Geotechnical Engineering ● Engineering Geology

DUE DILIGENCE – PRIVILEGED AND CONFIDENTIAL

## Geotechnical Design Report

# PROPOSED RESIDENTIAL / RETAIL DEVELOPMENT

## 2655 The Alameda
## Santa Clara, California

Prepared for:

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Project No. TET 22-235E
May 26, 2022

TETRA TECH

Project No. TET 22-235E
May 26, 2022

Ms. Anjuli Habbas
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Campbell, CA  95008

Subject: **DUE DILIGENCE – PRIVILEGED AND CONFIDENTIAL**
**GEOTECHNICAL DESIGN REPORT**
**PROPOSED RESIDENTIAL / RETAIL DEVELOPMENT**
**2655 The Alameda**
**Santa Clara, CA 95050**

Dear Ms. Habbas:

Tetra Tech is pleased to submit the results of our geotechnical investigation for the proposed residential / retail development located at 2655 The Alameda in the City of Santa Clara, California. The purpose of our investigation was to evaluate the subsurface conditions at the property and to provide recommendations for design and construction of the proposed development. We understand that this information will be utilized for your due-diligence assessment of the property as well as for future design and construction purposes should you continue to move forward with the project. This report includes a brief description of the proposed development, a discussion regarding the field exploration and laboratory testing, a description of subsurface conditions, a discussion on engineering seismology and geological hazards, and provides geotechnical conclusions and recommendations. The appendices to the report include logs of our exploratory borings, results of laboratory tests, seismic demand, and liquefaction evaluation.

We appreciate the opportunity to provide our professional services on this project. If you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned.

Respectfully submitted,
**Tetra Tech**

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# 1. INTRODUCTION

This report presents the results of Tetra Tech's geotechnical engineering evaluation and design recommendations for the proposed residential / retail development at 2655 The Alameda in the City of Santa Clara, Santa Clara County, California. The location of the site is shown on Figure 1 – Site Location Map.

The purpose of our investigation was to evaluate the subsurface conditions at the property and to provide recommendations for the design and construction of the proposed development. This report includes a brief description of the proposed development, a discussion of our field exploration, laboratory testing results, a description of subsurface conditions, a discussion regarding engineering seismology and geological hazards, and provides geotechnical conclusions and recommendations for design and construction of the proposed development.

## 2. SCOPE OF SERVICES

Tetra Tech's scope of services for this project consisted of the following tasks:

- Review the provided environmental reports for the site.
- Review aerial photographs, geotechnical literature, geologic maps, and seismic hazard maps relevant to the subject site.
- Notify Underground Service Alert (USA) prior to drilling for clearance of underground utilities.
- Procure a drilling permit from the Santa Clara Valley Water District.
- Perform a subsurface exploration consisting of drilling and sampling 2 soil borings to a depth of 31.5 and 81.5 feet.
- Conduct laboratory testing of selected samples recovered from the exploratory borings to evaluate geotechnical properties of the on-site soils.
- Process and evaluate the collected geotechnical data for use in developing geotechnical recommendations including the following items:
    - General subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials,
    - Liquefaction potential and seismic settlement of the on-site granular materials,
    - Sensitivity of the on-site fine-grained materials,
    - Suitability of on-site soils for the support of structures,
    - Seismic design parameters in accordance with 2019 California Building Code (2019 CBC),
    - Corrosion potential of the on-site soils to buried concrete and steel.
- Develop design and construction recommendations for the following items:
    - Temporary shoring,
    - Mat foundations including allowable bearing pressures, lateral resistance, and settlement estimates,
    - Static and seismic lateral earth pressures on basement walls,
    - Portland cement concrete pavement section for driveways.
- Prepare this report documenting reference maps and illustrations, collected field and laboratory data, and geotechnical recommendations for the design and construction of the proposed development.

## 3. SITE DESCRIPTION

The site is a triangular-shaped 0.4-acre property bordered by The Alameda to the northeast, by Park Avenue to the southwest and by the existing Safeway Center parking lot to the southeast. The site is currently a vacant lot that is covered with dirt and with chain-link fencing along the perimeter of the site. Groundwater and vapor monitoring wells associated with previous environmental studies are also present at the site.

Topographically, the site is relatively level. However, the general site vicinity slopes very gently to the northeast. Based on Google earth imagery, the site elevation is approximately 74 to 75 feet.

Based on our review of background environmental information provided to us, the site was previously occupied by a former Unocal-branded service station that operated from approximately 1930 to 1993. In March of 1993 all the above- and below-ground facilities were removed. These facilities reportedly included two 10,000-gallon gasoline underground storage tanks (USTs), one 8,000-gallon diesel UST, a 500-gallon waste-oil UST, an additional 100-gallon storage tank, a clarifier, product lines, three product dispenser islands, a station building and some other associated improvements. Excavations up to 14.5 feet deep were reportedly made in the area where the fuel storage tanks had been located, and up to 3.5 feet deep in the area where the product lines and dispenser islands had been located. These excavations were reportedly backfilled with imported soil materials. However, specific documentation regarding the placement and compaction of the imported soil material was not included in the reviewed documents. The approximate excavation limits associated with the removal of the UST's and previous underground improvements are shown on Figure 2 – Former Excavation Limit Map.

## 4. PROJECT DESCRIPTION

The proposed development will entail the construction of a mixed-use residential / retail building that will occupy most of the site. Based on the preliminary development plans shown in Figures 3A, 3B and 3C – Preliminary Development Plan, the proposed building will consist of a 4-story structure with a 14,143-square-foot subterranean parking garage under the footprint of the building. The street level grade will be used for retail and residential units. Levels 2 through 4 will be used for residential units. Total building height, including roof parapet, is approximately 52 to 55 feet. The horizontal setback for the below grade parking garage adjacent to The Alameda and Park Avenue is 4 feet. To the southeast, along the boundary with the Safeway Center parking lot, the parking garage is to be positioned along the property line.

Neither site grading nor foundation plans have been prepared for the project at this time. However, based on the preliminary development plan, future construction for the building and underground parking garage will generally require excavations of approximately 12 to 15 feet over much of the site with maximum excavation depths extending in localized areas up to approximately 20 feet to accommodate elevator pits and other utilitarian infrastructure. The proposed excavations will remove any uncertified fill within the area. Considering that the proposed building structure is adjacent to or along the property line, a shoring system will be required to support the temporary excavations.

Considering the underlying geologic conditions of the site, as presented in the Geology and Subsurface Condition section of this report, we anticipate that the building will be supported on a structural mat foundation. The mat foundation will likely be occupying almost the entire triangular lot with maximum dimensions of approximately 250 feet by 137 feet by 283 feet. Foundation pressures are not expected to exceed 2 ksf although at the time of writing no information regarding structural loads is available. Once the actual dimensions of the building and the foundation loads are finalized, this office should be contacted to verify that the recommendations provided herein are still applicable.

## 5. FIELD INVESTIGATION

The subsurface soil and groundwater conditions were explored by Tetra Tech on April 15, 2022 and included drilling, logging, and sampling of 2 exploratory borings, Tt-1 and Tt-2. The locations of the 2 borings are shown on Figure 4 – Boring Locations Map. Relevant boring information including latitude and longitude, approximate ground surface elevation, and exploration depth is summarized in Table 1 below.

**Table 1**
**Soil Boring Locations Information**

| Boring No. | General Location | Latitude (degrees) | Longitude (degrees) | Approximate Depth (ft) | Approximate Ground Elevation (ft) |
|---|---|---|---|---|---|
| Tt-1 | North area of site | 37.346365 | -121.937575 | 31.5 | 75 |
| Tt-2 | East-central portion of site | 37.346236 | -121.937266 | 81.5 | 75 |

Prior to initiation of the field exploration program, a field reconnaissance was conducted to observe surface conditions and to mark the locations of the planned subsurface exploration. Underground Service Alert was notified of the exploratory boring locations at least 48 hours prior to drilling. A drilling permit was also obtained through the Santa Clara Valley Water District.

The borings were excavated using a truck-mounted CME 85 drill rig equipped with 8-inch diameter hollow-stem augers. Bulk, driven California-type ring samples, and Standard Penetration Test (SPT) samples were collected during the drilling and transported to a laboratory for testing. The SPT sampler consisted of a 2-inch outside diameter, 1.4-inch inside diameter split barrel without liners, while the California-type sampler consisted of a 3-inch O.D., and a 2.4-inch I.D. split barrel. The interior of the California-type sampler was lined with 1-inch-long brass rings.

SPT testing was performed using an SPT sampler driven by an automatic 140-pound hammer with a drop of 30 inches in general accordance with ASTM D1586. The total number of hammer blows required to drive the sampler the final 12 inches is termed the “blowcount.” The hammer calibration record indicated an average energy transfer ratio of 75 percent. Ring-type samples were collected by driving the California-type sampler using the same equipment as for the SPTs. Sampling was generally carried out at 2.5- or 5-foot vertical intervals.

The borings were surface logged by a Geologist in general accordance with the visual-manual procedure for description and identification of soils per ASTM D2488. The Geologist prepared the recovered samples for subsequent laboratory testing. At the completion of drilling, the borings were backfilled with cement-bentonite grout. The excavated materials were drummed and disposed of at an approved disposal facility. The exploratory boring logs are presented in Appendix A – Logs of Exploratory Borings.

## 6. LABORATORY TESTING

Laboratory tests were performed on selected samples recovered from the borings to aid in the classification of soils and to evaluate pertinent engineering properties of the foundation soils. The following tests were performed:

- In-situ Moisture Content and Dry Density, ASTM D2937;
- Percent Passing #200, ASTM D1140;
- Atterberg Limits, ASTM D4318;
- Expansion Index, ASTM D4829;
- Consolidation, ASTM D2435;
- Direct Shear, ASTM D3080; and
- Corrosion Testing in Soils:
    pH and resistivity, CTM 643;
    Sulphates, CTM 417; and
    Chlorides, CTM 422.

Laboratory testing was performed in general accordance with applicable ASTM Standards and California Test Methods. Results of laboratory tests are presented in Appendix B – Results of Laboratory Testing. For ease of referral to the soil profile, selected laboratory results have been included on the boring logs in Appendix A.

## 7. GEOLOGIC AND SUBSURFACE CONDITIONS

Regionally, the site is in the Santa Clara Valley, which extends southeastward from San Francisco Bay and is a northwest / southeast trending valley within the Coast Ranges Geomorphic Province of Northern California. The Santa Clara Valley is a broad alluvium-filled basin located between the Santa Cruz Mountains to the southwest and the Diablo Range to the northeast. The sediments on the valley floor, as shown on the Figure 5 – Regional Geologic Map, are generally comprised of alluvial sands, silts, clays, and gravels associated with Holocene-age alluvial fan, levee, and active stream channel deposits, and with marine estuary deposits located along the bay margins. Major right-lateral strike-slips faults occur on either side of the Santa Clara Valley. These faults include the San Andreas fault on the southwest side of the valley and the Hayward and Calaveras faults on the northeast side of the valley.

The subsurface soils encountered during Tetra Tech's field exploration, consisted of undocumented artificial fill soils over native alluvial deposits. The soils were found to be generally consistent with the materials encountered during previous environmental studies at the site. Detailed descriptions of the soil units encountered during our field exploration are presented below and in the boring logs included Appendix A.

### 7.1. Undocumented Artificial Fill (af)

Undocumented artificial fill soils associated with previous site development and the demolition of former above- and below-ground site improvements are present over much of the site. These fills, based on our research, consist of both locally derived materials as well as imported fill soils. The locally derived artificial fill materials, as observed in exploratory boring Tt-1, were encountered to a depth of approximately 3 feet and consisted of brown clayey sands that were generally dry to damp and contained trace amounts of concrete fragments less than 4 inches in size. Imported fill soils were not encountered during our field exploration. They were reportedly used during backfill operations associated with the removal of former USTs and other assorted underground improvement excavations at the site in 1993. According to the referenced report by Pacific Environmental Group, Inc. (1994), the backfilled soils were up 14.5 feet deep. Specific documentation concerning the characteristics of these fills and backfill operations were not included in their report. The approximate limits of the former excavations that were backfilled with imported soils, as presented in the referenced report by Pacific Environmental Group, Inc. (1994), are shown on Figure 2.

### 7.2. Alluvium (Qya)

Quaternary-age alluvial deposits underlie the fill materials throughout the entire site and were encountered to the maximum explored depth of 81.5 feet during our field exploration. The alluvial soils encountered within the upper 5 feet to 12 feet of the ground surface generally consisted of brown sandy clays that were damp, soft to stiff, and contained gravels up to 3 inches in diameter. Below the upper alluvium to a depth of approximately 55 feet, the encountered alluvial soils consisted primarily of firm to stiff lean clays that were olive gray, yellowish brown, blueish gray, and black in color, and moist to wet. Also included in this sequence were occasional discontinuous silt, sand, and clayey sand layers. From about 55 feet to 73 feet in depth, the alluvial soils consisted primarily of interlayered clayey sand, silt, lean clay, and silty sand. These soils were generally blueish gray in color, medium dense or very stiff, and wet. Poorly graded gravels with coarse-grained sands were encountered in boring Tt-2 at a depth between approximately 73 and 81.5 feet. These materials were olive in color, very dense, and wet.

### 7.3. Groundwater

At the time of our exploration, groundwater was not noted in exploratory boring Tt-1 to a depth of 31.5 feet prior to backfill. However, exploratory boring Tt-2 had groundwater at a depth of about 18.5 feet after being left open for several hours prior to backfill. Groundwater depths measured by Tetra Tech at the existing groundwater monitoring wells on April 15, 2022 varied from about 16 to 20 feet. Based on our review of the available groundwater monitoring well data since 1993, groundwater levels beneath the site have fluctuated substantially over the years with measured groundwater depths as shallow as 4 feet in March 2019 and as deep as 28 feet in March 2015. Mapping by the State of California (California Department of Conservation, Division of Mines and Geology, 2002) for the San Jose West 7.5-minute Quadrangle indicates that the historic high groundwater level at the site is at a depth of about 10 feet (see Figure 6 – Historic High Groundwater Map).

A groundwater depth of 4 feet should be considered for the design of the proposed building. Furthermore, groundwater levels may fluctuate due to seasonal variations, rainfall, irrigation, or other factors and will need to be considered during future grading operations and site construction.

# 8. GEOLOGIC HAZARDS

## 8.1. General Seismic Setting

The Northern California region is known to be seismically active. Earthquakes occurring within approximately 60 miles of the site are generally capable of generating ground shaking of engineering significance to the proposed construction. The project area is located in the general proximity of several active faults as shown in Figure 7 – Regional Fault and Seismicity Map.

The closest active faults to the site include the Monte Vista-Shannon fault located approximately 6.57 miles southwest of the site, the Hayward-Rodgers Creek fault located approximately 8.85 miles north of the site, and the Calaveras fault approximately 9.81 miles to the northeast. The North San Andreas fault is the most significant fault in the area in terms of Maximum Magnitude and is located approximately 10.6 miles southwest of the site. Table 2 summarizes known active faults within a distance of engineering significance of approximately 60 miles from the project site as identified by the USGS Quaternary Fault Database and in the 2008 National Seismic Hazard Maps (https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=5a6038b3a1684561-a9b0aadf88412fcf and https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query-_main.cfm).

## 8.2. Historical Earthquakes

The epicenters of a large number of historical earthquakes with a magnitude of $M_w$ 5.0 and higher have been recorded within a 60-mile radius of site. The most significant historic earthquake near the project site was the 1906 San Francisco $M_w$ 7.9 earthquake located about 43.7 miles northwest of the site. Table 3 summarizes historic earthquakes with a magnitude greater than $M_w$ 6.5 within a distance of approximately 60 miles from the project site obtained using the USGS Earthquake Catalog (https://earthquake.usgs.gov/earthquakes/search/).

## Table 2
## Summary of Active Faults

Referenced Site Latitude and Longitude: 37.346191°, -121.937395°

| Fault Name | Approximate Fault Distance to Site (miles) | Slip Sense | Maximum Moment Magnitude $M_w$ |
|---|---|---|---|
| Monte Vista-Shannon | 6.57 | thrust | 6.50 |
| Hayward-Rodgers Creek | 8.85 | strike slip | 7.33 |
| Calaveras | 9.81 | strike slip | 7.03 |
| N. San Andreas | 10.6 | strike slip | 8.05 |
| Zayante-Vergeles | 17.88 | strike slip | 7.00 |
| Greenville Connected | 24.19 | strike slip | 7.00 |
| San Gregorio Connected | 24.63 | strike slip | 7.50 |
| Mount Diablo Thrust | 27.19 | thrust | 6.70 |
| Monterey Bay-Tularcitos | 31.04 | strike slip | 7.30 |
| Great Valley 7 | 34.99 | thrust | 6.90 |
| Ortigalita | 36.5 | strike slip | 7.10 |
| Green Valley Connected | 38.32 | strike slip | 6.80 |
| Quien Sabe | 42.87 | strike slip | 6.60 |
| Great Valley 8 | 43.26 | thrust | 6.80 |
| Great Valley 5, Pittsburg Kirby Hills | 46.47 | reverse | 6.70 |
| Rinconada | 47.32 | strike slip | 7.50 |
| Great Valley 9 | 54.99 | thrust | 6.80 |
| West Napa | 58.9 | strike slip | 6.70 |

**Table 3**
**Historic Earthquakes in Northern California**
Referenced Site Latitude and Longitude: 37.346191°, -121.937395°

| Year | Location | Earthquake Magnitude | Epicenter Location | Distance from Site (miles) |
|---|---|---|---|---|
| 1989 | Loma Prieta | 6.9 $M_w$ | 37.036°N/-121.880°W | 21.6 S |
| 1911 | 2 km SE of Morgan Hill | 6.5 $M_w$ | 37.111°N/-121.637°W | 23.2 SE |
| 1906 | San Francisco | 7.9 $M_w$ | 37.750°N/-122.550°W | 43.7 NW |
| 1868 | Hayward | 6.8 $M_L$ | 37.700°N/-122.100°W | 26.0 N-NW |
| 1865 | South of San Jose | 6.5 $M_w$ | 37.200°N/-121.900°W | 10.3 S |
| 1840 | Near San Juan Bautista | 6.5 $M_w$ | 36.850°N/-121.500°W | 41.9 SE |
| 1838 | San Francisco | 7.4 $M_w$ | 37.300°N/-122.150°W | 10.6 SW |

Notes:
* $M_w$ refers to Moment Magnitude scale
$M_L$ refers to Local Magnitude scale, commonly referred to as "Richter magnitude"

## 8.3. Seismic Hazards and Surface Fault Rupture Potential

The engineering seismology study for the subject site included reviewing local and regional fault maps, reviewing historical earthquake data, and reviewing regulatory maps prepared by the State and local governing agencies. Specifically, the following engineering seismology issues were addressed:

### 8.3.1. Seismic Hazards

The Seismic Hazards Mapping Act (SHMA) of 1990 directs the California Geological Survey (CGS, formerly California Department of Conservation, Division of Mines and Geology (CDMG)) to identify and map areas prone to earthquake hazards of liquefaction, earthquake-induced landslides and amplified ground shaking.

Maps of seismic hazard zones are issued by the California Geological Survey (CGS, formerly California Department of Conservation, Division of Mines and Geology (CDMG)) in accordance with the Seismic Hazards Mapping Act enacted in 1990. The intent of the Seismic Hazards Mapping Act is to provide for a statewide seismic hazard mapping and technical advisory program to assist cities and counties in developing compliance requirements to protect the public health and safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure and other seismic hazards caused by earthquakes.

Based on the Official Seismic Hazard Zones Map, released February 7, 2002, for the San Jose West Quadrangle, the proposed development is located within an area identified by the State of

California as subject to the hazard of liquefaction. The site is not located within an area identified by the State of California as subject to the hazard of seismically induced landslides (see Figure 8 – Seismic Hazard Zones Map).

### 8.3.2. Surface Fault Rupture

Earthquake Fault Zones (known as Special Studies Zones prior to 1994) have been established in accordance with the Alquist-Priolo Special Studies Zones Act enacted in 1972. The Act directs the State Geologist to delineate the regulatory zones that encompass surface traces of active faults that have a potential for future surface fault rupture. The purpose of the Alquist-Priolo Act is to regulate development near active faults in order to mitigate the hazard of surface fault rupture.

Based on our field exploration and literature review there are no known surface traces of any active or potentially active faults that pass directly through or project towards the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site is considered low.

## 8.4. Liquefaction Potential and Dynamic Settlement Assessment

The site, as previously discussed, is within an area identified by the State of California as subject to the hazard of liquefaction. Liquefaction of soils can be caused by ground shaking during earthquakes and is generally known to occur in saturated or nearly saturated cohesionless soils at depths shallower than about 50 feet. Research and historical data indicate that loose, relatively clean granular soils and low plasticity silts are susceptible to liquefaction whereas the stability of clayey silts, silty clays and clays are not typically adversely affected by ground shaking.

Dynamic settlement can occur in both unsaturated and saturated sands when loose to medium-dense granular soils tend to undergo contractive volumetric changes during ground shaking. The following sections present our assessment of liquefaction potential and dynamic settlement at the site.

### 8.4.1. Groundwater Level for Liquefaction Analysis

For the liquefaction analysis a groundwater depth of 4 feet was used based on the historic high groundwater as discussed in the Groundwater section of this report.

### 8.4.2. Liquefaction Seismic Demand

Based on the Structural Engineers Association of California (SEAOC) and the Office of Statewide Health Planning and Development (OSHPD) website application (https://seismicmaps.org/) the seismic demand was evaluated for the site with latitude 37.346191° and longitude -121.937395°. The mapped Geometric Mean Peak Ground Acceleration ($PGA_M$) for a ground motion corresponding to the Maximum Considered Earthquake (MCE) for a Site Class E was determined to be 0.605g. From the USGS Seismic Hazard Interactive Deaggregation website (https://earthquake.usgs.gov/hazards/interactive/) this ground motion corresponds approximately to a predominant earthquake with a magnitude of $M_W$ 7.5. The largest contributors to the seismic hazard at the site are the Hayward-Rodgers creek fault located about 8.85 miles northeast of the site, the San Andreas fault located about 10.6 miles southwest from the site, and the Calaveras fault located about 9.81 miles northeast of the site. These ground motion parameters were used in the liquefaction analyses. A summary of the seismic demand parameters is presented in Appendix C.

### 8.4.3. Evaluation of Liquefaction Potential and Sensitivity Analyses

The analyses were based on SPT blowcounts obtained from the 2 exploratory borings and laboratory test results. The field SPT blowcounts were converted considering the energy ratio correction factor $C_E$ of 1.25 to reflect the hammer calibration record. The borehole diameter factor $C_B$ of 1.0 was used per SP117 based on the internal diameter of the hollow stem auger system used during drilling. The blowcounts recorded for soils driven with the 3-inch O.D. California-type sampler with brass rings were converted to equivalent SPT blowcounts using a reduction factor of 0.7 as recommended by SP117.

The liquefaction potential of cohesionless (sandy) soils was evaluated in general accordance with the procedure published by Boulanger and Idriss and (2014) and in conformance with SP117A. The anticipated dynamic settlement of the saturated soils was evaluated using procedure by Yoshimine et al. (2006) which was further adjusted by a calibration factor of 0.9 as recommended by Cetin (2009).

Seismic sensitivity of fine-grained soils was evaluated based on the following 3 categories:

1. Soils with Plasticity Index < 7 (typically silts) are classified as fine-grained soils susceptible to liquefaction like coarse-grained soils.

2. Soils with Plasticity Index > 18 and a degree of sensitivity $S_t > 6$ are classified as seismically sensitive soils susceptible to significant loss of strength during seismic shaking and require additional evaluation. The sensitivity of the on-site fine-grained soils was evaluated based on the water content, Atterberg limits, and effective vertical stresses using the procedures suggested by Terzaghi, Peck and Mesri (1996).

3. Fine-grained soils falling outside the two categories described above are considered to behave like clays and are not considered susceptible to liquefaction or cyclic softening.

Evaluation of the liquefaction potential of granular soils and fine-grained soils with Plasticity Index less than 7 are presented in Appendix D and are summarized in Table 4 in the next section of the report. Evaluation of the sensitivity of the saturated fine-grained soils (i.e., Category 2) to ascertain the potential for cyclic softening was performed based on the boring logs for materials encountered at depths between 4 and 50 feet. Details on the sensitivity evaluation of fine-grained soils with a Plasticity Index greater than 18 are presented in Appendix D. The calculated sensitivity $S_t$ for these soils was less than 6. Therefore, the fine-grained soils encountered in the borings are not considered to be susceptible to cyclic softening during the design earthquake event.

### 8.4.4. Dynamic Settlement

Dynamic settlement can occur in both unsaturated and saturated sands when loose to medium-dense granular soils undergo volumetric changes during ground shaking. Dynamic settlement can occur in saturated sands due to liquefaction or in unsaturated sands due to densification of the soil matrix. The anticipated dynamic settlement of the saturated soils at the site was evaluated using SPT data from the current field exploration in accordance with the procedures outlined by Yoshimine et al (2006) and Cetin (2009).

