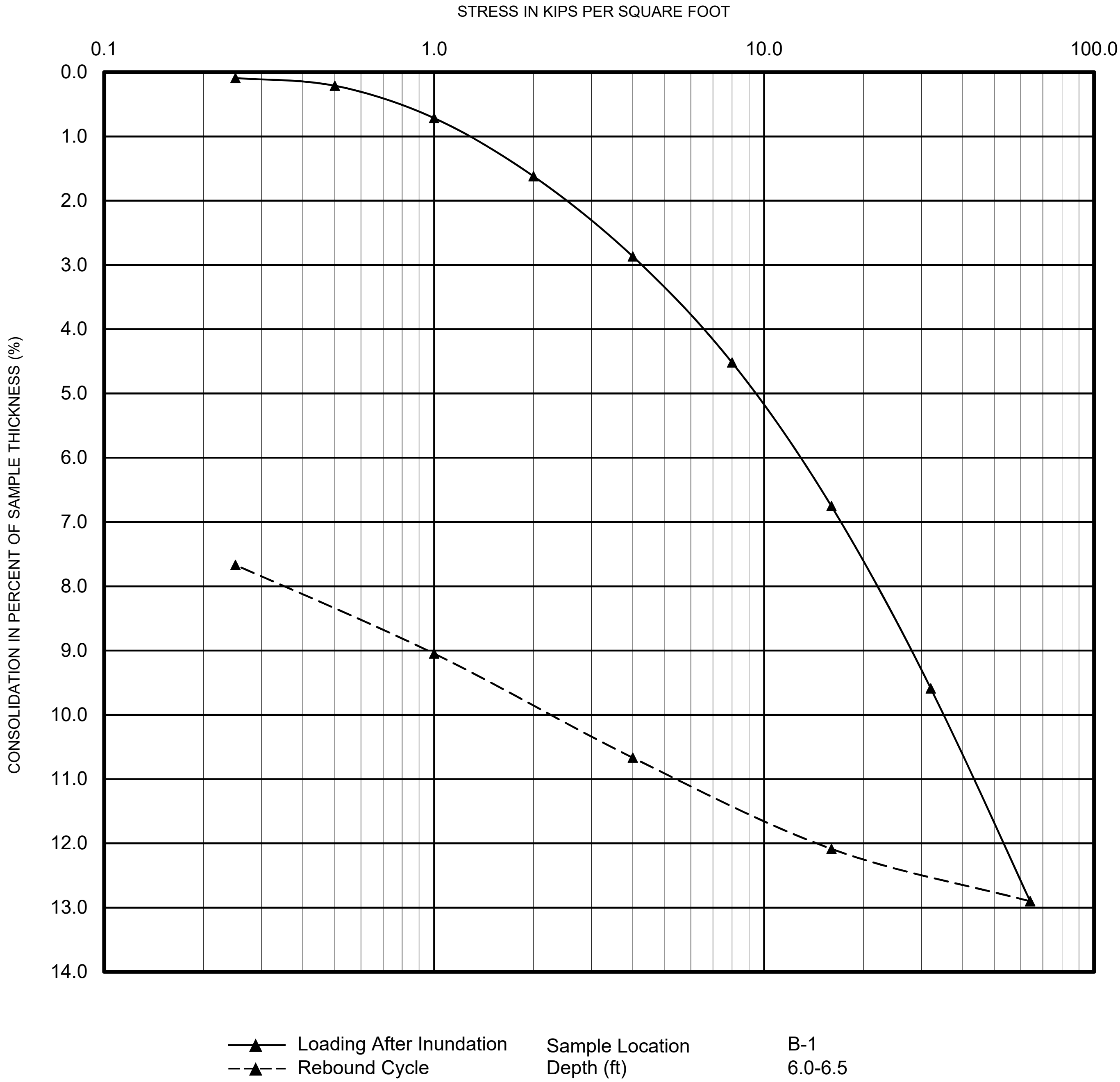
Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-7	19.0-19.5	--	--	--	--	--	--	--	--	87	CL

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-1	2.0-2.5	44	21	23	CL	CL
■	B-1	14.0-14.5	66	23	43	CH	CH
◆	B-4	3.0-3.5	31	20	11	CL	CL
○	B-7	5.5-6.0	39	21	18	CL	CL



PERFORMED IN ACCORDANCE WITH ASTM D 4318



Sample Location	B-1
Depth (ft)	6.0-6.5
Soil Type	CH

PERFORMED IN ACCORDANCE WITH ASTM D 2435

FIGURE C-4

CONSOLIDATION TEST RESULTS

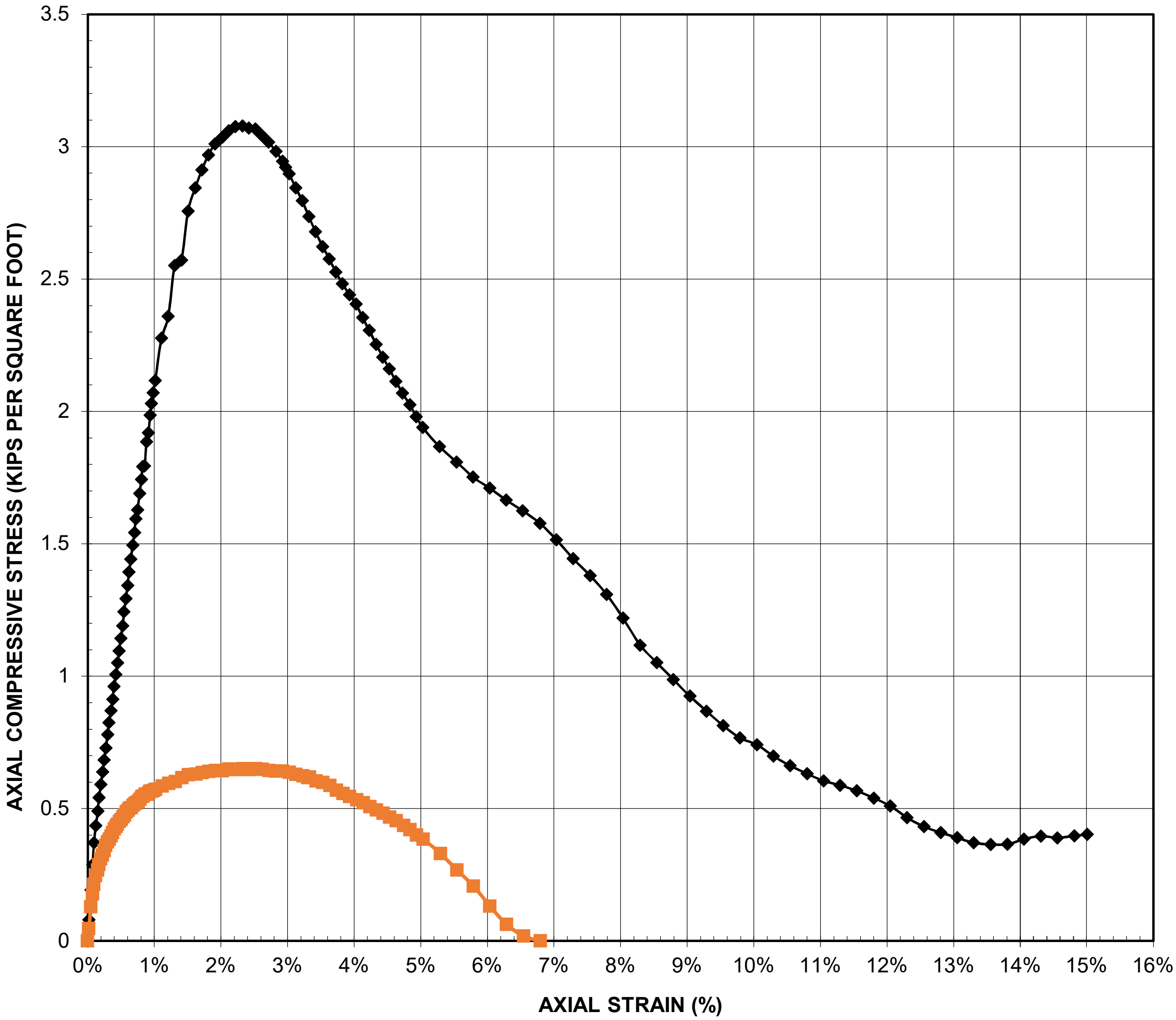
Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