The unsaturated dynamic settlement was calculated based on SPT data according to the procedure outlined by Pradel (1998a and 1998b). Table 4 presents the results of the liquefaction analyses and corresponding settlement evaluation as well as the dynamic settlement of unsaturated soils. Details of dynamic settlement analyses are presented in Appendix D.

As shown in Table 4, the total (combined) dynamic settlement was estimated to be about 1.0 to 1.5 inches. The differential seismic settlement was estimated to be 0.75 inches or less over a horizontal distance of 30 feet. Therefore, structural mitigation of the total and differential seismic settlement is considered acceptable for the project design.

**Table 4**
**Results of Liquefaction and Dynamic Settlement Analyses**

| Boring No. | Groundwater Depth (feet) | Liquefiable Zone Depth Interval (feet) | $FS_{liq}$ | Approximate Dynamic Settlement (inches) | | |
|---|---|---|---|---|---|---|
| | | | | Saturated Soils (liquefaction) | Unsaturated Soils | Combined |
| Tt-1 | 4 | 30-33 | 0.18 | 1.0 | negligible | 1.0 |
| Tt-2 | | 20-23<br>30-35 | 0.22<br>0.68 | 1.5 | | 1.5 |

### 8.5. Lateral Spreading

Due to the level topography and the absence of free face slopes lateral spreading is not considered to be a hazard at the site.

### 8.6. Landslide Hazard

The site is not located within a State designated hazard zone for earthquake-induced landslides. Due to the level topography the potential for landslides is not considered a hazard for the site.

### 8.7. Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from precipitation, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors which can cause unacceptable settlement or heave that could adversely affect the performance of structures or slabs supported on grade.

Expansion Index (EI) testing of the surficial sandy materials within the upper 5 feet showed an EI value of 4, indicating a very low expansion potential. The Expansion Index testing of fine-grained materials i.e., clays, at a depth of 15 feet showed an EI value of 60, indicating the presence of expansive soils. For the design purposes the on-site fine-grained soils should be considered to be expansive per CBC 2019 §1803.5.3, and appropriate recommendations are provided in the Recommendations section of this report.

### 8.8. Collapsible Soils

The phenomenon of hydro-consolidation is typically exhibited in geologically young, unconsolidated, low-density, loose, dry soils commonly present in arid to semi-arid regions. Collapsible soils are usually composed of granular particles that are supported by a clay or silt matrix that can be chemically cemented in place creating a porous structure. The bonds supporting this porous structure have enough shear strength to support loads at low moisture contents, however, once water is introduced the cemented bond structure breaks down and the granular particles are re-arranged causing significant volume loss.

Based on the consolidation testing it was observed that none of the samples collapsed upon wetting, therefore hydrocollapse is not considered a hazard at the site.

### 8.9. Tsunamis and Seiches

A tsunami is a sea wave generated by large-scale displacements of the ocean floor that causes a sudden surge of water onto the land. Tsunamis are most commonly caused by movement along faults and underwater landslides activated by earthquakes. Seiches are earthquake-induced displacements of water within an enclosed body of water such as a lake. Strong ground motions from an earthquake can cause the water to slosh back and forth onto land. The site is elevated at least 74 feet above sea level and is located at a substantial distance from a significant body of water within an enclosed basin. The site is also not located within a Tsunami Inundation Area, based on our review of the California Geological Survey Tsunami Hazard Area Map for the County of Santa Clara, dated July 8, 2021. Therefore, geologic hazards associated with a tsunami or seiches are not anticipated at the site.

### 8.10. Subsidence

Land subsidence is the lowering of the ground surface due to extraction or lowering of groundwater levels or other fluids (e.g., oil) within the subsurface soil pores. The fluid withdrawal causes the alluvial sediments in the basin to compact. Damage caused by subsidence can be visible soil cracks, fissures, or surface depression. The site is located in an area mapped by the United States Geological Survey (USGS) where either historical or current subsidence has been recorded (https://ca.water.usgs.gov/land_subsidence/california-subsidence-areas.html). Provided that groundwater management strategies that include subsidence mitigation are being employed in the area, ground subsidence beneath the site that could result in damage to future site improvements is unlikely to occur at the site.

# 9. DESIGN RECOMMENDATIONS

## 9.1. General

Based on the results of the field exploration and engineering analyses, it is Tetra Tech's opinion that the proposed development is feasible from a geotechnical standpoint, provided that the recommendations contained in this report are incorporated into the design plans and implemented during construction. The proposed building may be supported on a conventional mat foundation established on native soils. The primary design considerations identified from a geotechnical standpoint include:

- presence of deep relatively weak and compressible fine-grained soils (i.e., lean clay);
- presence of soft/wet soil conditions that may require special handling during excavation work as well as dry back, if used as compacted fills. Subgrade improvements are also expected where soft/wet soil conditions are exposed;
- potential for the presence of shallow groundwater level during construction that may require dewatering;
- need to design for uplift pressures;
- need for temporary shoring during the subterranean level excavation; and
- presence of expansive soils at the site.

The design recommendations presented below are based on Tetra Tech's current understanding of the project and the subsurface conditions of the site. Once the project configuration is finalized and the design is complete, Tetra Tech should review the plans and specifications to evaluate if the geotechnical design recommendations presented herein have been incorporated as intended. Although not considered likely to be the most efficient foundation system, alternatives to the proposed mat foundation, such as a combination of ground improvements and conventional shallow foundations, or deep foundations, e.g., piles, may be utilized for the support of the building. Specific recommendations for such alternatives are available upon request.

## 9.2. Site Preparation

The following sections present recommendations to prepare the site before the building foundations are built.

### 9.2.1. Clearing and Grubbing

Prior to commencement of the earthwork, the surface of areas to be graded should be cleared of any pavement, vegetation, undocumented fills, existing structures, trash and debris. Any subterranean installations not to be preserved, such as electrical lines, pipes, utility collectors, tanks, etc., should be relocated and/or abandoned in accordance with the recommendation of the Geotechnical Engineer and applicable regulations.

### 9.2.2. Subgrade Preparation

In order to create uniform and competent bearing conditions for the proposed building mat foundation and other site improvements, removal of undocumented existing fills and other unsuitable materials should be performed in accordance with the recommendations provided below.

- The building mat foundation will involve an excavation of about 12 to 15 feet. The total depth of excavation will need to include the depth of overexcavation required per this section. For planning purposes, it is likely that the foundation subgrade will likely expose wet and soft subgrade soils, and potentially groundwater. Therefore, subgrade stabilization will be required including the possibility of dewatering to facilitate providing a uniform, firm and unyielding subgrade. Although the soils below the mat subgrade are likely to be saturated or nearly saturated, the proposed stabilization will also help mitigate for the presence of expansive soils. A method to stabilize the potentially wet and soft subgrade is presented in Table 5. Any dewatering system, if needed, should keep the groundwater at a depth of at least 3 feet below the bottom of the excavation (including overexcavation depth per Table 5). It is recommended that a section of the footprint be selected on a trial basis to test the effectiveness of the proposed method, before undertaking the stabilization over the whole area.

  If it becomes impractical to use the method recommended in Table 5, soil stabilization could also be provided by using chemical stabilization, i.e., adding cement and/or lime to a depth of at least 2 feet below the mat subgrade.

**Table 5**
**Method for Handling Unstable Materials at the Excavated Subgrade**

| | |
|---|---|
| **Step 1** | Overexcavate at least 2 feet below the mat foundation bottom |
| **Step 2** | Improve the soft subgrade by working in open-graded aggregate material (particle size larger than 1 inch) as much as possible/practical into the subgrade. |
| **Step 3** | Place woven geotextile, Mirafi RS580i or approved equivalent, over the exposed surface. |
| **Step 4** | Place and compact 1-foot of well-graded fill (e.g., AB, CMB) to specified compaction over the geotextile. |
| **Step 5** | Place woven geotextile, Mirafi RS580i or approved equivalent, over the exposed surface. |
| **Step 6** | Place and compact 1-foot of well-graded fill (e.g., AB, CMB) to specified compaction over the geotextile. |

- New Pavement areas should be overexcavated and recompacted to a depth of at least 2 feet below the proposed pavement subgrade elevation, or to uniform competent soils, whichever is deeper. To the extent practicable, the zone of overexcavation should extend a horizontal distance of at least 2 feet beyond the outside perimeter of the pavement.

- Flatwork areas including exterior flatwork slabs should be overexcavated and recompacted to a depth of at least 1 foot below the proposed subgrade elevation, or to uniform acceptable soils, whichever is deeper. To the extent practicable, the zone of overexcavation should extend a horizontal distance of at least 1 foot beyond the perimeter of the exterior slabs and other flatwork and 2 feet beyond the outside perimeter of the pavement.

- Entrance Driveway Pavement areas where existing pavement will be replaced to full depth, no overexcavation is required. The exposed subgrade should be scarified to a depth of 6 inches, moisture-conditioned to at least 125 percent of optimum moisture content and compacted to at least 95 percent of relative compaction per ASTM D1557.

- Disturbed soils in structural and non-structural areas will likely occur after demolition of existing site improvements or during overexcavation. The disturbed soils should be overexcavated and recompacted to the total depth of the disturbed material.

### 9.2.3. Fill Placement

Excavated on-site soils may be re-used as compacted fill, provided they are free of organics, deleterious materials, debris and particles over 3 inches in largest dimension. In addition, site soils at a depth of 10 feet or more will likely be well over the optimum moisture content and will require drying back, if used directly as compacted fill.

Fills should be placed in loose lifts not more than 8 inches in thickness. Fill placement associated with recompaction of overexcavated soils, fill placed to achieve finish grade or subgrade, or utility trench backfill should be moisture-conditioned to at least 125 percent of the optimum moisture content if the soils are fine-grained, or moisture-conditioned wet of optimum moisture content if the soils are coarse-grained, and compacted to at least 90 percent of relative compaction per ASTM D1557.

The upper one foot of soils below pavements and flatwork should be processed and compacted to at least 95 percent of relative compaction per ASTM D1557.

Soil materials (including general fill, structural backfill, or base course materials) imported to the site should be sampled, tested, and approved by the Geotechnical Engineer prior to arrival on-site. In general, any soils imported to the site for use as fill should be predominantly granular with fines content less than 15 percent and have an Expansion Index less than 20. Import materials should be moisture-conditioned to at least wet of the optimum moisture content and compacted to at least 90 percent of relative compaction per ASTM D1557. Additional recommendations for site grading are provided in the General Site Grading Recommendations section of this report.

### 9.3. Excavation Characteristics, Temporary Slopes and Trench Excavations

The near surface soils are readily excavatable with conventional earth-moving equipment. However, the soils are anticipated to become wet and soft at depths generally over 10 feet. Therefore, special handling with track mounted equipment (i.e., excavator) will likely be required. Depending on groundwater levels at the time construction, dewatering may also be necessary.

All trench excavations should be performed in accordance with CalOSHA regulations. The on-site soils above the groundwater level may be considered a Type B soil, as defined by the current CalOSHA soil classification.

Sides of temporary, unsurcharged excavations less than 15 feet deep should be sloped back at an inclination of 1(H):1(V) or flatter. For Type B soils benching could be used as long as the overall slope is kept at an inclination of 1(H):1(V) or flatter, however the vertical excavation bench height may not exceed 4 feet. Where space for sloped sides is not available, shoring will be necessary. For any configurations where the depth of the excavation exceeds 15 feet, a slope stability analysis should be performed by the Geotechnical Engineer.

Stockpiled (excavated) materials should be placed no closer than half of the excavation depth or 4 feet from the top of the trench, whichever is greater. A greater setback may be necessary when considering surcharge loads such as heavy vehicles, concrete trucks and cranes. Tetra Tech should be advised of such heavy vehicle loadings so that specific setback requirements can be established for the used equipment. Alternatively, a shoring system may be designed to allow reduction in the setback distance.

The Geotechnical Engineer should observe the excavation progress so that appropriate modifications to the excavation design may be recommended, if necessary, due to conditions differing from the design assumptions.

### 9.4. Temporary Shored Excavations

Excavation depths for the construction of the proposed building are anticipated to be at a depth between 12 and 15 feet. Review of the groundwater depth from the existing monitoring wells at the site is required before construction in order to determine whether or not a dewatering system is needed. The permits for a dewatering system, if needed, will require disposing of the extracted water in accordance with local regulations.

Cantilevered shoring systems are typically suitable and practicable for retained heights less than about 15 feet. Alternatively, restrained shoring system with internal bracing may be considered. Presented herein are design recommendations for both shoring systems. The shoring designer will need to take into account the presence of adjacent structures, utilities, conduits, and other underground structures and their impact on the installation and performance of the shoring system.

All components of the shoring system, including the penetration depth, should be designed by a specialist Civil Engineer registered in the State of California and should further satisfy requirements of Cal-OSHA. It is recommended that the final shoring design be reviewed by the Geotechnical Engineer.

### 9.4.1. Temporary Dewatering

Recommendations are provided herein for excavations up to a depth of 15 feet which is expected to facilitate construction of the foundation subgrade. Since the groundwater depth at the site is anticipated to vary significantly throughout the year as discussed in the “Groundwater” section of this report, the actual depth of groundwater should be determined prior to construction based on groundwater depth measurements at the existing monitoring wells at the site. Since the dewatering is an important design and construction consideration, groundwater depth readings from the onsite monitoring wells should be taken prior to finalizing the design documents and completing the bidding process to conclusively establish the need for dewatering. Dewatering system should be designed to lower the groundwater to a depth of at least 3 feet below the bottom of the excavation (including any overexcavation depth) throughout the footprint of the excavation. Given the fine-grained nature of the soils, and the recommended subgrade stabilization method, if sheet piles are used, the dewatering, if needed, may likely be performed from within the excavation.

### 9.4.2. Shoring Design

Either sheet pile or soldier pile and lagging systems may be utilized for the proposed construction. However, sheet piles may be more efficient if dewatering is required. Table 6 below summarizes the governing geotechnical design parameters and loading diagrams for a shoring system for both cantilever and internal bracing conditions. These geotechnical parameters were developed based on the following assumptions:

- the shored soil grade is level;
- there are no hydrostatic pressures behind the wall;
- the groundwater remains below the excavation bottom; and
- the shoring is temporary.

If any of these assumed conditions cannot be met due to field conditions or the contractor’s method of construction, this office should be consulted regarding potential revisions to our recommendations. It is also noted that to protect structures located at or adjacent to the property line, the shoring designer for this zero-lot condition may consider increasing the recommended cantilevered and restrained lateral pressures by multiplying them by a factor of 1.5 and 1.2 respectively.

Any surcharge (live or dead load) located within a 1(H):1(V) plane drawn up from the excavation bottom should be accounted for in the calculation of the lateral earth pressures. For cantilevered and restrained shoring systems, the lateral contribution of a uniform surcharge load may be calculated by multiplying the surcharge by a factor of 0.35 and 0.46, respectively. The shoring designer for a zero-lot condition may consider increasing the recommended factors for cantilevered and restrained shoring systems by multiplying them by a factor of 1.5 and 1.2 respectively. This constant lateral load, i.e., independent of depth, should be applied throughout the whole exposed height of the sheet pile or soldier pile wall. As a minimum, 3 feet of equivalent soil surcharge, i.e., 360 psf, is recommended to be included to account for nominal construction surcharge and traffic loads on the adjacent roads. In addition, due to potential partial wetting of soils during construction, it is recommended that an additional swelling pressure be included in the design of the temporary shoring as indicated in Table 6.

### Table 6
### Geotechnical Design Parameters
### Cantilevered and Restrained Temporary Shoring

(without hydrostatic pressure build-up behind the wall)

| | | |
|---|---|---|
| Excavation bottom depth (including overexcavation depth) | Up to 15 feet | |
| Subsurface materials | **Alluvial Soils**<br>Mostly firm to stiff lean clays and silts to depth of 30 feet<br>Groundwater depth during construction assumed at a depth of 3 feet below the bottom of the excavation | |
| **SHORING SYSTEM** | **Cantilevered shoring** (excavation depth less than about 12 to 15 feet) | **Restrained shoring** with internal bracing |
| Soil unit weight, $\gamma$ | 125 pcf | |
| Design friction angle, $\phi$ | $29^{o}$ | $0^{o}$ |
| Design cohesion, c | 0 psf | 1,000 psf |
| $K_a$ … coefficient of active lateral pressure | 0.35 | n/a |
| Stability number, Ns = $\frac{\gamma \cdot H}{c}$ | n/a | 1.6-1.8 |
| **LOADING DIAGRAM ON SHORING** | | |
| Equivalent fluid density, EFD | 43 pcf | n/a |
| Loading Diagram | Triangular distribution | Trapezoidal load distribution based on stability numbers (see Diagram 1 below) |
| Additional Lateral Pressure due to Potential Swelling | 15 psf (constant with depth) | |
| **ALLOWABLE PASSIVE PRESSURE BELOW EXCAVATION BOTTOM** | | |
| Design friction angle, $\phi$ | $29^{o}$ | |
| $K_p$ … coefficient of passive lateral pressure | 2.88 | |
| Arching capability * | 2.0 (for design of soldier pile systems) | |
| Equivalent fluid density (pcf EFD) ** (triangular distribution)<br>– includes Safety Factor of 1.5<br>– ignore resistance within upper 12 inches | Sheet pile system: 240 / 120 (above / below groundwater)<br>Soldier pile system (includes arching): 480 / 240 (above/below groundwater) | |

* Per Caltrans Trenching and Shoring Manual (2011)
** Valid without reduction for soldier pile spacing > 2.0 times the effective pile width

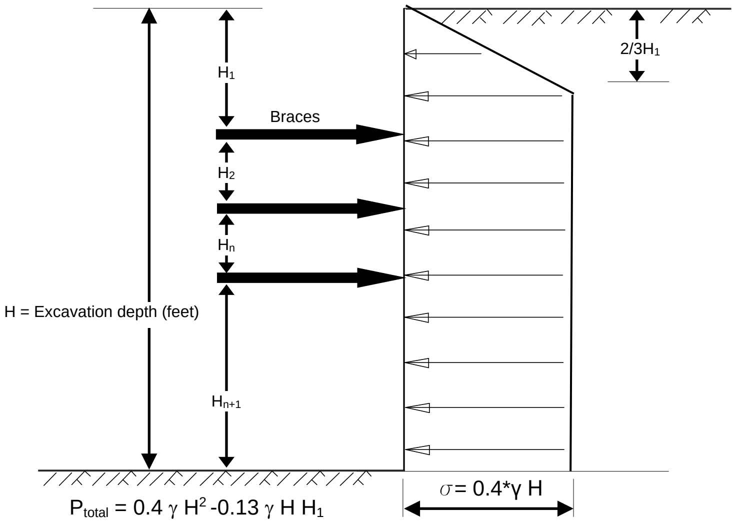

**Diagram 1. Lateral pressures loading for a restrained shoring with $N_s$ = <4**

The required penetration depth of the soldier or sheet pile wall below the excavation bottom may be calculated based on the estimated passive soil resistances provided in Table 6. Passive soil resistance should be ignored for the upper 12 inches below the excavation bottom to account for potential near-surface soil disturbance.

### 9.4.3. Deflection

It should be realized that some shroing deflection will likely occur. However, it is difficult to accurately predict the amount of deflection of a shored excavation because it depends not only on the shoring system design but also significantly on the quality of construction. The shoring system should be designed so that deflection at the top of the shored excavation is kept below 1 inch. If greater deflection occurs during construction, additional bracing or restraint may be necessary to minimize settlement of the nearby improvements. If it is desired to reduce the deflection of the shoring, a greater lateral earth pressure could be used for the shoring design.

### 9.4.4. Internal Bracing

Internal bracing can be provided with struts or rakers designed for lateral earth pressures provided in Table 6 and shown schematically in Diagram 1 for the restrained shoring system.

If used, raker bracing could be supported on temporary concrete footings. For design of temporary footings poured with the bearing surface normal to the rakers inclined at 45 to 60 degrees from the vertical, an allowable bearing capacity value of 1,700 psf may be used, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. To reduce the movement of the shoring wall, the rakers should be tightly wedged against the footings and shoring wall.

### 9.4.5. Construction Considerations

The shoring should be constructed utilizing a top-down method. Following the installation of the soldier or sheet piles, the soil is first partially excavated to allow for installation of the topmost row of internal bracing. For rakers, the staging will need to include installation of temporary rakers and their subsequent removal and replacement as the excavation advances. Following the installation of the topmost row of internal bracing, the excavation then proceeds, and each level of internal bracing should be installed as soon as practicable. The shoring designer should analyze each stage of internal bracing installation to ensure that the excavation has an adequate Factor of Safety.

For the soldier pile system, the lagging should be installed simultaneously as the excavation proceeds. To facilitate a tight connection between the lagging and the soils and to minimize settlement, any voids left behind the lagging should be filled with cement grout as the excavation advances. To continuously support the excavation, any unsupported height should not exceed 4 feet.

The anulus of the soldier pile beam borehole below the excavation bottom should be backfilled with concrete. Pea gravel may be used to backfill the hole from the excavation bottom to the top grade. If the soldier pile beams are to be retrieved after construction, they may be backfilled with a weaker cementitious slurry mix below the excavation bottom. If the contractor chooses to use a well-rounded uniform pea gravel material to fill the hole below the excavation bottom, a reduction of 33 percent should be applied to the passive soil resistance values provided in Table 6 to account for potential lateral yielding of the pea gravel backfill.

### 9.4.6. Shoring Performance Monitoring

Some means of monitoring the performance of the shoring system and nearby paved surfaces and structures is recommended. The monitoring should consist of periodic visual inspections and lateral and vertical surveying of the tops of the soldier / sheet piles and strategic survey points. It is recommended that a survey be performed before construction begins and then, as the excavation proceeds, the monitoring should be performed daily or whenever excavation activities are taking place. In addition, the Contractor should inspect the shoring daily and actively search for presence of cracks or excessive movements and report immediately to the shoring designer and the Geotechnical Engineer. This office can provide further recommendations of the monitoring when the design of the shoring system is finalized.

## 9.5. Seismic Design Parameters

Based on blowcount data obtained from the current field investigation and known geologic conditions, the site is classified as Site Class E, i.e., soft soil, in accordance with ASCE 7-16, Section 20.3.2. Given the values of the spectral acceleration parameters $S_s$ and $S_1$ and the Site Class E, a site-specific ground motion hazard analysis was required per Section 11.4.8 of ASCE 7-16.

The ground motion hazard analysis was performed in accordance with ASCE 7-16, Section 21.2 and required the evaluation of deterministic ($MCE_R$) ground motions and probabilistic ($MCE_R$) ground motions.

For the deterministic hazard analysis (DSHA) the closest 4 faults to the site listed in Table 2 were considered, assuming an average shear wave velocity in the top 30 meters ($Vs_{30}$) of 150 meters per second. The DSHA acceleration response spectrum (ARS) was determined using the Next Generation Attenuation (NGA) – West2 models. The NGA-West2 models were developed as part of a multidisciplinary research program coordinated by the Lifelines Program of the Pacific Earthquake Engineering Research Center (PEER), in partnership with the U.S. Geologic Survey (USGS) and the Southern California Earthquake Center (SCEC). Using the Excel spreadsheet developed by Seyhan (2015), the 84th-percentile median spectral acceleration values ($Sa_{RotD50}$) were calculated based on the equally weighted models by:

- Abrahamson and Silva (2014);
- Boore et al. (2014);
- Campbell and Bozorgnia (2014); and
- Chiou and Youngs (2014).

The 84th-percentile median spectral acceleration values ($Sa_{RotD50}$) were subsequently converted to risk-targeted maximum rotated direction ($Sa_{RotD100}$) values using the procedure suggested by Shahi and Baker (2014) and adjusted further by the risk coefficients provided in the web tool https://www.seaoc.org/page/seismicdesignmaptool.

The probabilistic seismic hazard analyses (PSHA) were performed for an earthquake event with a return period of 2,475 years, utilizing the USGS Unified Hazard tool (https://earthquake.usgs.gov/hazards/interactive/) for the dynamic: conterminous US 2014 update (v4.2.0) using a return period of 2,475 years. For the subject site the website utilizes an assumed average shear wave velocity in the top 30 meters ($Vs_{30}$) of 180 meters per second, which corresponds to a Site Class D/E (soft soil).

The seismic spectral acceleration parameters to be used in the design for buildings and structures subject to seismic shaking are provided in Table 7 below. The design spectrum and a summary of the site coefficients are provided in Appendix C.