1535-1575 INDUSTRIAL AVENUE
SAN JOSE, CALIFORNIA

403870001 | 11/20

SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
B-7	0.0-2.0	11.4	105.1	23.5	0.038	38	Low

PERFORMED IN ACCORDANCE WITH ASTM D 4829



SYMBOL	DESCRIPTION	SOIL TYPE	SAMPLE LOCATION	SAMPLE DEPTH (ft.)	MOISTURE CONTENT *w*, (%)	DRY DENSITY γ_d, (pcf)	STRAIN RATE (%/min.)	UNDRAINED SHEAR STR s_u, (ksf)
◆	Lean Clay	CL	B-1	3.0-3.5	17.9	105.0	1.00	1.54
■	Lean Clay	CL	B-2	6.0-6.5	23.3	91.0	1.00	0.33

PERFORMED IN ACCORDANCE WITH ASTM D 2166

FIGURE C-6

UNCONFINED COMPRESSION RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

1535-1575 INDUSTRIAL AVENUE
SAN JOSE, CALIFORNIA

403870001 | 11/20

SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R-VALUE
B-6	0.0-5.0	Lean Clay	8

PERFORMED IN ACCORDANCE WITH ASTM D 2844/CT 301

FIGURE C-7

R-VALUE TEST RESULTS

1535-1575 INDUSTRIAL AVE
SAN JOSE, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

APPENDIX D

Corrosivity Testing (CERCO Analytical)

California State Certified Laboratory No. 2153

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

12 November, 2020

Job No. 2010195
Cust. No.13270

Mr. David Seymour
Ninyo & Moore
2149 O'Toole Avenue, Suite 30
San Jose, CA 95131

Subject: Project No.: 403870001
Project Name: LBA 1535 Industrial Avenue, San Jose
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Seymour:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on October 29, 2020. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

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

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

The sulfate ion concentration is 90 mg/kg and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at this location.

The pH of the soil is 7.86 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 320-mV and is indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

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

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

Very truly yours,
CERCO ANALYTICAL, INC.

Cheryl McMillen for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

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

Client:	Ninyo & Moore
Client's Project No.:	403870001
Client's Project Name:	LBA 1535 Industrial Avenue, San Jose, CA
Date Sampled:	23-Oct-20
Date Received:	29-Oct-20
Matrix:	Soil
Authorization:	Signed Chain of Custody

Date of Report: 12-Nov-2020

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2010195-001	B-1/0-2'	320	7.86	-	1,100	-	N.D.	90

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	10-Nov-2020	10-Nov-2020	-	11-Nov-2020	-	10-Nov-2020	10-Nov-2020

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

__Quality Control Summary__ - **All laboratory quality control parameters were found to be within established limits**

Chain of Custody 2010195

1100 Willow Pass Court
Concord, CA 94520-1006
925 462 2771
Fax: 925 462 2775

CERCO analytical

Job No.	CU#	Client Project I.D.
403870001	13270	

Schedule
Analyte

Date Sampled	Date Due
10/23/20	

Full Name David Seymour Phone 4084359000 X 15308
Fax

Company and/or Mailing Address Ninyo & Moore 2149 O'Toole Avenue San Jose Cell ☒

Sample Source LBA 1535 Industrial Avenue San Jose CA

ANALYSIS — ASTM

Lab No.	Sample I.D.	Date	Time	Matrix	Contain.	Size	Preserv.	Qty.	Redox Potential	pH	Sulfate	Chloride	Resistivity-100% Saturated			Brief Evaluation						
001	B-1/0-2'			S					X	X	X	X	X			X						

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

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

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

		Date	Time
Relinquished By:	BCS	10/28/20	11:00am
Received By:	[signature]	10/29/20	10:30
Relinquished By:			
Received By:			
Relinquished By:			
Received By:			

Comments:

THERE IS AN ADDITIONAL CHARGE FOR EXTRUDING SOIL FROM METAL TUBES

Email Address dseymour@ninyoandmoore.com

APPENDIX E

Percolation Testing

APPENDIX E

PERCOLATION TESTING

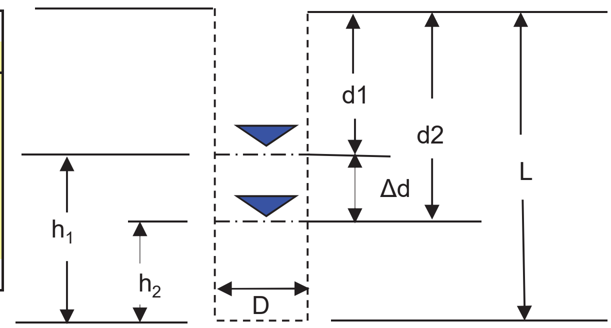
Field Procedure for Percolation Testing

The infiltration characteristics of the site soil were evaluated by field percolation testing. The test hole was excavated a depth of approximately 2 feet, with a diameter of about 8 inches. The subsurface conditions encountered in the test hole consisted of lean clay. The conditions encountered in the test hole are noted on Figure E-1. After cleaning the test hole of loose material, water was added to the test hole to achieve a water level approximately 18 inches below the top of the test hole. The drop in the water level was recorded over periodic intervals. Water was added to the test hole between measurement intervals to maintain sufficient water levels in the hole for percolation. The percolation rate reported is the percolation rate over the last measurement interval. The infiltration rate is the percolation rate adjusted by a reduction factor to exclude exfiltration occurring through the sidewalls of the test hole. The results of the percolation testing are presented on Figure E-1.

Project =	LBA 1535-1575 INDUSTRIAL AVE
Project No. =	403870001
Depth of Boring, L (ft) =	2.0
Diameter of Boring, D (in) =	8.0
Diameter of Pipe (in) =	0.0
Initial Depth to Water, d1 (in), (Final Period) =	18.00
Initial Height of Water, h1 (in), (Final Period) =	6.00
Water Level Drop, Δd (in), (Final Period) =	0.25
Reduction factor, Rf =	2.5

h1 = L - d1 (in inches)

Rf = ((2h1 - Δd)/DIA) +1



Test No. (Hole No.)	Time (hr:min)	Elapsed Time (min)	Depth to Water, d (in)	Water Level, h (in)	Change in Water Level, Δd (in)	Time Interval (hour)	Percolation Rate (inch/hour)	Adjusted Percolation Rate (inch/hour)
P-1	11:15	0	18.00	6.00				
	11:30	15	18.25	5.75	0.25	0.25	1.0	0.41
	11:30	15	18.00	6.00				
	11:45	30	18.25	5.75	0.25	0.25	1.0	0.41
	11:45	30	18.00	6.00				
	12:00	45	18.25	5.75	0.25	0.25	1.0	0.41
	12:00	45	18.00	6.00				
	12:15	60	18.25	5.75	0.25	0.25	1.0	0.41
	12:15	60	18.00	6.00				
	12:30	75	18.25	5.75	0.25	0.25	1.0	0.41
	12:30	75	18.00	6.00				
	12:45	90	18.25	5.75	0.25	0.25	1.0	0.41
	12:45	90	18.00	6.00				
	1:00	105	18.25	5.75	0.25	0.25	1.0	0.41
	1:00	105	18.00	6.00				
	1:15	120	18.25	5.75	0.25	0.25	1.0	0.41
	1:15	120	18.00	6.00				
	1:30	135	18.25	5.75	0.25	0.25	1.0	0.41
	1:30	135	18.00	6.00				
	1:45	150	18.25	5.75	0.25	0.25	1.0	0.41
	1:45	150	18.00	6.00				
	2:00	165	18.25	5.75	0.25	0.25	1.0	0.41
	2:00	165	18.00	6.00				
	2:15	180	18.25	5.75	0.25	0.25	1.0	0.41

FIGURE E-1

Geotechnical & Environmental Sciences Consultants

PERCOLATION TEST RESULTS

1535-1575 INDUSTRIAL AVE
SAN JOSE, CALIFORNIA

403870001 | 10/2020

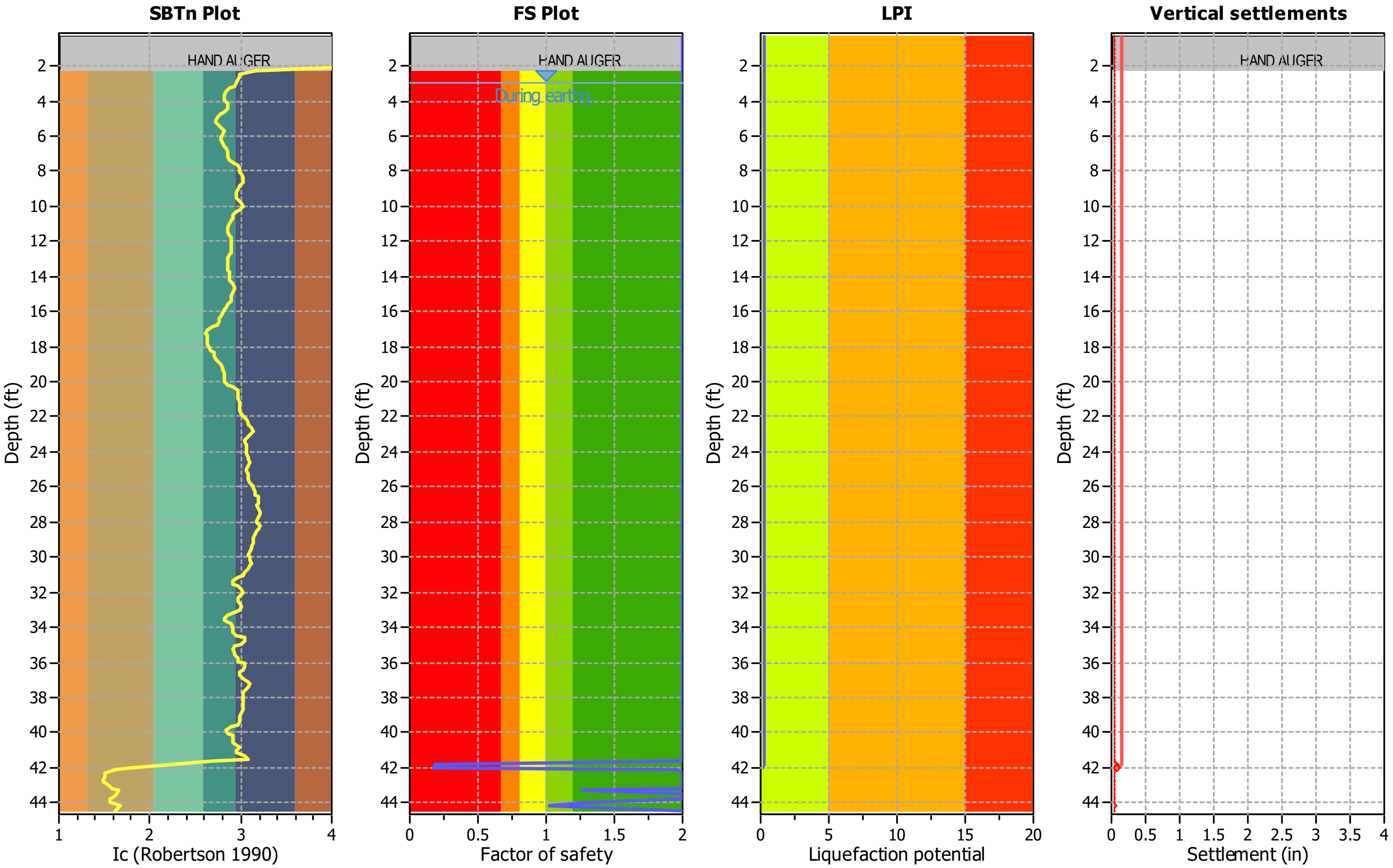
APPENDIX F

Liquefaction Analysis Plots

Ninyo & Moore
Geotechnical & Environmental Science Consultants
2149 O'Toole Avenue, Suite 30, San Jose, California 95131
www.ninyoandmoore.com

Project: 403870001 - 1535-1575 Industrial Road Project
Location: 1535 and 1575 Industrial Road, San Jose, California