## Table 7
## 2019 CBC and ASCE 7-16 Seismic Design Parameters

Referenced Site Latitude and Longitude: 37.346191°, -121.937395°

| | | |
|---|---|---|
| **Site Class**<br>Table 20.3-1 ASCE 7-16 | | E |
| **Coefficients for the Maximum Considered Earthquake, $MCE_R$, for Site Class B** | Short Period (0.2 seconds), $S_S$ | 1.5* |
| | 1 Second Period, $S_1$ | 0.6* |
| **Coefficients for the Maximum Considered Earthquake, $MCE_R$ (Site Modified)** | Short Period (0.2 seconds), $S_{MS}$ | 1.44** |
| | 1 Second Period, $S_{M1}$ | 2.76** |
| **Coefficients for the Design Earthquake** | Short Period (0.2 seconds), $S_{DS}$ | 0.96** |
| | 1 Second Period, $S_{D1}$ | 1.84** |
| **Design PGA (risk-targeted maximum rotated direction)** | | 0.35g** |
| **Site Modified Peak Ground Acceleration $PGA_M$ (geometric mean)** | | 0.605g* |

* Values obtained from Structural Engineers Association of California (SEAOC) and the Office of Statewide Health Planning and Development (OSHPD) website application, https://www.seaoc.org/page/seismicdesignmaptool **based on ASCE7-16 and 2018 International Building Code**.

** These parameters were obtained from a site-specific ground motion hazard analysis as required per ASCE 7-16, Section 21.2.

### 9.6. Mat Foundation

The proposed residential / retail building may be supported on a conventional mat foundation. An advantage of a mat foundation for this project is the reduction of the applied pressures on the relatively soft subgrade and the suitability for ease of installation of a reliable waterproofing system.

### 9.6.1. Design Parameters for the Mat Foundation

Recommendations for the design of the mat foundation for the proposed building are provided in Table 8 below. The mat foundation should be designed and reinforced in accordance with the recommendations of the Structural Engineer and should conform to the requirements of the 2019 CBC.

The recommendations provided herein do not account for eccentric loads or localized areas where vertical stress redistributions may be encountered. Once the foundation configuration and loads are finalized, this office should be contacted to evaluate this specific foundation configuration and loading condition prior to finalization of the mat foundation design.

### Table 8
### Geotechnical Design Parameters
### Mat Foundation

| | |
|---|---|
| **Embedment depth** | • Between 12 and 15 feet |
| **Dimensions (feet)** | • Triangularly shaped with side lengths of approximately 250 feet by 137 feet by 283 feet (max dimensions) |
| **Allowable Bearing Pressure** | • Average allowable bearing pressure 2,000 psf<br>• The allowable bearing value may be increased by one-third for transient live loads from wind and seismic loading. |
| **Estimated Settlement** | • Approximate 1.5 inches of static settlement.<br>• Approximate 1.5 inches of dynamic settlement<br>• Combined (static plus dynamic) differential settlement of approximately 1.5 inches over a distance of 30 feet. |
| **Modulus of Subgrade Reaction** | • For design of the mat foundations, a reference modulus of subgrade reaction $k_1$ of 35 pci derived for a square bearing plate with 1-foot x 1-foot dimensions may be used. For the on-site clayey soils, the modulus of subgrade reaction $k$ (in pci) for the design of a concrete of a given dimension can be calculated as:<br>$$k = k_1 \frac{1 + 0.5 * \frac{B}{L}}{1.5 * B}$$<br>Where $B$ and $L$ are the governing width and the length of the element in feet, but no more than 14 times the thickness of the mat foundation. |
| **Swelling Pressure**<br>(No Factor of Safety) | • Foundations should be designed to withstand an upward swelling pressure of 15 psf |
| **Allowable Adhesion at the base**<br>(incorporates Factor of Safety of 1.5) | • 500 psf<br>• Adhesion to be multiplied by contact area per 2019 CBC Section 1806.3.2. |
| **Allowable Lateral Passive Resistance**<br>(incorporates Factor of Safety of 2) | • 240 /120 pcf (above/below groundwater) (EFD, equivalent fluid density)<br>• The passive resistance derived of the upper 12 inches should be neglected |
| **Allowable Combined Lateral Resistance** | • Total allowable resistance to lateral loads can be calculated by combining lateral resistance due to adhesion at the base and lateral passive resistance.<br>• Passive resistance values may be increased by one-third when considering transient wind or seismic loading |
| **Uplift Capacity** | • The weight of the soil that contributes to the uplift capacity can be estimated as a zone defined by an angle of 30 degrees from the vertical projected from the top edge of the mat to the adjacent grade.<br>• A total unit weight of 125 pcf may be used for the soil.<br>• The shallowest depth of embedment from the adjacent grade shall be used in the estimations. |

### 9.6.2. Groundwater Considerations

Building basement slabs will extend well below the design groundwater level and will be subject to hydrostatic uplift pressure. For the analyses of hydrostatic pressure and building uplift pressure, we recommend that a groundwater level at a depth of about 4 feet below the existing grade be assumed. Basement walls and mat foundations should incorporate a complete waterproofing system. Waterproofing system should be designed by an Engineer with extensive experience in waterproofing for basement walls for residential or commercial use.

## 9.7. Shallow Foundations Adjacent to Utility Trenches

The bottom of any trenches that are required for any buried utilities should be kept outside a zone defined by a 1(H):1(V) plane projected downward from the outside bottom edge of any existing or proposed foundation. Backfill materials and procedures shall conform to the recommendations provided in the “Site Preparation” and “General Site Grading Recommendations” sections of this report. If any utilities need to be placed within the zone of influence, the utility conduit (pipes, cables) should be designed to account for the increased surcharge from the foundation pressures and to withstand potential differential settlement between the surcharged and unsurcharged segments of the pipe. Generally, the utility conduits within the impacted zone should be protected by concrete encasement or utilidors.

For utility conduits that cross underneath foundations the piping and encasement should be designed to withstand differential settlements of up to 1 inch over a distance equal to half of the depth of the pipe crown below the bottom of the foundation element. Tetra Tech should be contacted to review any specific utility interaction configurations and their proposed mitigation.

## 9.8. Exterior Concrete Slabs on Grade

Exterior slabs should be placed on subgrade prepared in accordance with the recommendations provided in the “Site Preparation” section of this report. A Structural Engineer or an Engineer specialized in concrete design should be consulted if cracking of the exterior slabs is to be minimized. As a minimum for exterior walkways, it is recommended that narrow strip concrete slabs, such as sidewalks, be reinforced with at least No.4 reinforcing bars placed longitudinally at 18 inches on center. Wide exterior slabs should be reinforced with at least No.4 reinforcing bars placed 18 inches on center, each way. Reinforcement should extend through the control joints to reduce the potential for differential movement.

Control joints should be provided in concrete slabs-on-grade as recommended by American Concrete Institute (ACI PRC-224.3-95) guidelines and at a maximum spacing (in feet) of 2 to 3 times of the slab thickness (in inches), but generally no more than 10 feet. All joints should form approximately square patterns to reduce potential for randomly oriented shrinkage cracks. The control joints should be tooled at the time of the pour or sawcut to ¼ of slab depth within 6 to 8 hours of concrete placement. All joints in flatwork should be sealed to prevent moisture, vermin, or foreign material intrusion. Precautions should be taken to prevent curling of slabs.

## 9.9. Basement Walls

Buried building walls for the subterranean parking lot level will act as basement walls which will support lateral earth pressures.

### 9.9.1. Lateral Loading

The 2019 CBC requires that basement walls be designed for at-rest earth pressures for static conditions. The design values presented in Table 9 below were developed based on a level backfill condition. In addition, it is required that a drainage system will be installed behind the wall so that external water pressure will not develop. If a drainage system is not installed, hydrostatic pressures will need to be incorporated into the design. A design groundwater depth of 4 feet may be used to calculate the hydrostatic pressures acting on the basement walls.

Based on the Section 1803.5.12 of the 2019 CBC the design of retaining structures higher than 6 feet, as measured from the bottom of the footing, needs to consider seismically induced lateral earth pressures. According to the 2019 CBC the seismically induced lateral earth pressures should be determined using the design earthquake ground motions. Based on the seismic design parameters provided in Table 7 of this report, the Design Peak Ground Acceleration (PGA) at the site is approximately 0.35g. The seismically induced earth pressure increments were estimated using the method recommended by Agusti and Sitar (2013).

The combined static and seismic lateral passive earth pressures during the design seismic event were computed as recommended by Taciroglu (2013). The calculated passive earth pressures are included in Table 9.

Basement walls should be appropriately waterproofed in accordance with 2019 CBC Section 1805.3. The on-site soils are predominately fine-grained and have a potential for expansion. Therefore, the on-site soils should not be re-used for backfill within 2 feet behind basement walls. The backfill within at least 2 feet immediately behind the basement walls should be imported materials with an Expansion Index of less than 20 and fines content (passing #200 sieve) of less than 15 percent. Where bare ground is present behind the top of the wall, the backfill should be capped with a concrete swale or with at least 12 inches of relatively impervious clayey material and sloped to prevent ponding of water. The granular backfill should be moisture-conditioned to at least 110 percent of optimum moisture content and compacted in loose lifts not more than 8 inches to at least 90 percent of the maximum dry density as evaluated by the latest version of ASTM D1557.

## Table 9
## Geotechnical Design Parameters for Basement Walls

| | | |
|---|---|---|
| **At rest Pressure for Non-yielding Walls, i.e., Basement Walls** | | |
| Static pressure (psf) | Level ground behind wall<br>(above groundwater) | $62z + 0.52Q$ |
| | Level ground behind wall<br>(below groundwater at depth $z > z_w$) | $62z_w + 95(z - z_w) + 0.52Q$ |
| Seismic pressure increment (psf) | Level ground behind wall | $30z$ |
| **Passive Pressure Resistance** | | |
| Static resistance (psf)<br>(incorporates Factor of Safety of 2) | Level ground behind wall<br>(above groundwater) | $180z_1$ |
| Seismic ultimate resistance (psf)<br>(No Factor of Safety included) | | $275z_1$ |
| Static resistance (psf)<br>(incorporates Factor of Safety of 2) | Level ground behind wall<br>(below groundwater at depth $z_1 > z_w$) | $180z_w + 90 (z_1 – z_w)$ |
| Seismic ultimate resistance (psf)<br>(No Factor of Safety included) | | $275z_w + 137 (z_1 – z_w)$ |

Notes:

- Lateral pressures due to seismic loading are based on a PGA=0.35g.
- The appropriate total seismic force (at rest plus seismic increment for non-yielding walls) should be calculated be assuming a downward increasing tringle equivalent fluid pressure distribution. The resulting force should be assumed to act at 1/3 of the height of the wall above the heel of the wall.
- Pressures are based on soil with $\phi = 29^o$, $c = 0$ psf, $\gamma_t = 125$ pcf (above and below groundwater)

Legend:

$z$ … Depth (ft) below the grade behind the wall – depth measured from the ground surface to the depth where the soil lateral pressure is being evaluated.
$z_1$ … Depth (ft) below the grade where passive conditions apply, i.e., usually in front of the wall – depth measured from the ground surface to the depth where the soil lateral pressure is being evaluated.
$z_w$ … Depth to groundwater (ft) – depth measured from the ground surface to the groundwater (a constant), $z_w = 4$ feet.
Q … Uniform live surcharge (psf) within a 1(H):1(V) plane drawn upward from the heel of the wall footing.

If the basement walls are built in contact with the native expansive soils, the walls should be designed for an additional uniform lateral swelling pressure of 15 psf acting over the full height of the wall.

### 9.10. Uplift of Buried Structures

Structures founded at depths greater than 4 feet should be designed to resist uplift forces due to buoyancy exerted by groundwater below such depth to prevent buried structures including utilities/pipelines from floating or shifting upward. The designer must consider all the downward and upward forces on the structures and design for the worst-case scenario.

## 9.11. Portland Cement Concrete (PCC) Pavement Design

### 9.11.1. Pavement Structural Section

Pavements are anticipated for the entrance driveway to the site. The pavement subgrade should be prepared as recommended in Section 9.3 of this report. The recommended PCC pavement sections are presented in Table 10 below. Tetra Tech does not practice in the field of structural PCC pavement structural design and applies pavement section design method per ACI 330-08 based on the subgrade soil materials and the geotechnical conditions. Design and detailing of the reinforcement, jointing, doweling, mix design, concrete placement, curing, and specifications should be performed in accordance with ACI recommendations.

The PCC pavement sections were designed for a 20-year design life and assumed average daily truck traffic (ADTT) of 25. Contraction, construction, and isolation joints should be placed per ACI recommendations. The design assumes that 8 inches of Aggregate Base Materials will be placed on top of the compacted subgrade. The aggregate base materials should conform to the Specifications for Public Works Construction (Green Book) Section 200-2. The aggregate base course should be compacted to 95 percent or more of the relative density, as evaluated by the latest version of ASTM D1557.

The design of the pavement assumes that the Portland cement concrete will have a 28-day flexural strength (modulus of rupture determined by the third-point method) of at least 550 psi (equivalent compressive strength of about 4,000 psi) or 650 psi (equivalent compressive strength of about 5,000 psi). A modulus of subgrade reaction (*k* value) of 75 pci was assumed for the top of the compacted subgrade soils.

**Table 10**
**Portland Cement Concrete Pavement Sections**

| | | | | | |
|---|---|---|---|---|---|
| **Subgrade Soil** | | Lean Clay | | | |
| **Modulus of Subgrade Reaction, k** | | 75 pci | | | |
| **Thickness of Aggregate Base Material** | | 8 inches | | | |
| **PCC Pavement 28-day concrete compressive strength** | | 4,000 psi | | 5,000 psi | |
| **ADTT** | **Pavement Traffic Description** | **PCC Thickness (inches)** | **Max Joint Spacing (feet)** | **PCC Thickness (inches)** | **Max Joint Spacing (feet)** |
| 25 | Shopping Center Entrance and Service Lanes | 6 | 11 | 5 | 10 |

### 9.11.2. Construction Considerations

Paved areas should be properly sloped, and surface drainage facilities should be established to reduce water infiltration into the pavement subgrade. Curbing located adjacent to paved areas should be founded in the soil subgrade in order to provide a cutoff to reduce water infiltration into the base course.

### 9.12. Feasibility of Onsite Stormwater Infiltration

Given the presence of native fine-grained soils (i.e., lean clay) from the ground surface to a depth of at least 30 feet, and the presence of shallow groundwater (e.g., historical high groundwater at a depth of 4 feet), The onsite stormwater infiltration is not considered feasible.

### 9.13. Soil Corrosion

The corrosion potential of the on-site materials to buried concrete and steel was evaluated based on laboratory testing. Table 11 below presents the results of the corrosivity testing.

**Table 11**
**Corrosivity Test Results**

| **Boring ID** | **Sample ID** | **Depth (feet)** | **pH** CTM 643 | **Minimum Resistivity (ohm-cm)** CTM 643 | **Soluble Sulfate Content (ppm / %)** CTM 417 | **Soluble Chloride Content (ppm / %)** CTM 422 |
|---|---|---|---|---|---|---|
| Tt-1 | SK-1 | 0-5 | 9.1 | 3,458 | 22/0.0022 | 20/0.0020 |
| Tt-2 | SPT-3 | 10-11.5 | 7.9 | 1,010 | 510/0.0510 | 25/0.0025 |

Per 2019 CBC, Section 1904.1, concrete subject to exposure to sulfates shall comply with the requirements set forth in ACI 318, Section 19.3. Based on the measured water-soluble sulfate results the exposure of buried concrete to sulfate attack should be considered "not a concern", i.e., exposure class S0 per ACI 318, Table 19.3.1.1. Consequently, injurious sulfate attack is not a concern for concrete with a minimum 28-day compressive strength of 2,500 psi.

Per 2019 CBC, Section 1904.1, concrete reinforcement should be protected from corrosion and exposure to chlorides in accordance with ACI 318, Section 19.3.

The evaluation of potential for corrosion of buried metals was based on the minimum resistivity per NACE (1984) and our experience with similar soils. The on-site soils are anticipated to have a "moderately corrosive" potential to buried ferrous metals. A corrosion specialist should be consulted regarding suitable types of piping and necessary protection for underground metal conduits. The corrosion potential of the on-site soils should be verified during construction for each encountered soil type. Imported fill materials should be tested prior to placement to confirm that their corrosion potential is not more severe than the one assumed for the project.

### 9.14. Drainage Control

The intent of this section is to provide general information regarding the control of surface water. The control of surface water is essential to the satisfactory performance of the building construction and site improvements. Surface water should be controlled so that conditions of uniform moisture are maintained beneath and adjacent to the structure, even during periods of heavy rainfall. The following recommendations should be considered as minimal.

- Ponding and areas of low flow gradients should be avoided.

- Paved surfaces within 10 feet from the building foundation should be provided with a gradient of at least 2 percent sloping away from improvements.

- Bare soil, e.g., planters, within 10 feet of the structure should be sloped away from the improvement at a gradient of 5 percent.

- Positive drainage devices, such as graded swales, paved ditches, and/or catch basins should be employed to accumulate and convey water to appropriate discharge points.

- Concrete walks and flatwork should not obstruct the free flow of surface water.

- Area drains should be recessed below grade to allow free flow of water into the basin.

- Enclosed raised planters should be sealed at the bottom and provided with an ample flow gradient to a drainage device. Recessed planters and landscaped areas should be provided with area inlet and subsurface drainpipes.

- To the extent practicable, planters should not be located adjacent to the structure. If planters are to be located adjacent to the structure, the planters should be positively sealed, should incorporate a subdrain, and should be provided with free discharge capacity to a drainage device.

- Planting areas at grade should be provided with positive drainage. Wherever possible, the grade of exposed soil areas should be established above adjacent paved grades. Drainage devices and curbing should be provided to prevent runoff from adjacent pavement or walks into planted areas.

- Gutter and downspout systems should be provided to capture discharge from roof areas. The accumulated roof water should be conveyed to an off-site disposal area by a pipe or concrete swale system.

- Landscape watering should be performed judiciously to preclude either soaking or desiccation of soils. The watering should be such that it just sustains plant growth without excessive infiltration. Sprinkler systems should be checked periodically to detect leakage and irrigation efforts should be reduced or halted during the rainy season.

## 10. GENERAL SITE GRADING RECOMMENDATIONS

The intent of this section is to provide general information regarding the site grading. Site grading operations should conform with applicable local building and safety codes and to the rules and regulations of those governmental agencies having jurisdiction over the subject construction.

The grading contractor is responsible for notifying governmental agencies, as required, the Geotechnical Engineer at the start of site cleanup, at the initiation of grading, and any time that grading operations are resumed after an interruption. Each step of the grading should be accepted in a specific area by the Geotechnical Engineer, and where required, should be approved by the applicable governmental agencies prior to proceeding with subsequent work.

The following site grading recommendations should be regarded as minimal. The site grading recommendations should be incorporated into the project plans and specifications.

1. Prior to grading, existing vegetation, trash, surface structures and debris should be removed and disposed off-site at a legal dumpsite. Any existing utility lines, or other subsurface structures which are not to be utilized, should be removed, destroyed, or abandoned in compliance with current governmental regulations.

2. Subsequent to cleanup operations, and prior to initial grading, a reasonable search should be made for subsurface obstructions and/or possible loose fill or detrimental soil types. This search should be conducted by the contractor, with advice from and under the observation of the Geotechnical Engineer.

3. Prior to the placement of fill or foundations within the building area, the site should be prepared in accordance with the recommendations presented in the section "Site Preparation" of this report. All undocumented fill or disturbed soils within the building areas should be removed and processed as recommended by the Geotechnical Engineer.

4. The exposed subgrade and/or excavation bottom should be observed and approved by the Geotechnical Engineer for conformance with the intent of the recommendations presented in this report and prior to any further processing or fill placement. <u>It should be understood that the actual encountered conditions may warrant excavation and/or subgrade preparation beyond the extent recommended and/or anticipated in this report.</u>

5. On-site inorganic granular soils that are free of debris or contamination are considered suitable for placement as compacted fill. Any rock or other soil fragments greater than 3 inches in size should not be placed within 5 feet of the foundation subgrade.

6. Any imported fill material required for backfill or grading should be tested and approved prior to delivery to the site.

7. Visual observations and field tests should be performed during grading by a Geotechnical Engineer. This is necessary to assist the contractor in obtaining the proper moisture content and required degree of compaction. Wherever, in the opinion of the Geotechnical Engineer,

an unsatisfactory condition is being created in any area, whether by cutting or filling, the work should not proceed in that area until the condition has been corrected.

## 11. DESIGN REVIEW AND CONSTRUCTION MONITORING

Geotechnical review of plans and specifications and participation during construction are an integral part of the geotechnical design practice. The following sections present our recommendations relative to the review of construction documents and the monitoring of construction activities.

### 11.1. Plans and Specifications

Upon completion, the civil, structural, and shoring design plans and specifications should be reviewed and approved by Tetra Tech prior to submittal for issuance of grading and construction permit and prior to bidding of construction tasks as the geotechnical recommendations may need to be re-evaluated based on the actual design configuration and loads. This review is necessary to evaluate whether the recommendations contained in this report have been incorporated into the project plans and specifications as intended.

### 11.2. Construction Monitoring

The objective of the construction quality assurance (CQA) is to assist in the construction of the soils and soils-structure interaction components of the project. Continuous observation of site excavation, processing and assessment of fill materials, fill placement, ground improvement installation, and other site grading operations by a representative of the Geotechnical Engineer should be implemented during construction to allow for evaluation of the geotechnical-related conditions as they are encountered. This process provides the Geotechnical Engineer with the opportunity to recommend appropriate revisions as needed.

### 11.3. Grading Observations

The Geotechnical Engineer should observe the excavation, subgrade preparation for building foundations, concrete slabs, pipelines, and pavements, and fill placement so that appropriate modifications to the design, extent, or procedure may be provided, as necessary, should conditions encountered during grading differ from the design assumptions.

### 11.4. Foundation Subgrade Observations

The Geotechnical Engineer should observe and evaluate the presence of satisfactory materials at the mat foundation subgrade. The foundations excavations should be observed by the Geotechnical Engineer to verify if soft or loose soils or other unsatisfactory materials are encountered, and whether or not such materials should be removed and replaced with compacted fill prior to pouring the mat foundation.

### 11.5. Pavement Construction Observations

Preparation of the pavement subgrade and the placement of base course and pavement sections should be observed by the Geotechnical Engineer. Careful observation is recommended to evaluate that the pavement subgrade is uniformly compacted, and the recommended pavement and base course thicknesses are achieved.

### 11.6. Construction Quality Assurance Reporting

The following list is intended to provide basic minimum guidelines for the reporting during the excavation and backfilling operations:

- A Daily Field Report should be generated each time a representative of the Geotechnical Engineer is performing QA work at the site.

- The Daily Field Reports should contain, at a minimum, a detailed description of the field activities, utilized equipment, areas of work, date, time, weather, and locations and results of all observations and performed tests.

- Provisions should be made for vertical and horizontal control for recording observations and test locations.

- A complete set of Daily Field Reports should be submitted as a part of formal final reporting.

## 12. LIMITATIONS

The recommendations and opinions expressed in this report are based on Tetra Tech's review of background documents and on information obtained from field explorations and associated laboratory testing. It should be noted that this study did not evaluate the possible presence of hazardous materials on any portion of the site.

Due to the limited nature of the field explorations, conditions not observed and described in this report may be present on the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during grading operations, for example, the extent of unsuitable soil and the associated additional effort required to mitigate them.

Site conditions, including groundwater level, can change with time as a result of natural processes or the activities of man at the subject site or at nearby sites. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result of 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 Tetra Tech has no control. Therefore, this report should be reviewed and recertified by Tetra Tech if it were to be used for a project design commencing more than one year after the date of issuance of this report.

Tetra Tech's recommendations for this site are, to a high degree, dependent upon appropriate quality control of subgrade preparation, fill placement, and foundation construction. Accordingly, the recommendations are made contingent upon the opportunity for Tetra Tech to observe grading operations and foundation excavations for the proposed construction. If parties other than Tetra Tech are engaged to provide such services, such parties are automatically assuming complete responsibility as the Geotechnical Engineer of Record for the project and are deemed concurring with the recommendations in this report or are obligated to provide alternative recommendations.

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. Tetra Tech should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. Reliance by others on the data presented herein or for purposes other than those stated in the text is authorized only if so permitted in writing by Tetra Tech. It should be understood that such an authorization may incur additional expenses and charges.

Tetra Tech has endeavored to perform its evaluation using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area in similar soil conditions. No other warranty, either expressed or implied, is made as to the conclusions and recommendations contained in this report.

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# Figures

FIGURE 1

**TETRA TECH**

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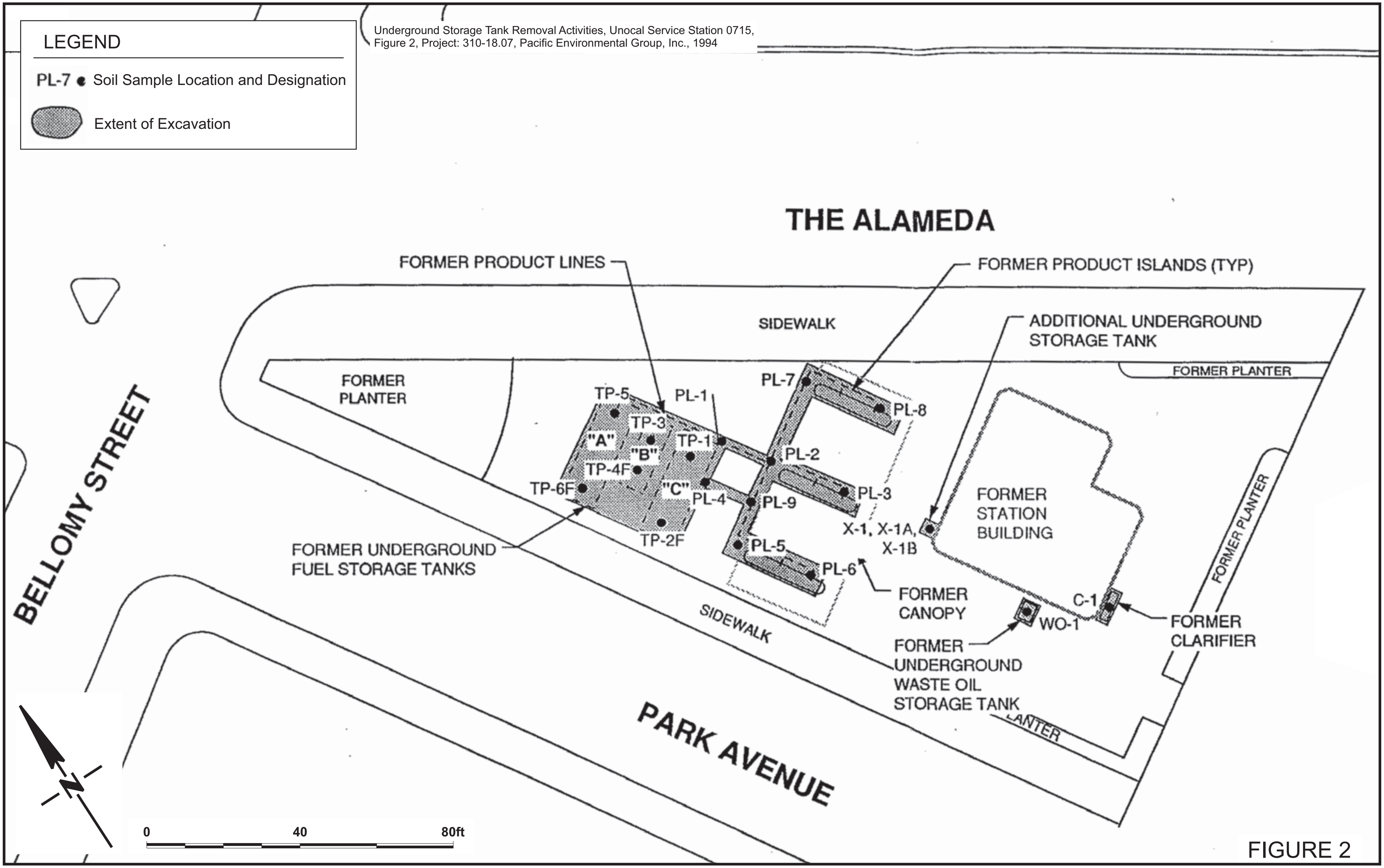
2655 THE ALAMEDA PROPOSED DEVELOPMENT - SANTA CLARA, CA

## SITE LOCATION MAP

| | |
|---|---|
| JOB NO. | TET 22-235E |
| DATE | MAY 2022 |
| DRAWN BY | SCM |
| CHECKED BY | MS |



Underground Storage Tank Removal Activities, Unocal Service Station 0715,
Figure 2, Project: 310-18.07, Pacific Environmental Group, Inc., 1994

FIGURE 2

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FORMER EXCAVATION LIMIT MAP

JOB NO. TET 21-235E
DATE MAY 2022
DRAWN BY SCM
CHECKED BY MS

TABULATION

PROJECT DESCRIPTION
4-STORY BUILDING, CONSISTING OF A 23 STALLS BELOW GRADE PARKING GARAGE. 1,500 SF STREET LEVEL RETAIL WITH 3 RESIDENTIAL UNITS AT GRADE. LEVELS 2-4 CONSISTING OF 36 RESIDENTIAL UNITS. TOTAL BUILDING HEIGHT OF 52'-55' INCLUDING ROOF PARAPET.

| | | |
|---|---|---|
| SITE AREA | | 17,781 SF(0.4AC) |
| BUILDING | | 48,148 SF |
| UNITS | | 39 |
| FAR | | 98 UNITS/AC |
| PARKING | | 33 STALLS |
| PARKING GARAGE | | 23 |
| SITE | | 10 |
| RATIO | | |
| RETAIL | 3 STALLS | 2/1000 |
| RESIDENTIAL | 30 STALLS | 0.75/UNIT |
| SETBACKS | | |
| THE ALAMEDA | | 4 FEET |
| PARK AVENUE | | 4 FEET |

## 2655 THE ALAMEDA

SANTA CLARA, CALIFORNIA
KAPITAL PARTNERS LLC
KENNETH RODRIGUES & PARTNERS, INC.

36.652
DECEMBER, 2021

STREET LEVEL1
9,487 SF

FIGURE 3A

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2655 THE ALAMEDA PROPOSED DEVELOPMENT - SANTA CLARA, CA

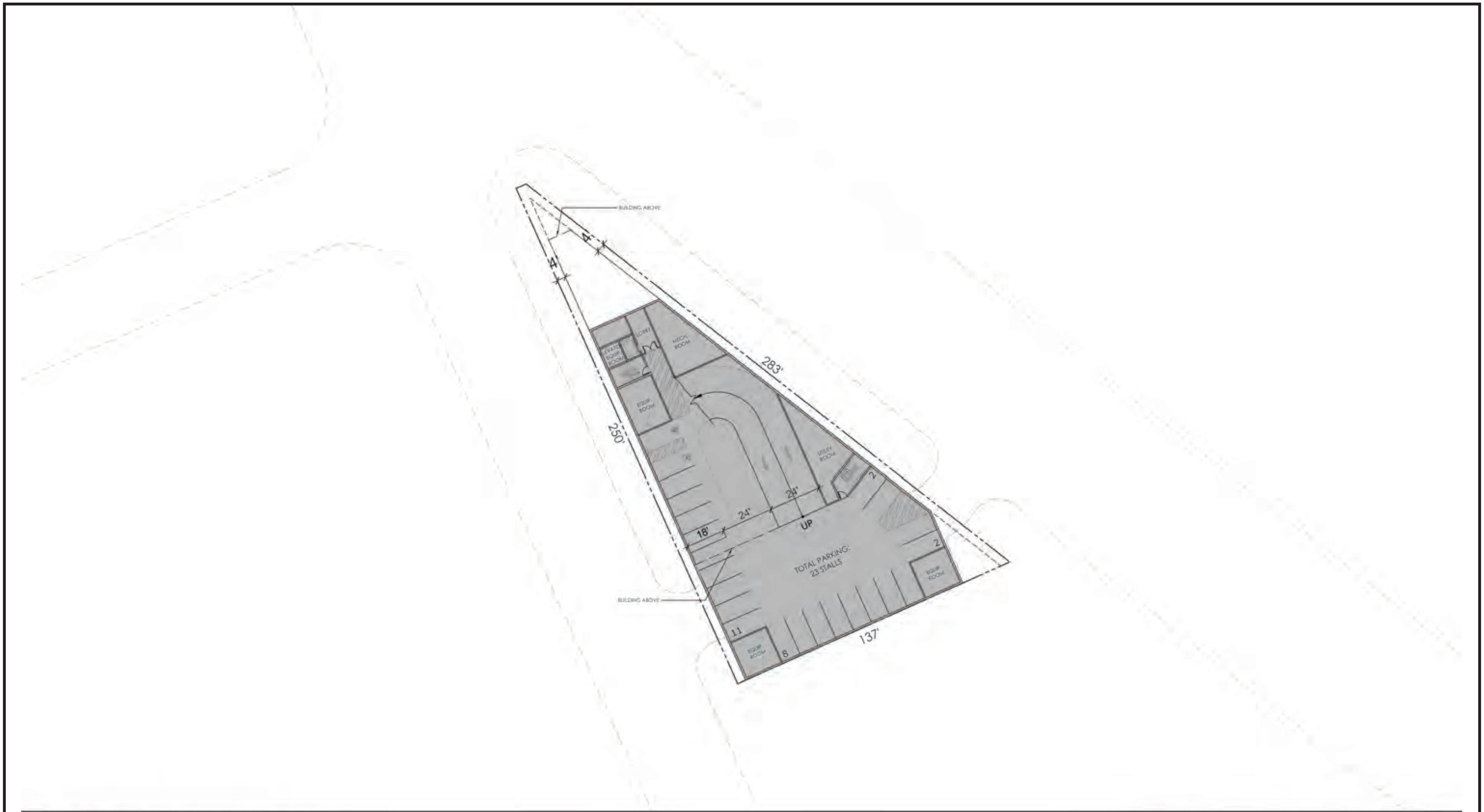
PRELIMINARY DEVELOPMENT PLAN

JOB NO.
TET 21-235E

DATE
MAY 2022

DRAWN BY
SCM

CHECKED BY
MS



# 2655 THE ALAMEDA

SANTA CLARA, CALIFORNIA
KAPITAL PARTNERS LLC
KENNETH RODRIGUES & PARTNERS, INC.

36.652
DECEMBER, 2021

BASEMENT PARKING
LEVEL B1
14,143 SF

FIGURE 3B

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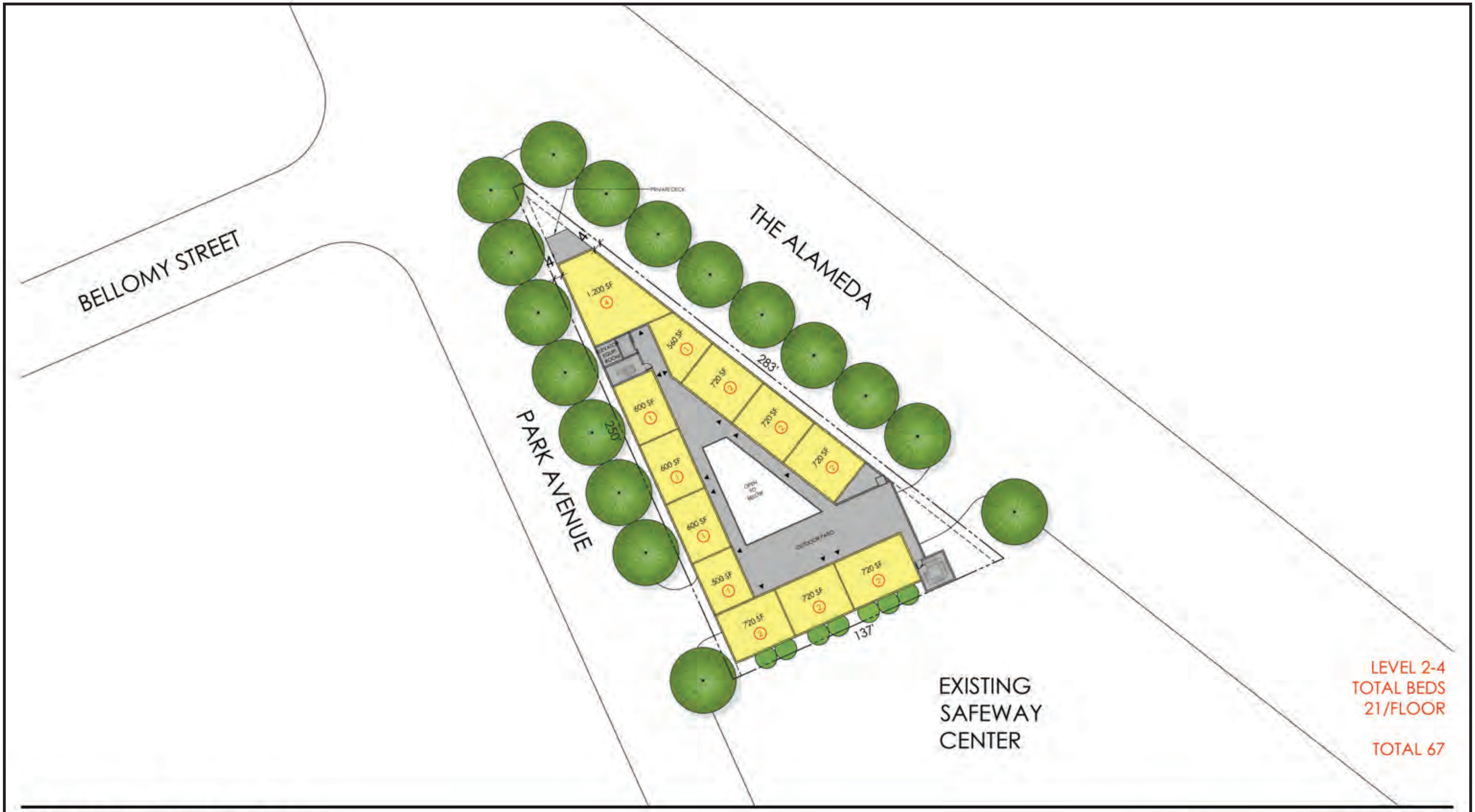
2655 THE ALAMEDA PROPOSED DEVELOPMENT - SANTA CLARA, CA

PRELIMINARY DEVELOPMENT PLAN

JOB NO. TET 21-235E
DATE MAY 2022
DRAWN BY SCM
CHECKED BY MS



LEVEL 2-4
TOTAL BEDS
21/FLOOR

TOTAL 67

# 2655 THE ALAMEDA

SANTA CLARA, CALIFORNIA
KAPITAL PARTNERS LLC
KENNETH RODRIGUES & PARTNERS, INC.

36.652
DECEMBER, 2021

N

0 20 40'

FLOOR PLAN
LEVEL 2-4
12,887 SF

FIGURE 3C

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2655 THE ALAMEDA PROPOSED DEVELOPMENT - SANTA CLARA, CA

PRELIMINARY DEVELOPMENT PLAN

JOB NO. TET 21-235E
DATE MAY 2022
DRAWN BY SCM
CHECKED BY MS

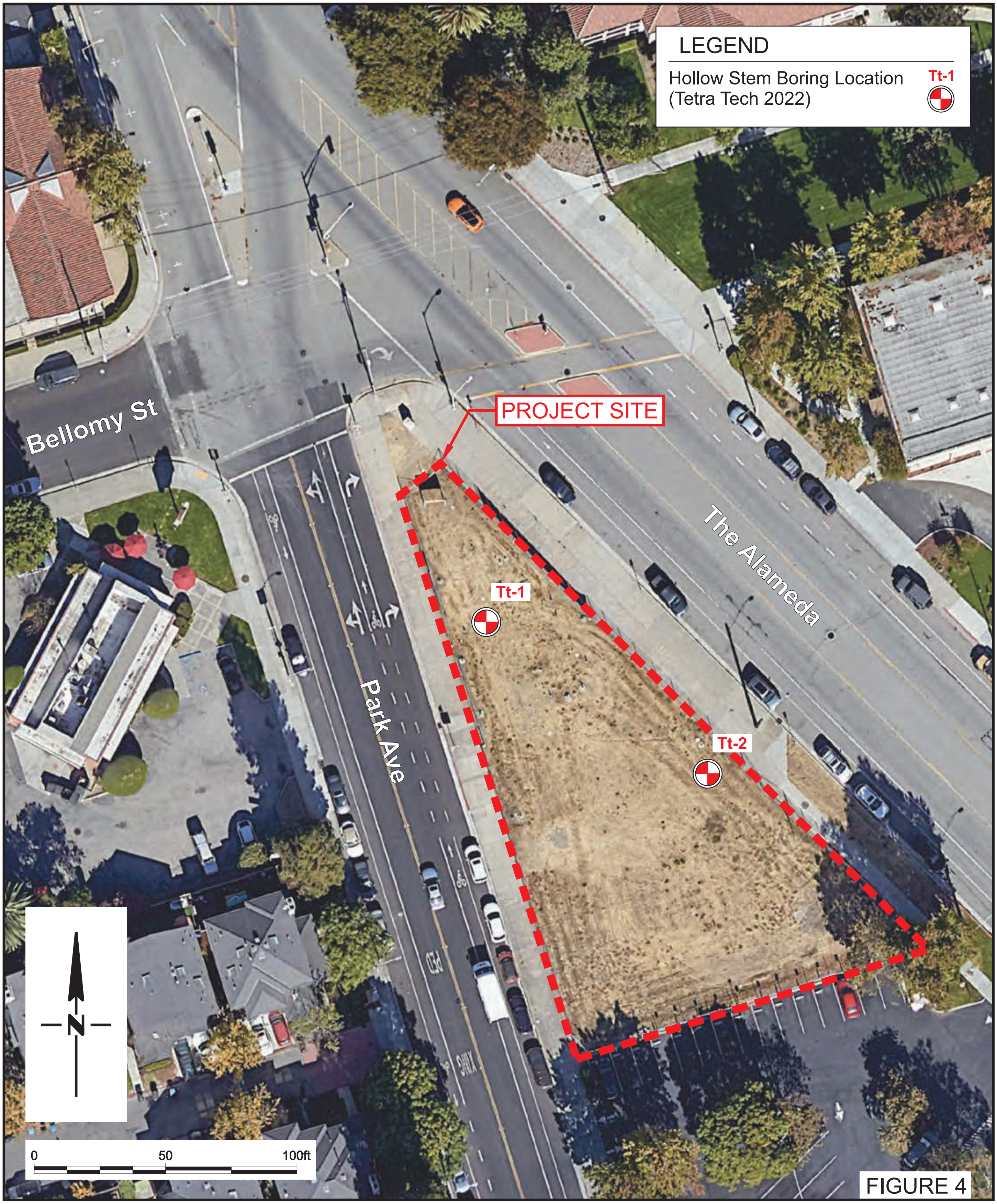

FIGURE 4

**TETRA TECH**

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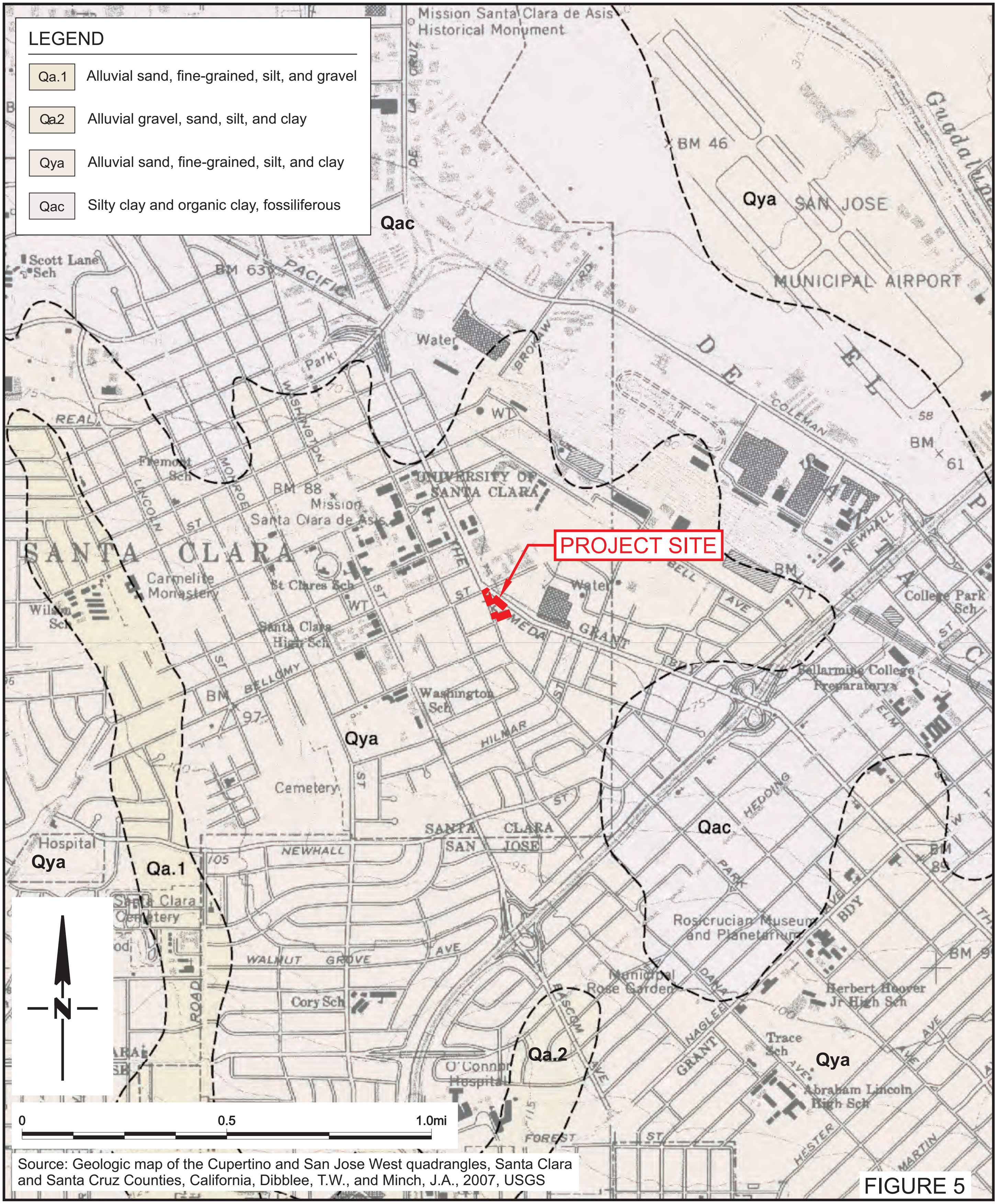
2655 THE ALAMEDA PROPOSED DEVELOPMENT - SANTA CLARA, CA

# BORING LOCATION MAP

JOB NO. TET 22-235E
DATE MAY 2022
DRAWN BY SCM
CHECKED BY MS



Source: Geologic map of the Cupertino and San Jose West quadrangles, Santa Clara and Santa Cruz Counties, California, Dibblee, T.W., and Minch, J.A., 2007, USGS

FIGURE 5

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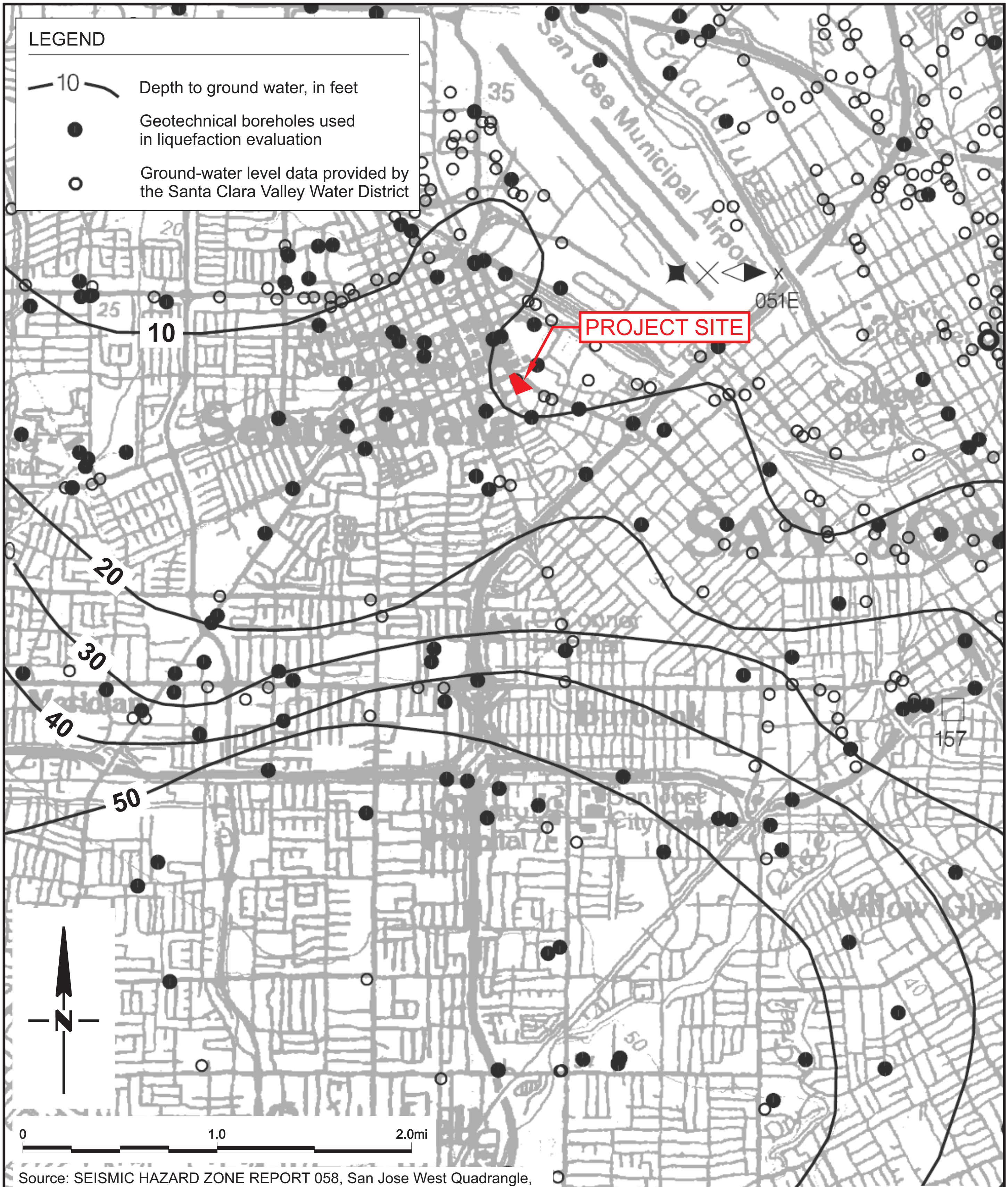
2655 THE ALAMEDA PROPOSED DEVELOPMENT - SANTA CLARA, CA

## REGIONAL GEOLOGIC MAP

JOB NO. TET 22-235E
DATE MAY 2022
DRAWN BY SCM
CHECKED BY MS



Source: SEISMIC HAZARD ZONE REPORT 058, San Jose West Quadrangle, Plate 1.2, CA Department of Conservation, Division of Mines and Geology, 2002.