CPT: CPT-1
Total depth: 44.46 ft



Analysis method:	B&I (2014)	G.W.T. (in-situ):	3.50 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.90	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.70	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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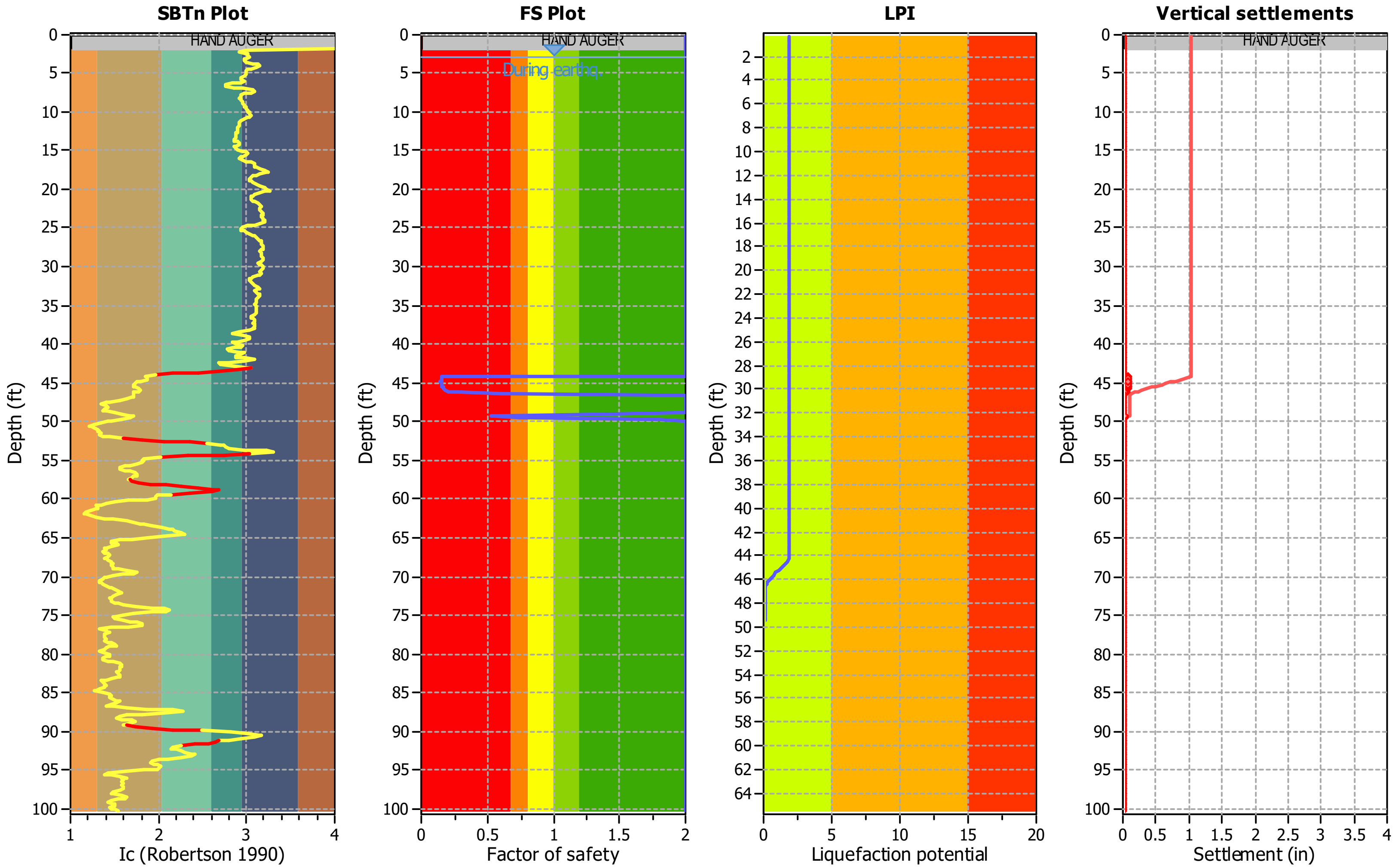
Ninyo & Moore
Geotechnical & Environmental Science Consultants
2149 O'Toole Avenue, Suite 30, San Jose, California 95131
www.ninyoandmoore.com

Project: 403870001 - 1535-1575 Industrial Road Project

Location: 1535 and 1575 Industrial Road, San Jose, California

CPT: CPT-2

Total depth: 100.23 ft



Analysis method:	B&I (2014)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	3.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.90	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.70	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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2149 O’Toole Avenue, Suite 30 | San Jose, California 95131 | p. 408.435.9000

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