FIGURE 6

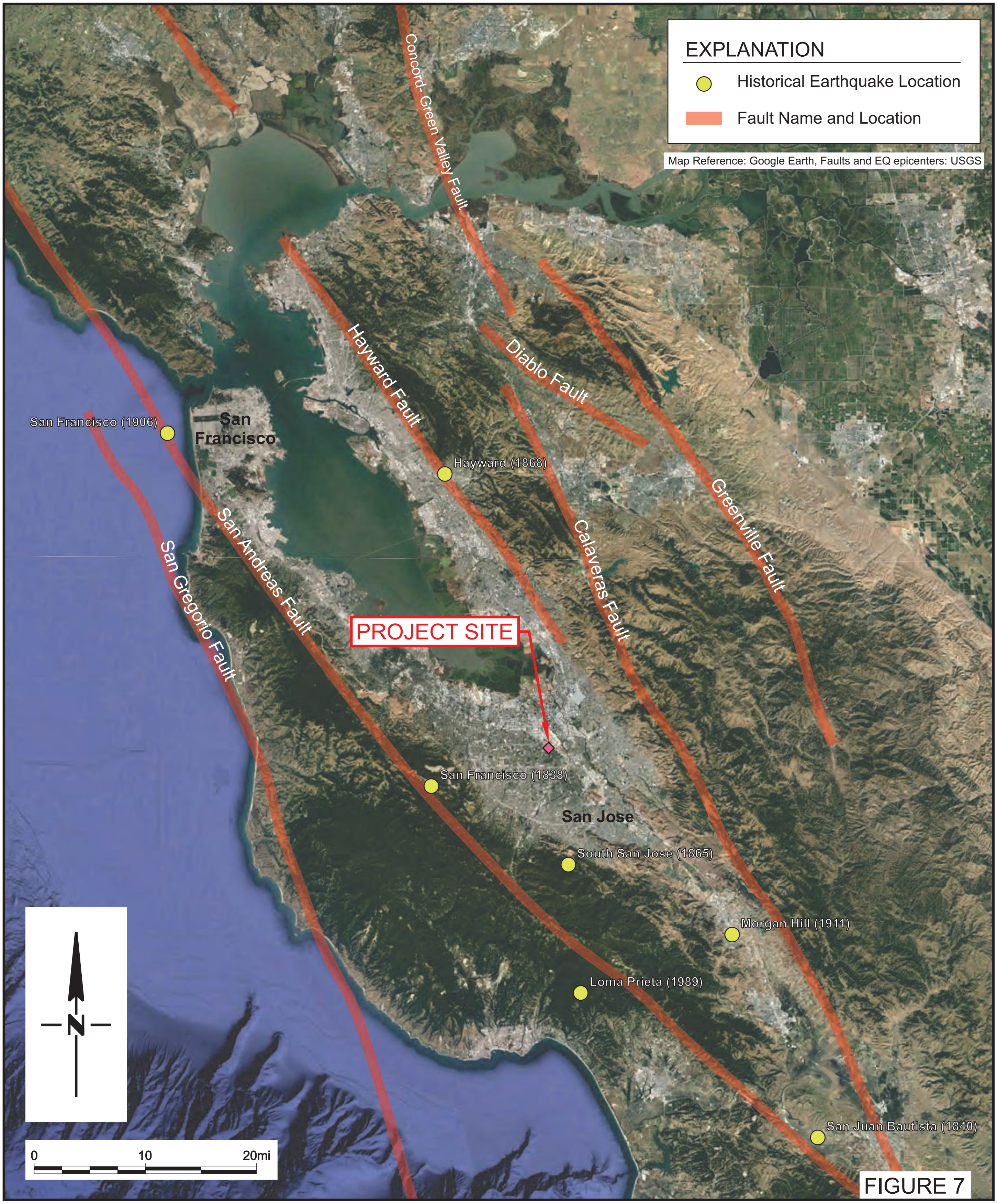
**TETRA TECH**

21700 Copley Drive, Suite 200, Diamond Bar, CA 91765
TEL 909.860.7777 www.tetratech.com

2655 THE ALAMEDA PROPOSED DEVELOPMENT - SANTA CLARA, CA

# HISTORIC HIGH GROUNDWATER MAP

| | |
|---|---|
| JOB NO. | TET 22-235E |
| DATE | MAY 2022 |
| DRAWN BY | SCM |
| CHECKED BY | MS |



Map Reference: Google Earth, Faults and EQ epicenters: USGS

FIGURE 7

**TETRA TECH**

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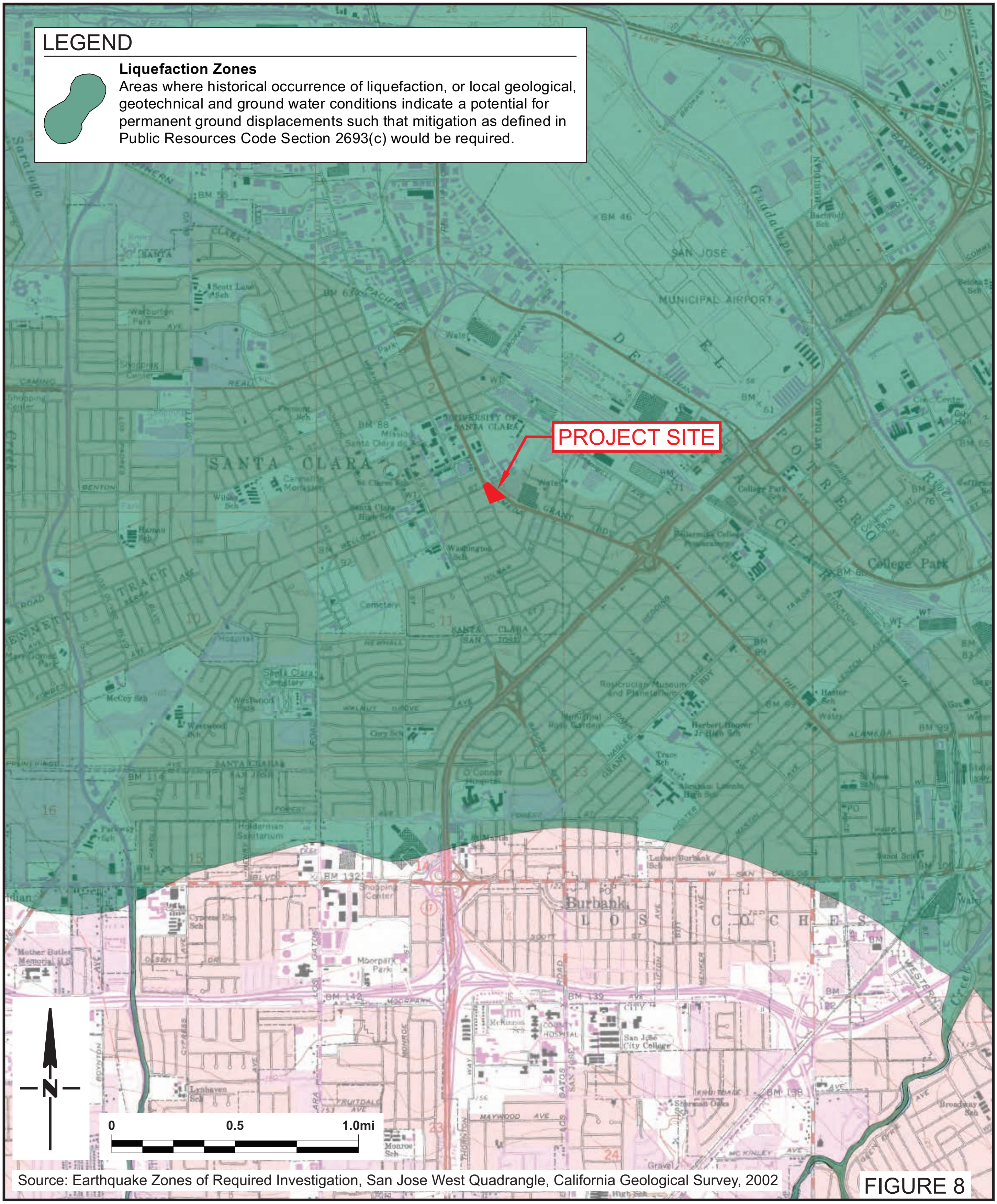
2655 THE ALAMEDA PROPOSED DEVELOPMENT - SANTA CLARA, CA

# REGIONAL FAULT AND SEISMICITY MAP

JOB NO. TET 22-235E
DATE MAY 2022
DRAWN BY TC/SCM
CHECKED BY MS



LEGEND

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

Source: Earthquake Zones of Required Investigation, San Jose West Quadrangle, California Geological Survey, 2002

FIGURE 8

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2655 THE ALAMEDA PROPOSED DEVELOPMENT - SANTA CLARA, CA

# SEISMIC HAZARD ZONES MAP

| | |
|---|---|
| JOB NO. | TET 22-235E |
| DATE | MAY 2022 |
| DRAWN BY | SCM |
| CHECKED BY | MS |

# Appendix A

# Logs of Exploratory Borings

# Appendix A

## Logs of Exploratory Borings

Bulk and relatively undisturbed drive samples were obtained in the field during our subsurface evaluation. The samples were tagged in the field and transported to our laboratory for observation and testing. The drive samples were obtained using the California Split Barrel Drive and Standard Penetration Test (SPT) sampler as described below.

### California-Type Split Barrel Drive Sampler

The split barrel drive sampler was driven with a 140-pound hammer allowed to drop freely 30 inches. The number of blows per foot recorded during sampling is presented in the logs of exploratory borings. The sampler has external and internal diameters of approximately 3.0 and 2.4 inches, respectively, and the inside of the sampler is lined with 1-inch-long brass rings. The relatively undisturbed soil sample within the rings is removed, sealed, and transported to the laboratory for observation and testing.

### Standard Penetration Test Sampler

The standard penetration test sampler is driven with a 140-pound hammer allowed to drop freely 30 inches in general accordance with ASTM D1586. The number of blows (N-value) required to drive the SPT sampler 12 inches is shown on the boring logs. The sampler has external and internal diameters of approximately 2.0 and 1.4 inches respectively. The sampling tube consists of an unlined split-tube barrel. The disturbed soil sample is removed, sealed, and transported to the laboratory for testing.

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# BORING Tt-1

Sheet 1 of 1

**PROJECT NAME** 2655 The Alameda
**CLIENT** Kapital Partners
**GROUND ELEVATION** 75 ft
**PROJECT NUMBER** 197-4552-0235
**PROJECT LOCATION** Santa Clara, CA
**GROUNDWATER DEPTH** Not encountered
**DATES DRILLED** 4/15/2022 -4/15/2022
**DRILLING CONTRACTOR** Cascade Drilling
**BOREHOLE DEPTH** 31.5 ft
**LOGGED/CHECKED BY** SCM/MS
**DRILLING METHOD** Hollow Stem Auger, CME 85
**BOREHOLE DIAMETER** 8 in
**COORDINATES** 37.346365°, -121.937575°
**HAMMER DATA** Auto 140 Lb Hammer, 30" drop
**BACKFILL** Cement grout
**LOCATION DESCRIPTION** North corner of property

◪ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧓ California-Type Ring Sample
☒ No Recovery
▮ Grab/Bulk Sample

| DEPTH (ft) | SAMPLE TYPE | NUMBER | BLOWCOUNT blows/6" (bpf) | GRAPHIC LOG | USCS | MATERIAL DESCRIPTION | DRY UNIT WT. (pcf) | MOISTURE CONTENT (%) | Notes | ELEVATION (ft) |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | ▮ | SK-1 |  | ▨ | CL | [FILL] Artificial Fill (af)<br>Sandy Lean CLAY, brown (10YR 4/3), dry to damp, trace debris (concrete fragments <4") |  |  | CORR, EI- 60, VOC-0.0ppm | 75 |
|  |  |  |  | ▨ | CL | [NATIVE] Alluvial Deposits (Qya)<br>Sandy Lean CLAY, stiff, brown (10YR 4/3), damp, coarse grained, with gravel up to 3" diameter |  |  |  |  |
| 5 | ◪ | SPT-3 | 2-4-4<br>(8) | ▨ |  |  |  |  | VOC- 0.0ppm | 70 |
|  | ⧓ | R-4 | 3-7-9<br>(16) | ▨ |  |  | 125.9 | 9.4 | DS, VOC- 0.0ppm |  |
| 10 | ◪ | SPT-5 | 1-1-2<br>(3) | ▨ |  | ...(10.0') soft, with gravel up to 1" diameter |  |  | VOC- 0.0ppm | 65 |
|  |  |  |  | ▨ | CL | Lean CLAY, firm, moist to wet, black (5Y 2.5/1) |  |  |  |  |
|  | ⧓ | R-6 | 2-4-5<br>(9) | ▨ |  |  | 83.9 | 34.3 | VOC- 0.9ppm |  |
| 15 | ◪ | SPT-7 | 1-3-5<br>(8) | ▨ |  | ...(15.0') stiff, petroleum odor |  |  | LL/PL/PI = 18/42/24, EI, VOC- 88.5ppm | 60 |
| 20 | ⧓ | R-8 | 1-3-4<br>(7) | ▨ |  | ...(20.0') firm | 89.5 | 32.2 | CONSOL, DS, VOC-4.6ppm | 55 |
| 25 | ◪ | SPT-9 | 0-1-3<br>(4) | ▨ |  |  |  |  | VOC- 484.3ppm | 50 |
|  |  |  |  | ▨ | CL | Sandy Lean CLAY, firm, bluish gray (5B 1/1), wet, gradational contact |  |  |  |  |
|  |  |  |  | ▨ | SC | Clayey SAND, loose, bluish gray (5B 1/1), wet |  |  |  |  |
| 30 | ⧓ | R-10 | 2-2-4<br>(6) | ▨ |  |  |  |  | VOC- 40.9ppm | 45 |

Notes:
1) Total depth: 31.5' bgs.
2) Groundwater not encountered before backfilling.
3) Backfilled with neat cement grout to ground surface.
4) Location from handheld GPS, elevation from Google Earth.

Tetra Tech
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Diamond Bar, CA 91765
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# BORING Tt-2

Sheet 1 of 2

**PROJECT NAME** 2655 The Alameda
**CLIENT** Kapital Partners
**GROUND ELEVATION** 75 ft
**PROJECT NUMBER** 197-4552-0235
**PROJECT LOCATION** Santa Clara, CA
**GROUNDWATER DEPTH** 30 ft
**DATES DRILLED** 4/15/2022 -4/15/2022
**DRILLING CONTRACTOR** Cascade Drilling
**BOREHOLE DEPTH** 81.5 ft
**LOGGED/CHECKED BY** SCM/MS
**DRILLING METHOD** Hollow Stem Auger, CME 85
**BOREHOLE DIAMETER** 8 in
**COORDINATES** 37.346236°, -121.937266°
**HAMMER DATA** Auto 140 Lb Hammer, 30" drop
**BACKFILL** Cement grout
**LOCATION DESCRIPTION** East side of property

◩ Standard Penetration Test (SPT)
○ No Recovery
■ Shelby Tube
⧓ California-Type Ring Sample
☒ No Recovery
▮ Grab/Bulk Sample

| DEPTH (ft) | SAMPLE TYPE NUMBER | BLOWCOUNT blows/6" (bpf) | GRAPHIC LOG | USCS | MATERIAL DESCRIPTION | DRY UNIT WT. (pcf) | MOISTURE CONTENT (%) | Notes | ELEVATION (ft) |
|---|---|---|---|---|---|---|---|---|---|
| 0 | ▮ SK-1 | | | CL | [NATIVE] Alluvial Deposits (Qya)<br>Sandy Lean CLAY, brown (10YR 4/3), damp, coarse grained, with gravel up to 3" diameter | | | | 75 |
| 5 | ⧓ R-2 | 13-11-12 (23) | | CL | ...(5.0') with gravel up to 3" diameter<br>Lean CLAY, very stiff, light olive gray (5Y 6/2) variegated olive yellow (2.5Y 6/6) staining, damp to moist, trace fine sand | | | | 70 |
| 10 | ◩ SPT-3 | 2-2-2 (4) | | | ...(10.0') firm | | | LL/PL/PI = 15/25/10, CORR, VOC- 0.1ppm | 65 |
| | ⧓ R-4 | 4-3-3 (6) | | | ...(12.5') moist to wet | 83.9 | 36.9 | CONSOL, DS | |
| 15 | ◩ SPT-5 | 3-2-2 (4) | | | ...(15.0') as above | | | LL/PL/PI = 24/50/26, #200- 91.0%, VOC-0.1ppm | 60 |
| ▽ | ⧓ R-6 | 2-3-4 (7) | | | ...(17.5') firm, mottled light yellowish brown (10YR 6/4) | 95.8 | 26.4 | CONSOL, DS | |
| | | | | ML | SILT with Clay, firm, light olive gray (5Y 6/2), moist to wet, trace fine sand, no mottling | | | | |
| 20 | ◩ SPT-7 | 1-2-3 (5) | | | ...(21.0') 1-inch sand layer | | | LL/PL/PI = 26/31/5, #200- 81.0%, VOC-0.2ppm | 55 |
| | | | | CL | Lean CLAY, firm, mottled light yellowish brown (10YR 6/4), moist to wet, trace fine sand, micaceous | | | | |
| 25 | ⧓ R-8 | 1-5-4 (9) | | | | 100.5 | 23.0 | CONSOL, DS | 50 |
| 30 ▽ | ◩ SPT-9 | 4-11-13 (24) | | | | | | VOC- 0.1ppm | 45 |
| | | | | SP | Poorly graded SAND, medium dense, bluish gray (5B 5/1), wet, fine to medium grained | | | | |
| 35 | ⧓ R-10 | 1-2-5 (7) | | CL | Lean CLAY, firm, bluish gray (5B 5/1), wet, with silt, trace fine sand and gravel | 108.2 | 20.6 | CONSOL, DS, #200-65.4%, VOC- 0.6ppm | 40 |
| 40 | | | | | | | | | 35 |

Continued on next page

# BORING Tt-2

Sheet 2 of 2

**PROJECT NAME** 2655 The Alameda **CLIENT** Kapital Partners **GROUND ELEVATION** 75 ft
**PROJECT NUMBER** 197-4552-0235 **PROJECT LOCATION** Santa Clara, CA **GROUNDWATER DEPTH** 30 ft

◪ Standard Penetration Test (SPT) ○ No Recovery ■ Shelby Tube
⧓ California-Type Ring Sample ☒ No Recovery ▮ Grab/Bulk Sample

| DEPTH (ft) | SAMPLE TYPE | NUMBER | BLOWCOUNT blows/6" (bpf) | GRAPHIC LOG | USCS | MATERIAL DESCRIPTION | DRY UNIT WT. (pcf) | MOISTURE CONTENT (%) | Notes | ELEVATION (ft) |
|---|---|---|---|---|---|---|---|---|---|---|
| 40 | ◪ | SPT-11 | 1-2-3<br>(5) | | | ...(40.0') strong petroleum odor | | | LL/PL/PI = 23/31/8, VOC- 424.6ppm | 35 |
| 45 | ⧓ | R-12 | 1-4-6<br>(10) | | | ...(45.0') firm, no silt or sand | 101.0 | 24.5 | CONSOL | 30 |
| 50 | ◪ | SPT-13 | 1-3-5<br>(8) | | | ...(50.0') stiff | | | VOC- 12.3ppm | 25 |
| 55 | ◪ | SPT-14 | 0-4-8<br>(12) | | | ...(55.0') increase in silt | | | | 20 |
| | | | | | SC | Clayey SAND, medium dense, bluish gray (5B 5/1), wet, fine grained, trace silt, gradational contact | | | | |
| | | | | | ML | SILT with Clay, very stiff, bluish gray (5B 5/1), wet, micaceous, trace fine sand, sporadic black (5Y 2.5/1) stringers and vegetative debris | | | | |
| 60 | ◪ | SPT-15 | 2-7-10<br>(17) | | | | | | VOC- 5.0ppm | 15 |
| | | | | | CL | Lean CLAY, very stiff, bluish gray (5B 5/1), wet, trace silt | | | | |
| 65 | ◪ | SPT-16 | 1-7-10<br>(17) | | | | | | | 10 |
| | | | | | SM | Silty SAND, medium dense, bluish gray (5B 5/1), wet, fine grained | | | | |
| 70 | ◪ | SPT-17 | 7-6-6<br>(12) | | | | | | | 5 |
| | | | | | GP | Poorly Graded GRAVEL with Sand, very dense, olive (5Y 4/2), fine to coarse grained, wet, with coarse grained sand | | | | |
| 75 | ◪ | SPT-18 | 15-33-44<br>(77) | | | | | | | 0 |
| 80 | ◪ | SPT-19 | 10-35-34<br>(69) | | | | | | | -5 |

Notes:
1) Total depth: 81.5' bgs.
2) Groundwater measured at 18.2' bgs 4 hours after drilling.
3) Backfilled with neat cement grout to ground surface.
4) Location from handheld GPS, elevation from Google Earth.

GEOTECH BH COLUMNS - GINT STD US LAB.GDT - L:\03 - TECHNICAL RESOURCES\02 SOFTWARE LIBRARY\GINT\PROJECTS\197-4552-0235 2655 THE ALAMEDA.GPJ

# Appendix B

# Results of Laboratory Testing

# Appendix B
# Results of Current Laboratory Testing

### Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

### In-Place Moisture and Dry Density Tests

The moisture contents and dry densities of relatively undisturbed samples obtained from the exploratory boreholes were evaluated in general accordance with the latest version of ASTM D2937. The test results are presented on the log of the exploratory borings in Appendix A.

### Percent Passing #200 Sieve

An evaluation of the percent passing #200 sieve for selected soil samples were performed in general accordance with ASTM D1140. The results of the analysis are presented the borehole logs in Appendix A and in the back of this Appendix B.

### Atterberg Limits Tests

Liquid Limit, Plastic Limit, and Plasticity Index of selected and representative on-site materials were performed in general accordance with ASTM D4318. The results of this test are presented on the borehole logs in Appendix A and in the back of this Appendix B.

### Consolidation Tests

Consolidation tests were performed on selected relatively undisturbed soil samples in general accordance with the latest version of ASTM D2435. The samples were inundated during testing to represent adverse field conditions. The percent consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. Load versus deformation curves are presented in the back of this Appendix B.

### Direct Shear Tests

The sample were tested under three different normal loads. The samples were sheared at a constant rate of strain selected in general accordance with the consolidation characteristics of the soils (Section 7.3 of ASTM D3080). The samples were inundated during shearing to represent potentially adverse field conditions. The strain rate used for shear tests was 0.002 inches per minute. Shearing of the specimens was continued until the shear stress became essentially constant or until a deformation of approximately 10 percent of the original diameter had been reached. The results of 3-point direct shear tests are presented in the back of this Appendix B.

### Corrosivity Series

The corrosivity of selected samples was evaluated in general accordance with the latest version of California Test Method (CTM) No. 417, 422 and 643. The results of these tests are presented in Table 11 in the report and in the back of this Appendix B.

TETRA TECH

# MOISTURE CONTENT AND DENSITY

## ASTM D7263

| | | | |
|---|---|---|---|
| **Job Name:** | **Tt Alameda** | **Date Sampled:** | 4/20/2022 |
| **Job Number:** | **TET-22-235E** | **Date Completed:** | 4/28/2022 |
| **Tested By:** | **MG** | **Note:** | |

| **Boring / Test Pit / Trench** | | **Tt-1** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| **Sample Number** | | **R-6** | | | | | | | | |
| **Sample Depth** | *ft* | **12.5-14** | | | | | | | | |
| **USCS Soil Description** | | Olive Gray Native CL | | | | | | | | |
| Number of Rings | | 6 | | | | | | | | |
| Total Weight Rings + Soil | *grams* | 1075.70 | | | | | | | | |
| Volume of Rings | *$ft^3$* | 0.0159 | | | | | | | | |
| Weight of Rings | *grams* | 261.26 | | | | | | | | |
| Weight of Soil | *grams* | 814.44 | | | | | | | | |
| Wet Density | *pcf* | 112.61 | | | | | | | | |
| | Container ID | X33 | | | | | | | | |
| Tare | *grams* | 10.5 | | | | | | | | |
| Wet Soil + Tare | *grams* | 151.5 | | | | | | | | |
| Dry Soil + Tare | *grams* | 115.5 | | | | | | | | |
| Weight of Water | *grams* | 36 | | | | | | | | |

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| **Dry Density** | ***pcf*** | **83.9** | | | | | | | | |
| **Moisture Content** | ***%*** | **34.3** | | | | | | | | |

# PERCENT PASSING # 200 SIEVE

## ASTM D1140

| | | | |
|---|---|---|---|
| **Job Name:** | Alameda Tt | **Date Completed:** | 4/20/2022 |
| **Job Number:** | TET-22-235E | **Date Sampled:** | April 28, 2022 |
| **Note:** | | | |
| **Tested By :** | MG | | |

| Boring Number | Sample Number | Depth (ft) | Weight Before Wash - Dry (grams) | Weight After Wash - Dry (grams) | Percent Passing # 200 Sieve | USCS Classification |
|---|---|---|---|---|---|---|
| **Tt-2** | **SPT-7** | 20-21.5 | 177 | 33.2 | 81% | ML |
| **Tt-2** | **SPT-5** | 15-16.5 | 199.9 | 18.7 | 91% | CL |
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# PERCENT PASSING NO. 200 SIEVE

## ASTM D1140

Client: Tetra Tech

Project Name: 2655 The Alameda Santa Clara, CA

Project Number: TET 22-235E

AP Lab No.: 22-0458

Test Date: 04/22/22

| Boring No. | Sample No. | Depth (ft) | Percent Fines (%) |
|---|---|---|---|
| Tt-2 | R-10 | 35-36.5 | 65.4 |
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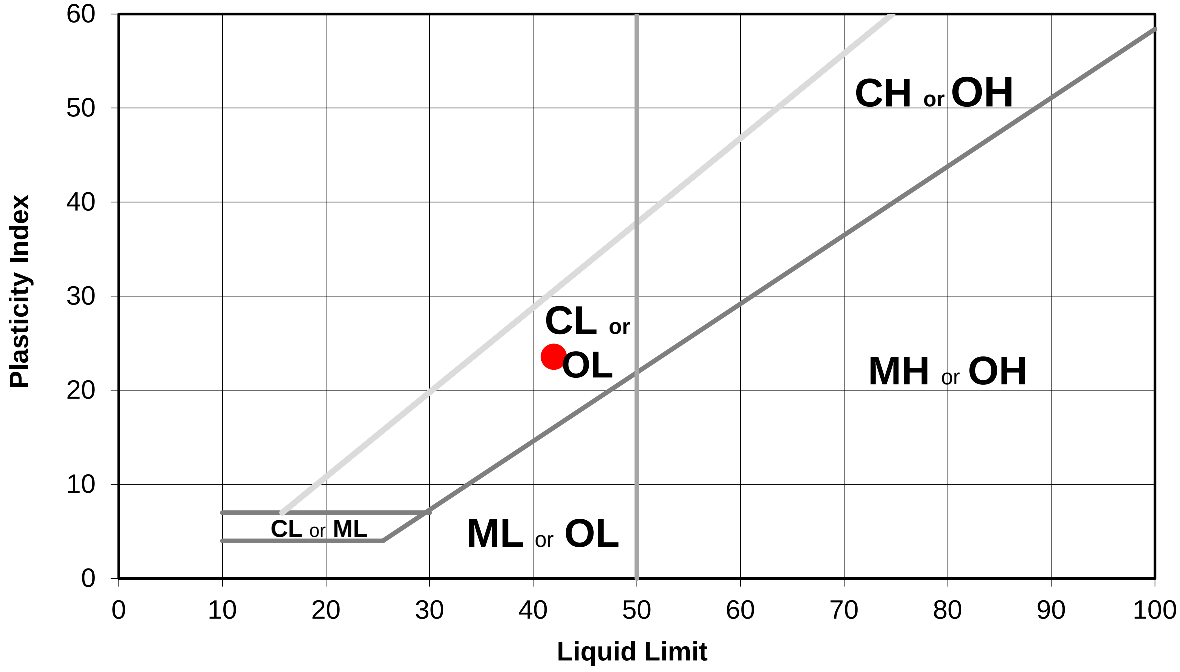
**TETRA TECH**

# ATTERBERG LIMITS

## ASTM D4318

| | | | |
|---|---|---|---|
| **Job Name:** | Alameda Tt | **Date Sampled:** | 4/20/2022 |
| **Job Number:** | TET-22-235E | **Date Completed:** | 5/9/2022 |
| **Tested By:** | MG | **Sample Identification:** | TT-1, SPT-7 |
| **Note:** | | **Sample Depth:** | 15-16.5ft |
| **Sample Description:** | Olive Gray CLAY, CL | | |

| | | Plastic Limit | | Liquid Limit | | | |
|---|---|---|---|---|---|---|---|
| **Test No.** | | **1** | **2** | **1** | **2** | **3** | **4** |
| Number of Blows | | | | 35 | 26 | 16 | |
| Container ID | | D1 | D5 | N11 | P13 | S17 | |
| Wet Weight of Soil + Cont. | *grams* | 21.50 | 21.30 | 43.70 | 46.70 | 48.50 | |
| Dry Weight of Soil + Cont. | *grams* | 20.10 | 19.90 | 38.70 | 40.40 | 41.50 | |
| Weight of Container | *grams* | 12.40 | 12.40 | 25.50 | 25.40 | 25.70 | |
| Moisture Weight | *grams* | 1.40 | 1.40 | 5.00 | 6.30 | 7.00 | |
| Weight of Dry Soil | *grams* | 7.70 | 7.50 | 13.20 | 15.00 | 15.80 | |
| **Moisture Content** | *%* | **18.2** | **18.7** | **37.9** | **42.0** | **44.3** | |



| | |
|---|---|
| **Plastic Limit** | **18** |
| **Liquid Limit** | **42** |
| **Plasticity Index** | **24** |

**USCS Classification** **CL**

Based on Atterberg Limits only

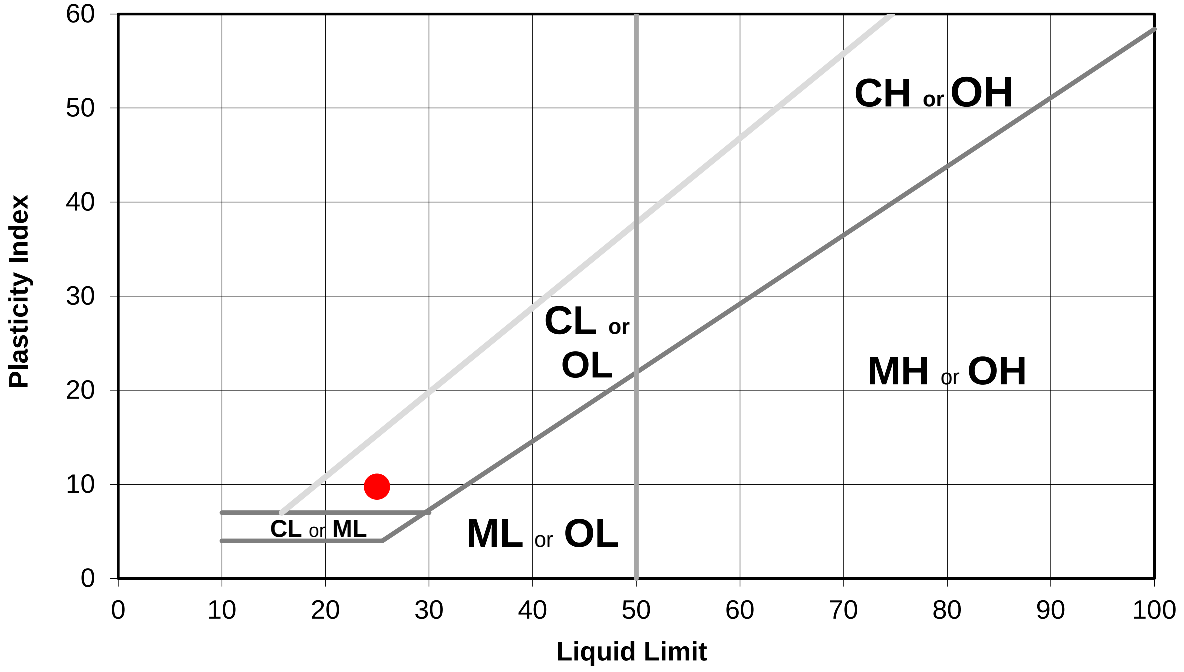
TETRA TECH

# ATTERBERG LIMITS

## ASTM D4318

| | | | |
|---|---|---|---|
| **Job Name:** | Alameda Tt | **Date Sampled:** | 4/20/2022 |
| **Job Number:** | TET-22-235E | **Date Completed:** | 5/9/2022 |
| **Tested By:** | MG | **Sample Identification:** | TT-2, SPT-3 |
| **Note:** | | **Sample Depth:** | 10-11.5ft |
| **Sample Description:** | Brown Lean CLAY with Sand, CL | | |

| | | Plastic Limit | | Liquid Limit | | | |
|---|---|---|---|---|---|---|---|
| **Test No.** | | **1** | **2** | **1** | **2** | **3** | **4** |
| Number of Blows | | ▨ | ▨ | 35 | 24 | 15 | |
| Container ID | | F3 | P8 | S5 | M14 | T23 | |
| Wet Weight of Soil + Cont. | *grams* | 21.30 | 20.10 | 53.80 | 46.30 | 60.60 | |
| Dry Weight of Soil + Cont. | *grams* | 20.10 | 19.10 | 48.80 | 42.10 | 52.80 | |
| Weight of Container | *grams* | 12.40 | 12.40 | 25.70 | 25.40 | 25.80 | |
| Moisture Weight | *grams* | 1.20 | 1.00 | 5.00 | 4.20 | 7.80 | |
| Weight of Dry Soil | *grams* | 7.70 | 6.70 | 23.10 | 16.70 | 27.00 | |
| **Moisture Content** | *%* | **15.6** | **14.9** | **21.6** | **25.1** | **28.9** | |



| | |
|---|---|
| **Plastic Limit** | **15** |
| **Liquid Limit** | **25** |
| **Plasticity Index** | **10** |

**USCS Classification** **CL**

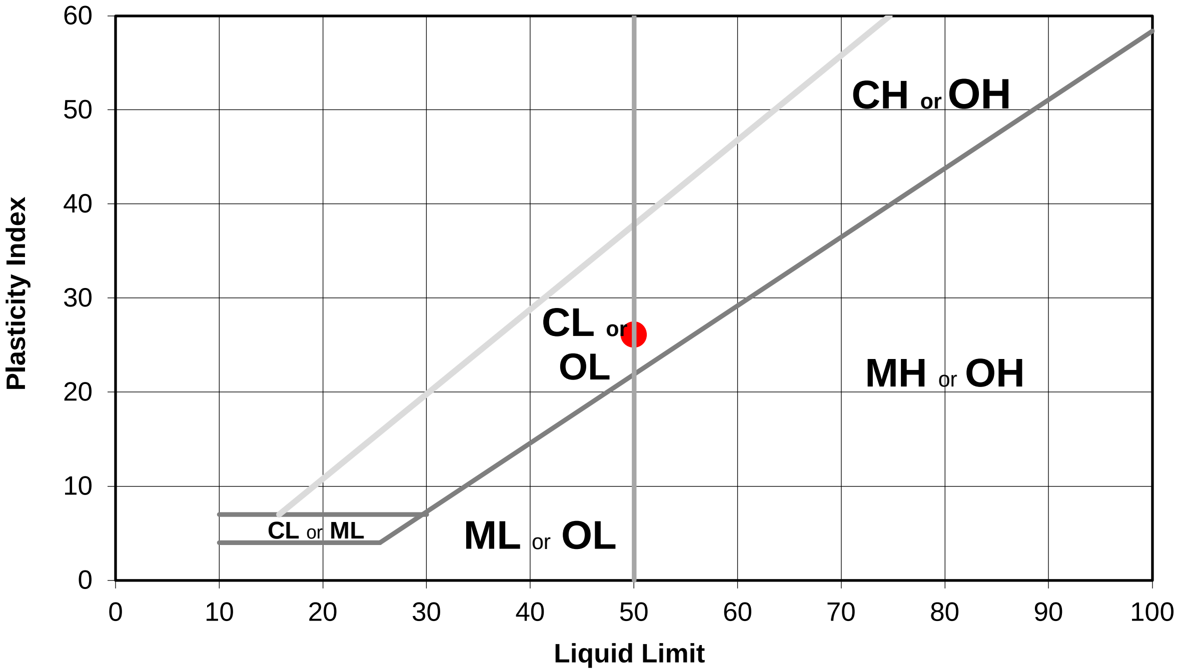
Based on Atterberg Limits only

# ATTERBERG LIMITS

## ASTM D4318

| | | | |
|---|---|---|---|
| **Job Name:** | Alameda Tt | **Date Sampled:** | 4/20/2022 |
| **Job Number:** | TET-22-235E | **Date Completed:** | 4/28/2022 |
| **Tested By:** | MG | **Sample Identification:** | Tt-2 SPT-5 |
| **Note:** | | **Sample Depth:** | 15-16.5ft |
| **Sample Description:** | Dark Olive Gray CL Native | | |

| | | **Plastic Limit** | | **Liquid Limit** | | | |
|---|---|---|---|---|---|---|---|
| **Test No.** | | **1** | **2** | **1** | **2** | **3** | **4** |
| Number of Blows | | ▨ | ▨ | 36 | 25 | 16 | |
| Container ID | | F3 | D1 | P40 | S5 | S17 | |
| Wet Weight of Soil + Cont. | *grams* | 23.20 | 22.90 | 48.80 | 43.30 | 49.90 | |
| Dry Weight of Soil + Cont. | *grams* | 21.20 | 20.80 | 41.10 | 37.40 | 41.20 | |
| Weight of Container | *grams* | 12.40 | 12.40 | 24.20 | 25.70 | 25.70 | |
| Moisture Weight | *grams* | 2.00 | 2.10 | 7.70 | 5.90 | 8.70 | |
| Weight of Dry Soil | *grams* | 8.80 | 8.40 | 16.90 | 11.70 | 15.50 | |
| **Moisture Content** | *%* | **22.7** | **25.0** | **45.6** | **50.4** | **56.1** | |



| | |
|---|---|
| **Plastic Limit** | **24** |
| **Liquid Limit** | **50** |
| **Plasticity Index** | **26** |

**USCS Classification** **CL/CH**

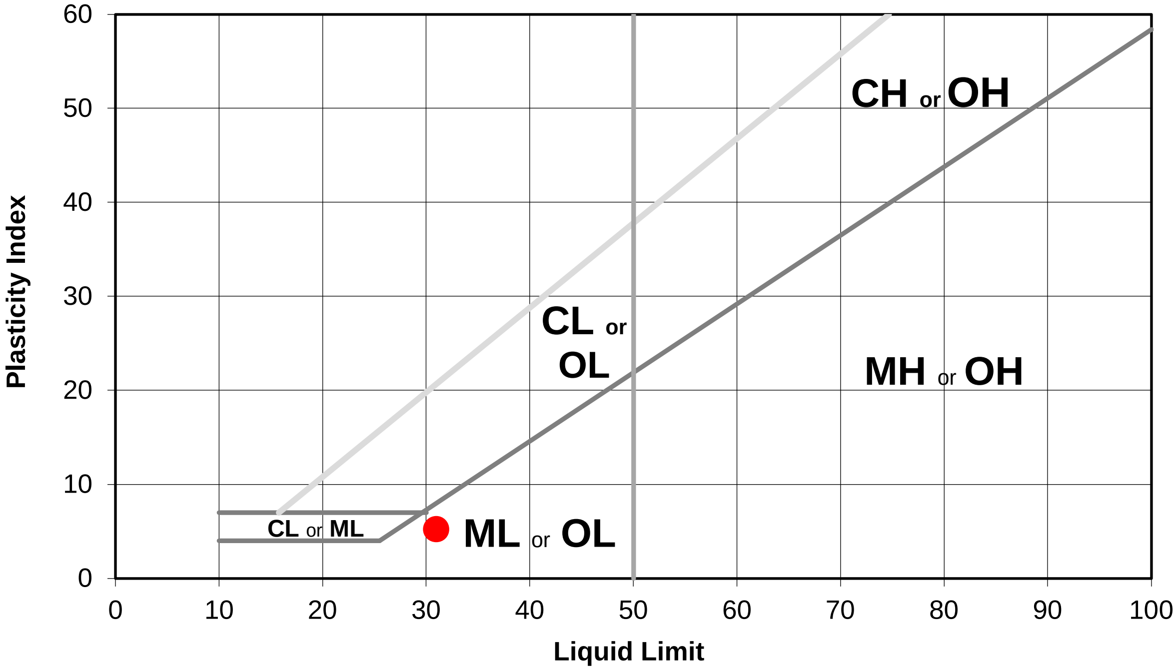
Based on Atterberg Limits only

# ATTERBERG LIMITS

## ASTM D4318

| | | | |
|---|---|---|---|
| **Job Name:** | Alameda Tt | **Date Sampled:** | 4/20/2022 |
| **Job Number:** | TET-22-235E | **Date Completed:** | 4/28/2022 |
| **Tested By:** | MG | **Sample Identification:** | Tt-2 SPT-7 |
| **Note:** | | **Sample Depth:** | 20-21.5ft |
| **Sample Description:** | Olive Gray ML Native | | |

| | | Plastic Limit | | Liquid Limit | | | |
|---|---|---|---|---|---|---|---|
| **Test No.** | | **1** | **2** | **1** | **2** | **3** | **4** |
| Number of Blows | | ▨ | ▨ | 34 | 25 | 15 | |
| Container ID | | P8 | P5 | T38 | P26 | P13 | |
| Wet Weight of Soil + Cont. | *grams* | 24.80 | 21.20 | 51.80 | 55.50 | 52.20 | |
| Dry Weight of Soil + Cont. | *grams* | 22.40 | 19.30 | 45.80 | 48.20 | 45.30 | |
| Weight of Container | *grams* | 12.40 | 12.40 | 24.20 | 25.20 | 25.40 | |
| Moisture Weight | *grams* | 2.40 | 1.90 | 6.00 | 7.30 | 6.90 | |
| Weight of Dry Soil | *grams* | 10.00 | 6.90 | 21.60 | 23.00 | 19.90 | |
| **Moisture Content** | *%* | **24.0** | **27.5** | **27.8** | **31.7** | **34.7** | |



| | |
|---|---|
| **Plastic Limit** | **26** |
| **Liquid Limit** | **31** |
| **Plasticity Index** | **5** |

**USCS Classification** **ML**

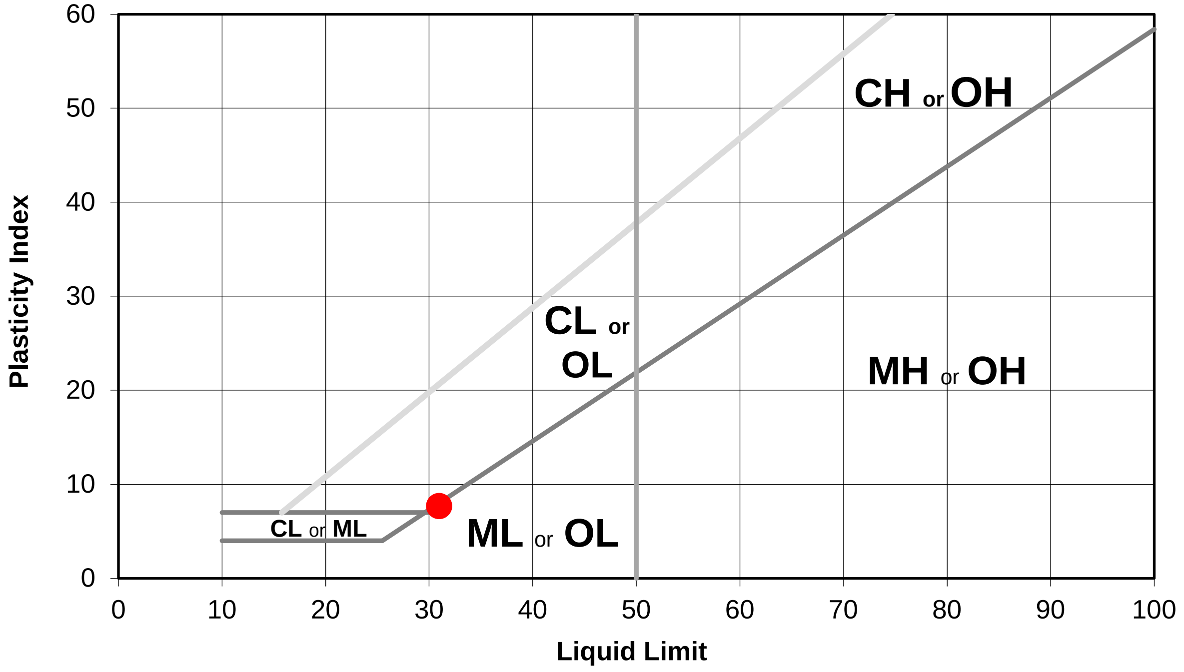
Based on Atterberg Limits only

# ATTERBERG LIMITS

## ASTM D4318

| | | | |
|---|---|---|---|
| **Job Name:** | Alameda Tt | **Date Sampled:** | 4/20/2022 |
| **Job Number:** | TET-22-235E | **Date Completed:** | 5/9/2022 |
| **Tested By:** | MG | **Sample Identification:** | TT-2, SPT-11 |
| **Note:** | | **Sample Depth:** | 40-41.5ft |
| **Sample Description:** | Brown CLAY or SILT CL/ML | | |

| | | Plastic Limit | | Liquid Limit | | | |
|---|---|---|---|---|---|---|---|
| **Test No.** | | **1** | **2** | **1** | **2** | **3** | **4** |
| Number of Blows | | ▨ | ▨ | 32 | 25 | 15 | |
| Container ID | | P4 | F10 | A1 | P26 | S12 | |
| Wet Weight of Soil + Cont. | *grams* | 20.00 | 20.70 | 50.20 | 51.50 | 49.50 | |
| Dry Weight of Soil + Cont. | *grams* | 18.50 | 19.20 | 44.50 | 45.30 | 43.50 | |
| Weight of Container | *grams* | 12.40 | 12.40 | 25.30 | 25.20 | 25.60 | |
| Moisture Weight | *grams* | 1.50 | 1.50 | 5.70 | 6.20 | 6.00 | |
| Weight of Dry Soil | *grams* | 6.10 | 6.80 | 19.20 | 20.10 | 17.90 | |
| **Moisture Content** | *%* | **24.6** | **22.1** | **29.7** | **30.8** | **33.5** | |



| | |
|---|---|
| **Plastic Limit** | **23** |
| **Liquid Limit** | **31** |
| **Plasticity Index** | **8** |

**USCS Classification** **CL or ML**

Based on Atterberg Limits only

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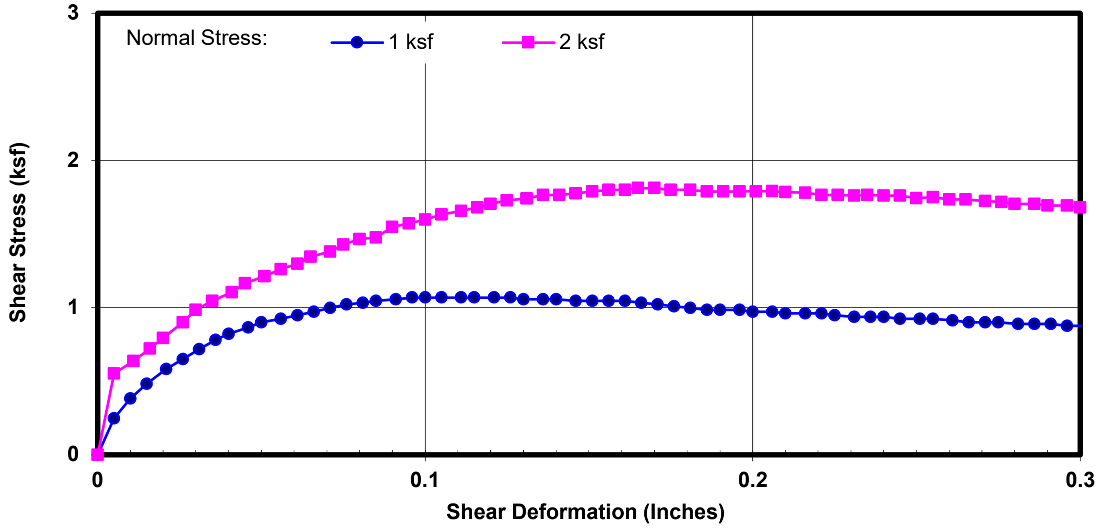
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

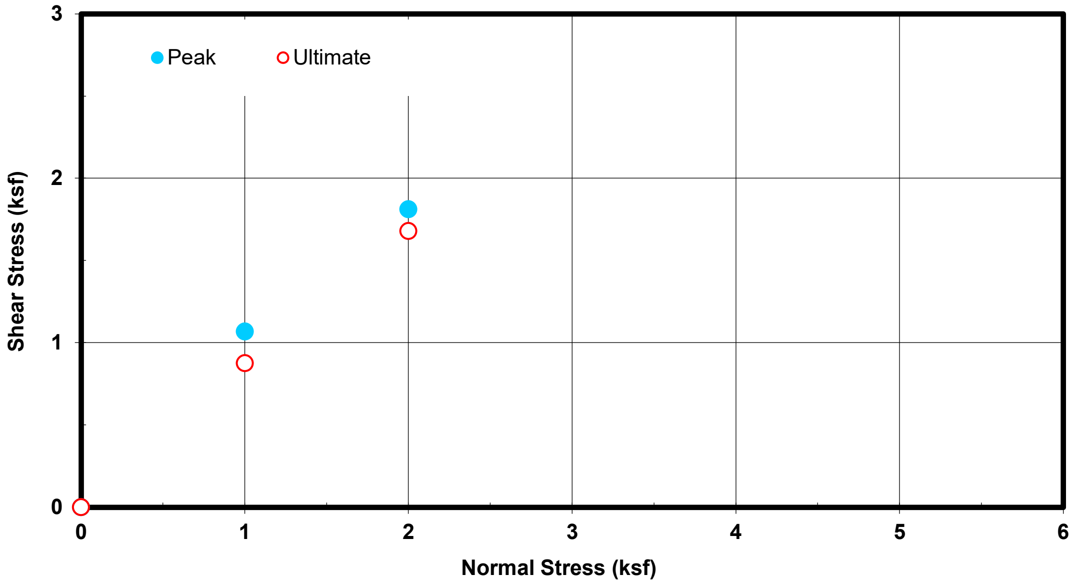
# DIRECT SHEAR TEST RESULTS
## ASTM D 3080

**Client:** Tetra Tech
**Project Name:** 2655 The Alameda Santa Clara, CA
**Project No.:** TET 22-235E
**Boring No.:** Tt-1
**Sample No.:** R-4 **Depth (ft):** 7.5-9
**Sample Type:** Mod. Cal.
**Soil Description:** Sandy Clay
**Test Condition:** Inundated **Shear Type:** Regular

**Tested By:** ST **Date:** 04/27/22
**Computed By:** NR **Date:** 04/28/22
**Checked by:** AP **Date:** 05/04/22

| Wet Unit Weight (pcf) | Dry Unit Weight (pcf) | Initial Moisture Content (%) | Final Moisture Content (%) | Initial Degree Saturation (%) | Final Degree Saturation (%) | Normal Stress (ksf) | Peak Shear Stress (ksf) | Ultimate Shear Stress (ksf) |
|---|---|---|---|---|---|---|---|---|
| 137.8 | 125.9 | 9.4 | 12.3 | 75 | 98 | 1 | 1.068 | 0.876 |
| | | | | | | 2 | 1.812 | 1.680 |
| | | | | | | | | |



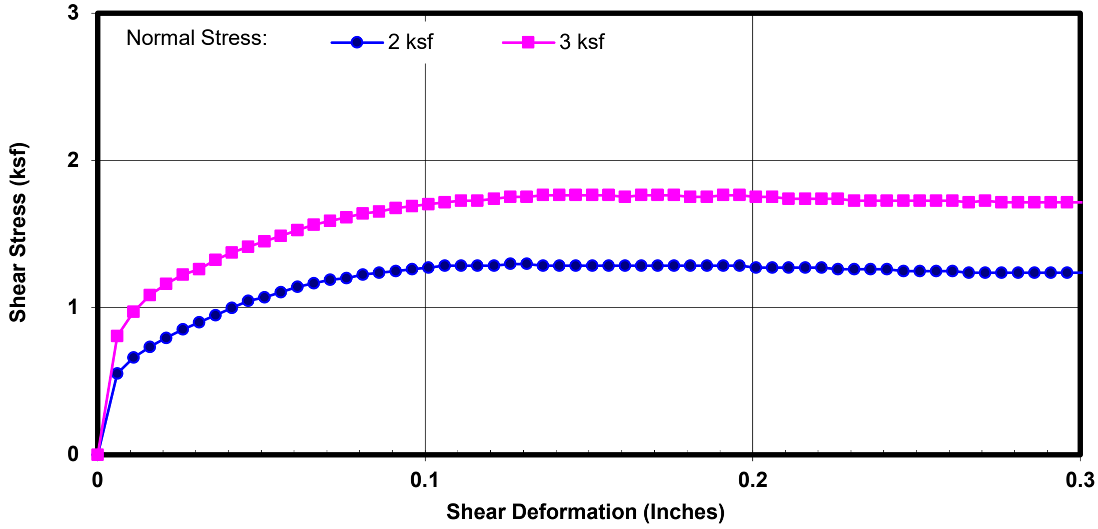


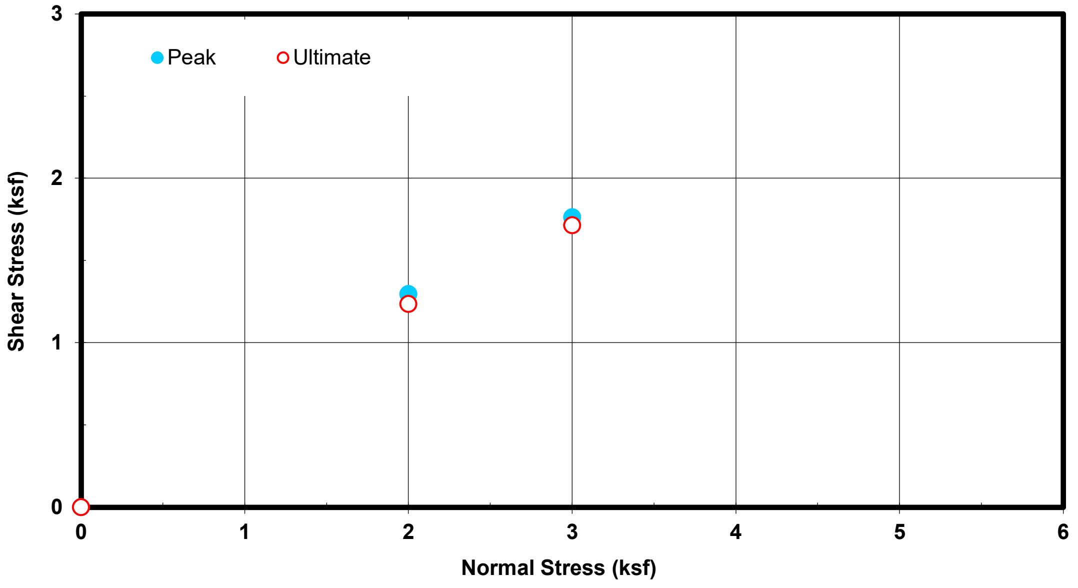
# DIRECT SHEAR TEST RESULTS
## ASTM D 3080

**Client:** Tetra Tech
**Project Name:** 2655 The Alameda Santa Clara, CA
**Project No.:** TET 22-235E
**Boring No.:** Tt-1
**Sample No.:** R-8 **Depth (ft):** 20-21.5
**Sample Type:** Mod. Cal.
**Soil Description:** Clay
**Test Condition:** Inundated **Shear Type:** Regular

**Tested By:** ST **Date:** 04/27/22
**Computed By:** NR **Date:** 04/28/22
**Checked by:** AP **Date:** 05/04/22

| Wet Unit Weight (pcf) | Dry Unit Weight (pcf) | Initial Moisture Content (%) | Final Moisture Content (%) | Initial Degree Saturation (%) | Final Degree Saturation (%) | Normal Stress (ksf) | Peak Shear Stress (ksf) | Ultimate Shear Stress (ksf) |
|---|---|---|---|---|---|---|---|---|
| 117.4 | 88.8 | 32.2 | 33.3 | 97 | 100 | 2 | 1.296 | 1.236 |
| | | | | | | 3 | 1.764 | 1.714 |
| | | | | | | | | |



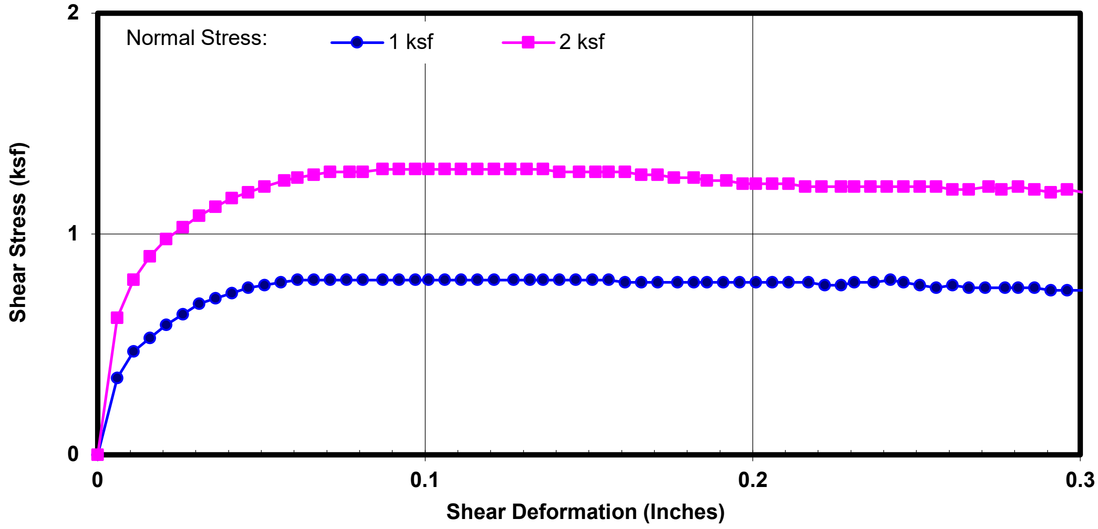


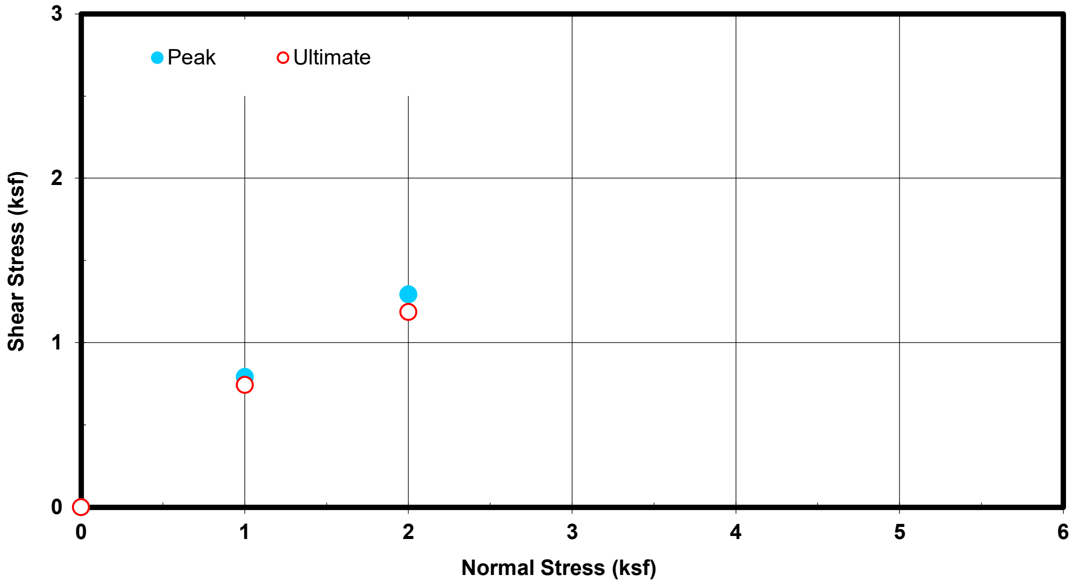
# DIRECT SHEAR TEST RESULTS
## ASTM D 3080

**Client:** Tetra Tech
**Project Name:** 2655 The Alameda Santa Clara, CA
**Project No.:** TET 22-235E
**Boring No.:** Tt-2
**Sample No.:** R-4 **Depth (ft):** 12.5-14
**Sample Type:** Mod. Cal.
**Soil Description:** Lean Clay
**Test Condition:** Inundated **Shear Type:** Regular

**Tested By:** ST **Date:** 04/27/22
**Computed By:** NR **Date:** 04/28/22
**Checked by:** AP **Date:** 05/04/22

| Wet Unit Weight (pcf) | Dry Unit Weight (pcf) | Initial Moisture Content (%) | Final Moisture Content (%) | Initial Degree Saturation (%) | Final Degree Saturation (%) | Normal Stress (ksf) | Peak Shear Stress (ksf) | Ultimate Shear Stress (ksf) |
|---|---|---|---|---|---|---|---|---|
| 114.8 | 83.9 | 36.9 | 37.6 | 99 | 100 | 1 | 0.792 | 0.744 |
| | | | | | | 2 | 1.294 | 1.188 |
| | | | | | | | | |





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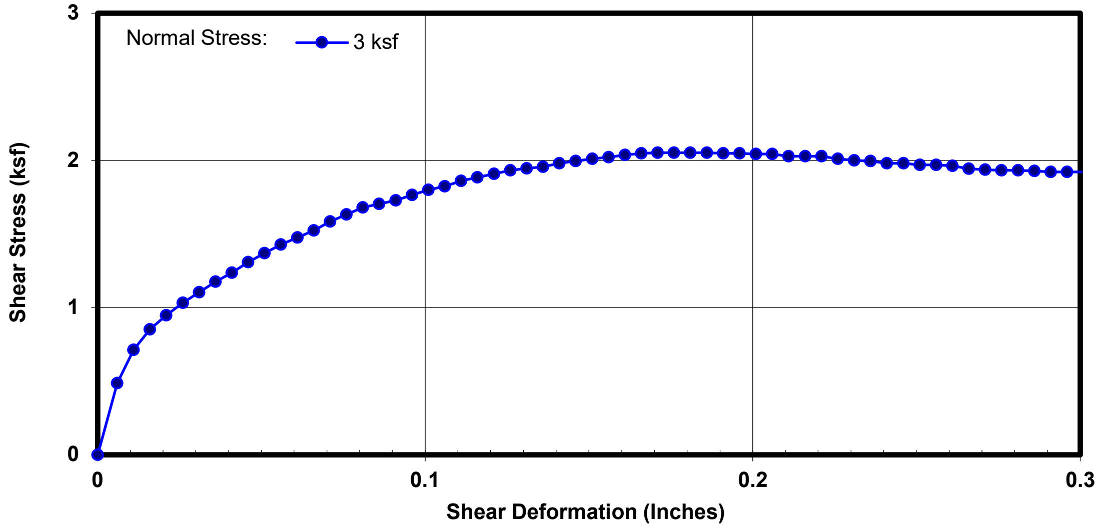
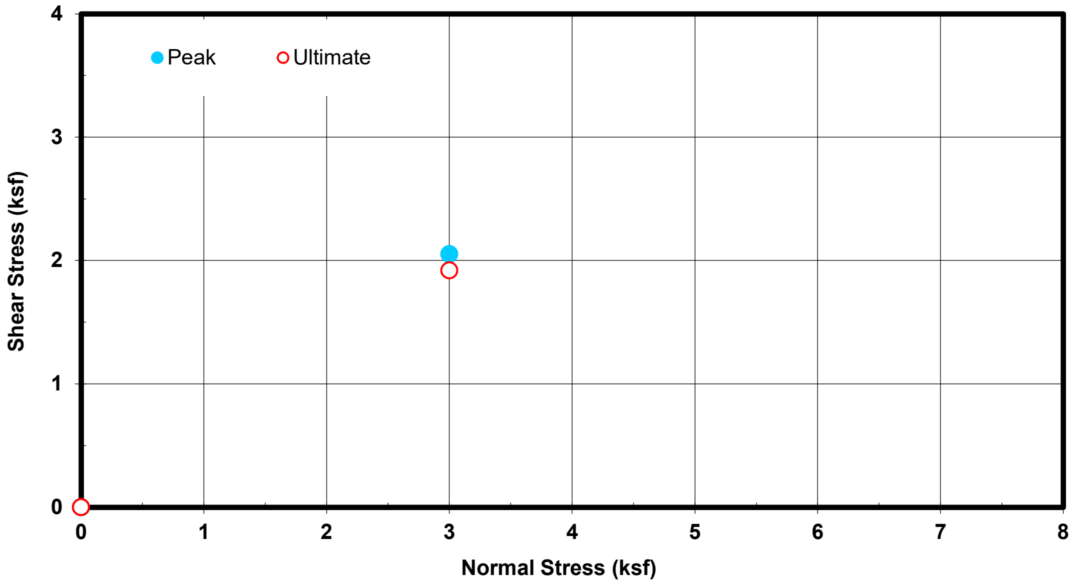
# DIRECT SHEAR TEST RESULTS
## ASTM D 3080

**Client:** Tetra Tech
**Project Name:** 2655 The Alameda Santa Clara, CA
**Project No.:** TET 22-235E
**Boring No.:** Tt-2
**Sample No.:** R-6 **Depth (ft):** 17.5-19
**Sample Type:** Mod. Cal.
**Soil Description:** Clay
**Test Condition:** Inundated **Shear Type:** Regular

**Tested By:** LS **Date:** 04/27/22
**Computed By:** NR **Date:** 05/03/22
**Checked by:** AP **Date:** 05/04/22

| Wet Unit Weight (pcf) | Dry Unit Weight (pcf) | Initial Moisture Content (%) | Final Moisture Content (%) | Initial Degree Saturation (%) | Final Degree Saturation (%) | Normal Stress (ksf) | Peak Shear Stress (ksf) | Ultimate Shear Stress (ksf) |
|---|---|---|---|---|---|---|---|---|
| 121.1 | 95.8 | 26.4 | 28.2 | 94 | 100 | 3 | 2.052 | 1.920 |
| | | | | | | | | |
| | | | | | | | | |



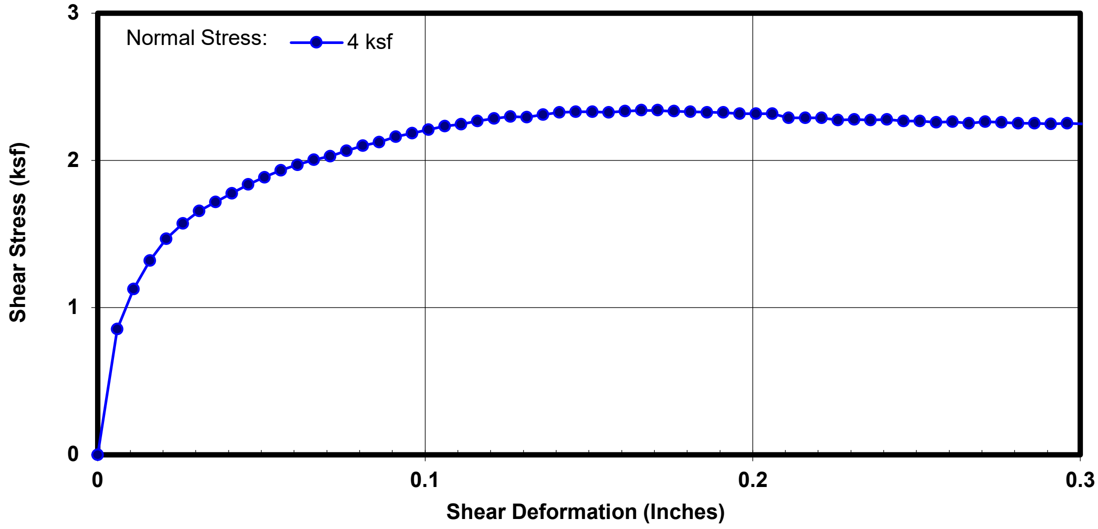


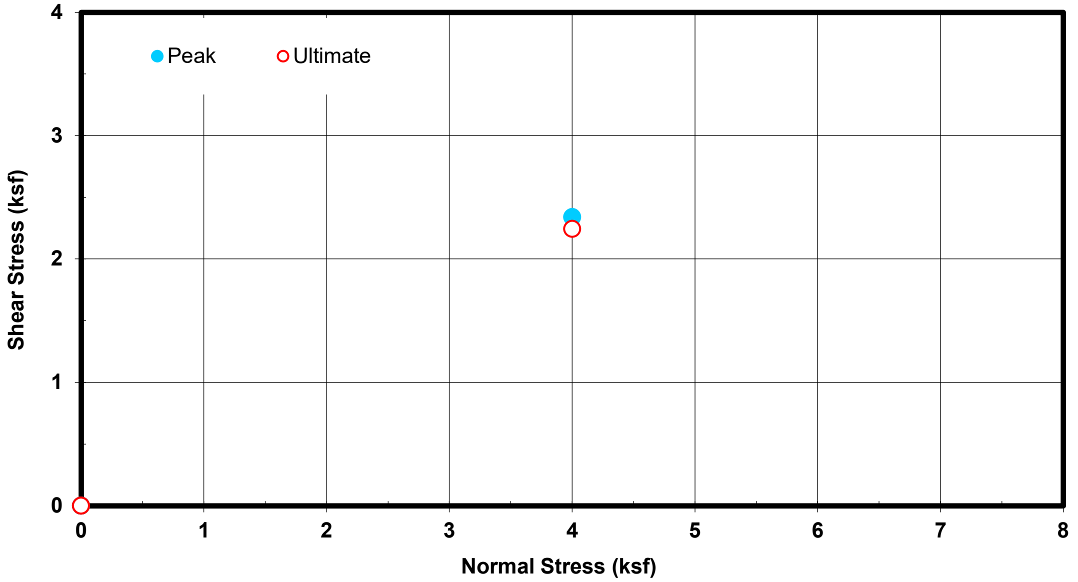
# DIRECT SHEAR TEST RESULTS
## ASTM D 3080

**Client:** Tetra Tech
**Project Name:** 2655 The Alameda Santa Clara, CA
**Project No.:** TET 22-235E
**Boring No.:** Tt-2
**Sample No.:** R-8 **Depth (ft):** 25-26.5
**Sample Type:** Mod. Cal.
**Soil Description:** Lean Clay
**Test Condition:** Inundated **Shear Type:** Regular

**Tested By:** LS **Date:** 04/27/22
**Computed By:** NR **Date:** 05/03/22
**Checked by:** AP **Date:** 05/04/22

| Wet Unit Weight (pcf) | Dry Unit Weight (pcf) | Initial Moisture Content (%) | Final Moisture Content (%) | Initial Degree Saturation (%) | Final Degree Saturation (%) | Normal Stress (ksf) | Peak Shear Stress (ksf) | Ultimate Shear Stress (ksf) |
|---|---|---|---|---|---|---|---|---|
| 123.7 | 100.5 | 23.0 | 25.0 | 92 | 100 | 4 | 2.340 | 2.246 |
| | | | | | | | | |
| | | | | | | | | |



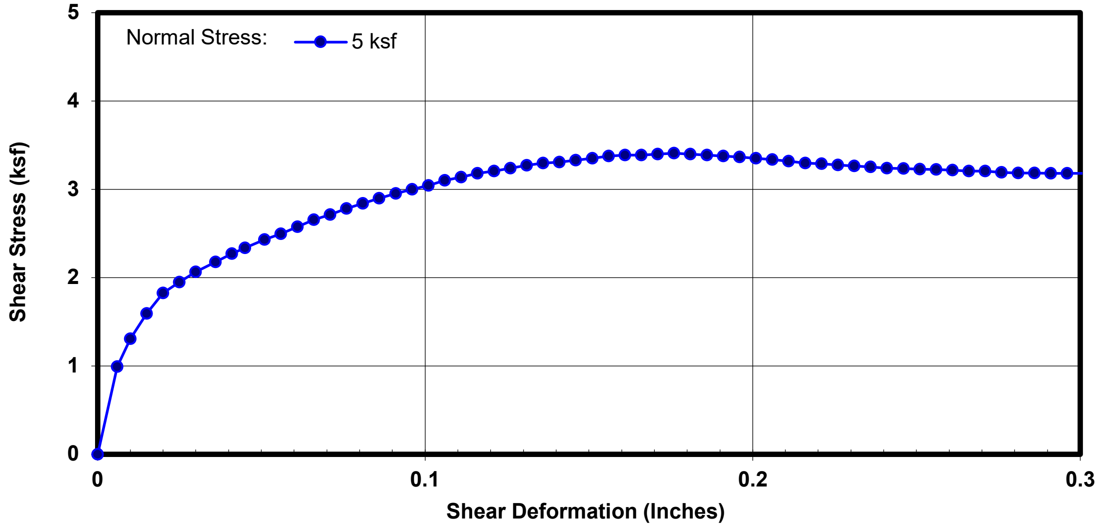


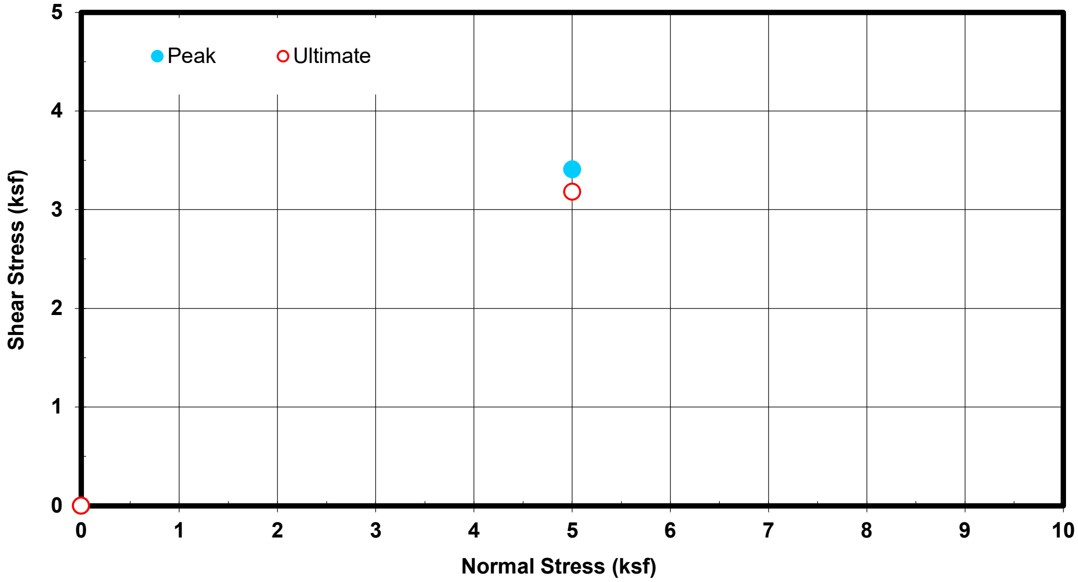
# DIRECT SHEAR TEST RESULTS
## ASTM D 3080

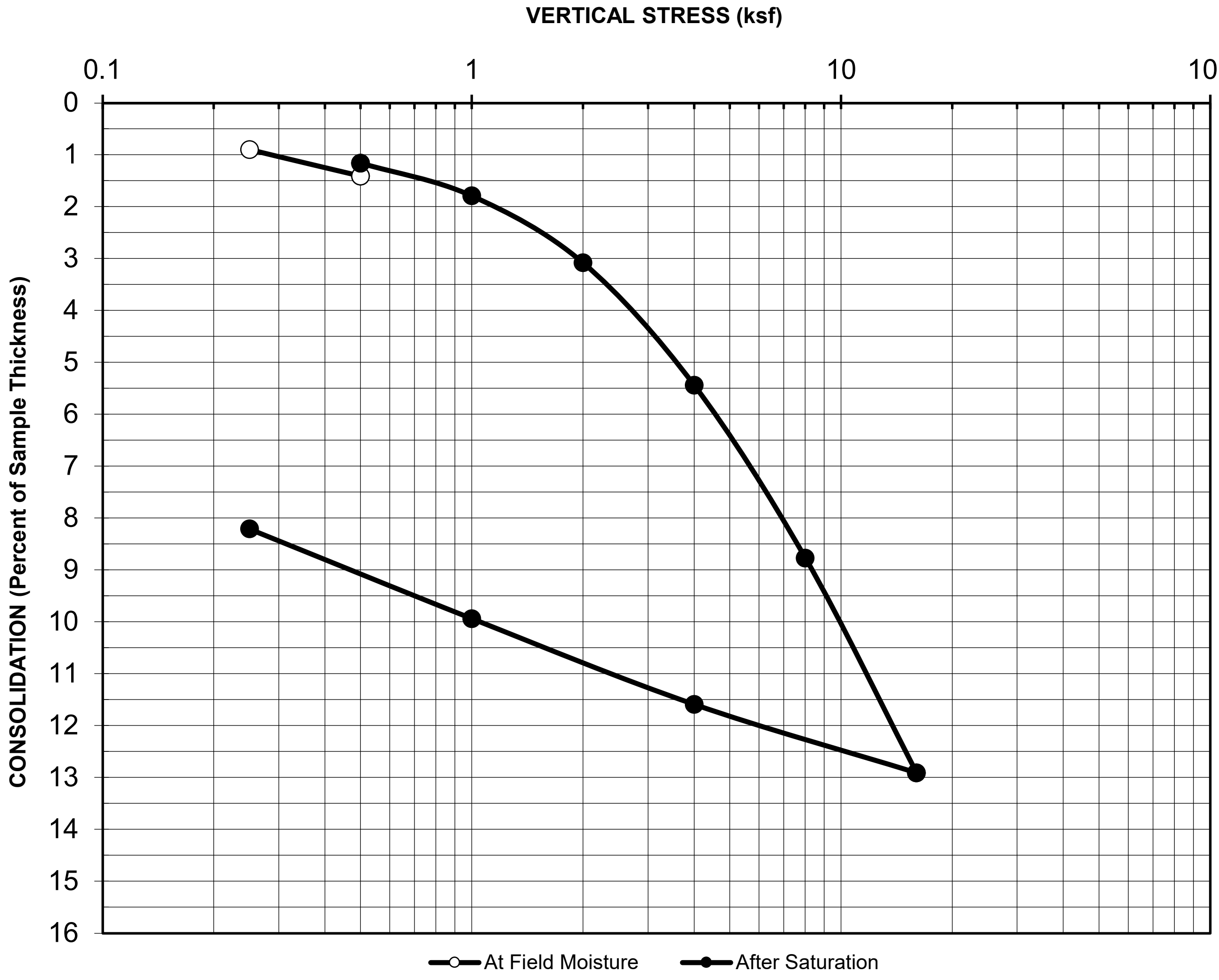
**Client:** Tetra Tech
**Project Name:** 2655 The Alameda Santa Clara, CA
**Project No.:** TET 22-235E
**Boring No.:** Tt-2
**Sample No.:** R-10 **Depth (ft):** 35-36.5
**Sample Type:** Mod. Cal.
**Soil Description:** Clay w/gravel
**Test Condition:** Inundated **Shear Type:** Regular

**Tested By:** LS **Date:** 04/27/22
**Computed By:** NR **Date:** 05/03/22
**Checked by:** AP **Date:** 05/04/22

| Wet Unit Weight (pcf) | Dry Unit Weight (pcf) | Initial Moisture Content (%) | Final Moisture Content (%) | Initial Degree Saturation (%) | Final Degree Saturation (%) | Normal Stress (ksf) | Peak Shear Stress (ksf) | Ultimate Shear Stress (ksf) |
|---|---|---|---|---|---|---|---|---|
| 130.4 | 108.2 | 20.6 | 20.7 | 99 | 100 | 5 | 3.409 | 3.181 |
| | | | | | | | | |
| | | | | | | | | |



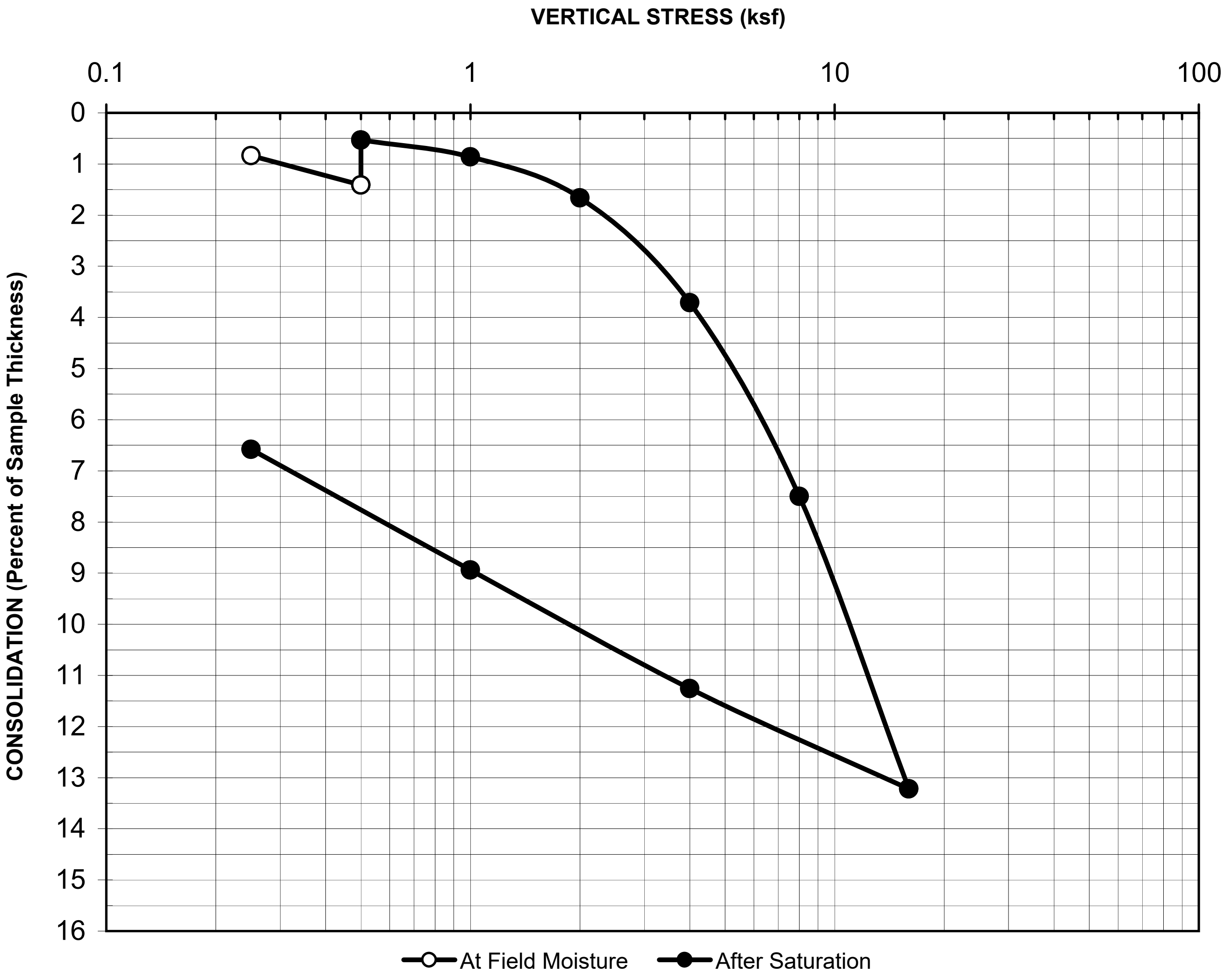




| | | | |
|---|---|---|---|
| Boring No. : | Tt-1 | Initial Dry Unit Weight (pcf): | 89.8 |
| Sample No.: | R-8 | Initial Moisture Content (%): | 32.2 |
| Depth (feet): | 20-21.5 | Final Moisture Content (%): | 32.1 |
| Sample Type: | Mod Cal | Assumed Specific Gravity: | 2.7 |
| Soil Description: | Clay | Initial Void Ratio: | 0.88 |
| Remarks: | Swell= 0.25% upon inundation | | |

**CONSOLIDATION CURVE**
**ASTM D 2435**

**Project Name:** 2655 The Alameda Santa Clara, CA
**Project No.:** TET 22-235E
**Date:** 4/22/2022
**AP No:** 22-0458 **Sheet No:** 1



Boring No. : Tt-2

Sample No.: R-4

Depth (feet): 12.5-14

Sample Type: Mod Cal

Soil Description: Lean Clay

Remarks: Swell= 0.88% upon inundation

Initial Dry Unit Weight (pcf): 83.9

Initial Moisture Content (%): 36.9

Final Moisture Content (%): 37.0

Assumed Specific Gravity: 2.7

Initial Void Ratio: 1.01

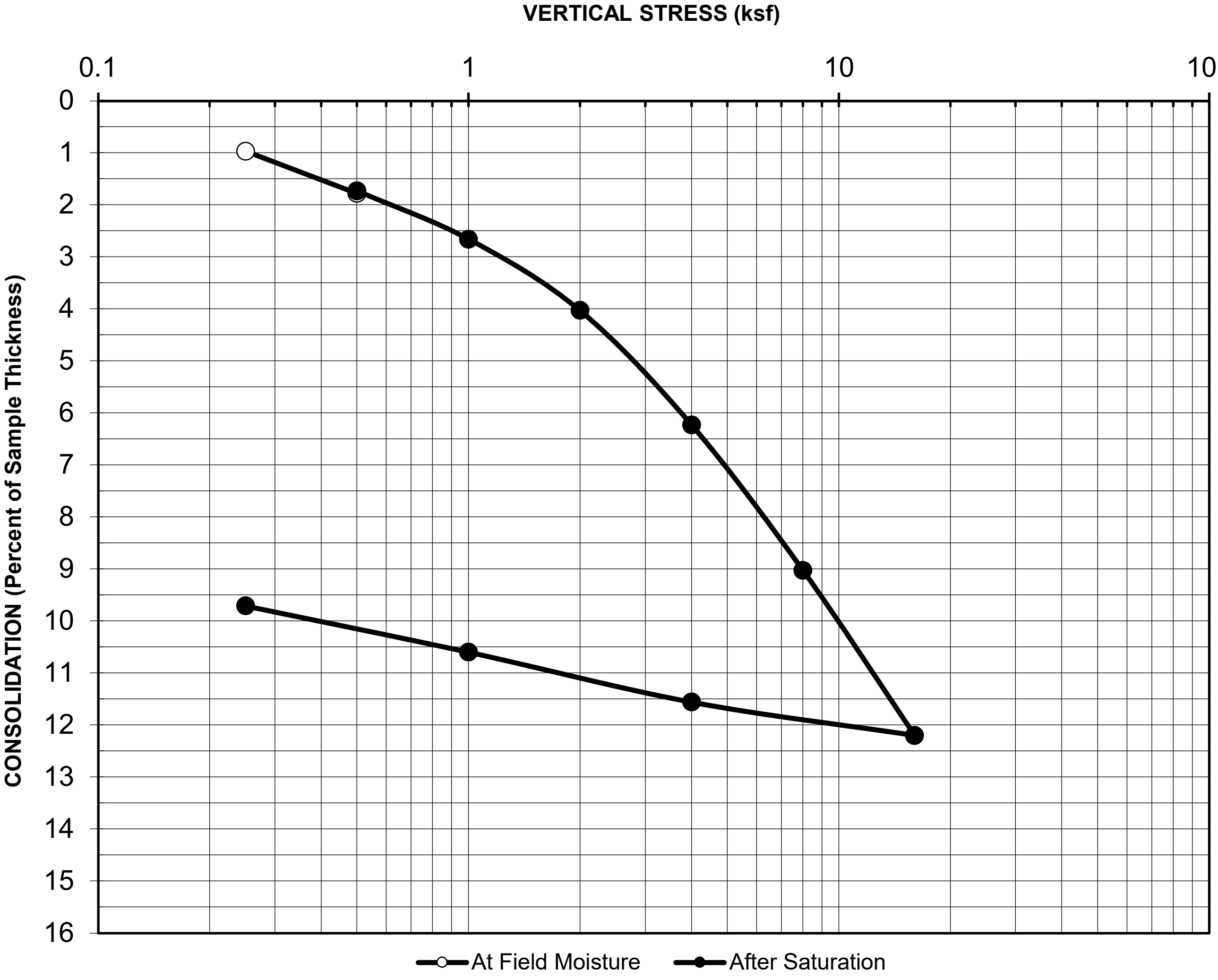
## CONSOLIDATION CURVE
## ASTM D 2435

**Project Name:** 2655 The Alameda Santa Clara, CA

**Project No.:** TET 22-235E

**Date:** 4/22/2022

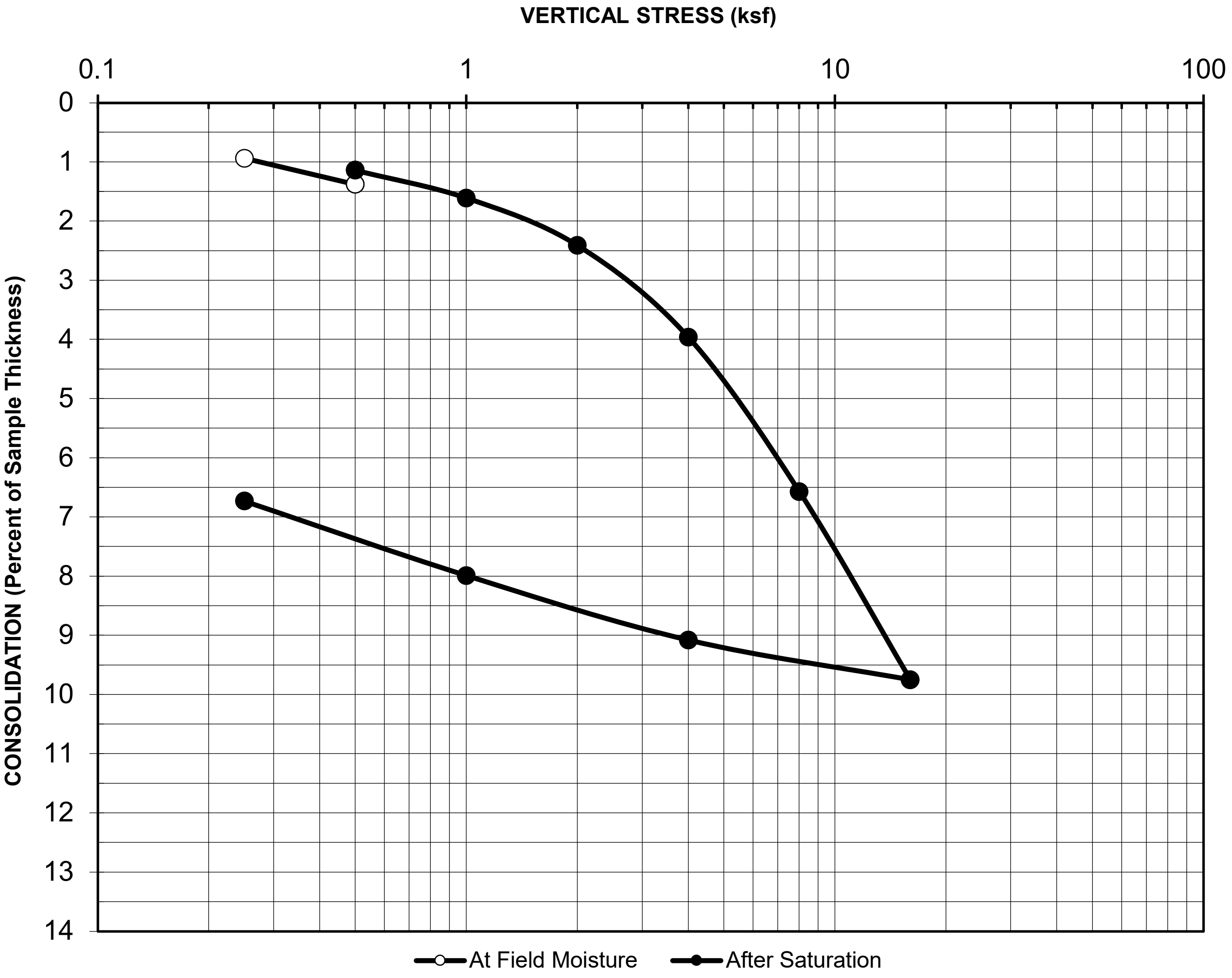
**AP No:** 22-0458 **Sheet No:** 1



| | | | |
|---|---|---|---|
| Boring No. : | Tt-2 | Initial Dry Unit Weight (pcf): | 98.0 |
| Sample No.: | R-6 | Initial Moisture Content (%): | 26.4 |
| Depth (feet): | 17.5-19 | Final Moisture Content (%): | 26.3 |
| Sample Type: | Mod Cal | Assumed Specific Gravity: | 2.7 |
| Soil Description: | Clay | Initial Void Ratio: | 0.72 |
| Remarks: | Swell= 0.05% upon inundation | | |

**CONSOLIDATION CURVE**
**ASTM D 2435**

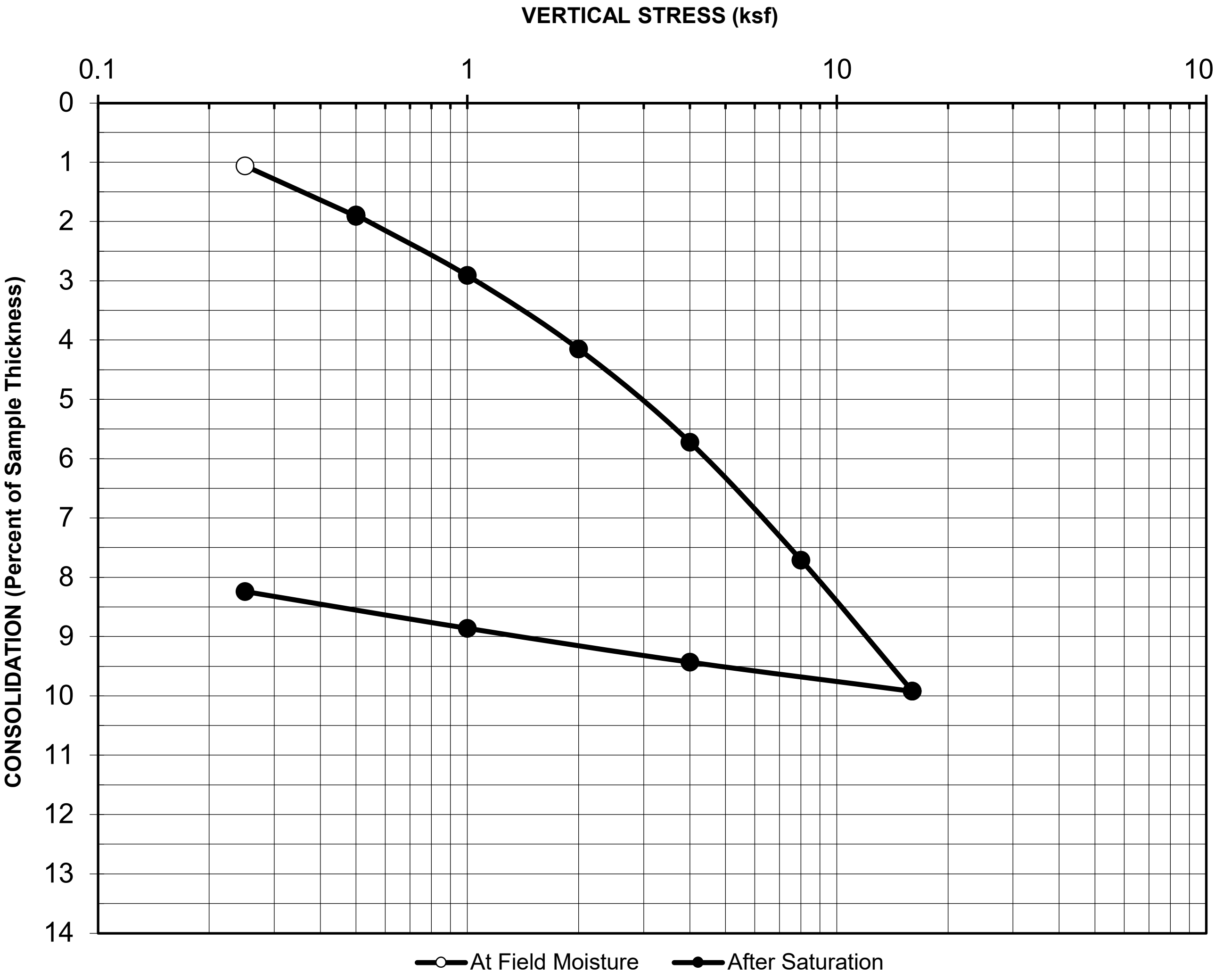
**Project Name:** 2655 The Alameda Santa Clara, CA
**Project No.:** TET 22-235E
**Date:** 4/22/2022
**AP No:** 22-0458 **Sheet No:** 1



| | | | |
|---|---|---|---|
| Boring No. : | Tt-2 | Initial Dry Unit Weight (pcf): | 102.2 |
| Sample No.: | R-8 | Initial Moisture Content (%): | 23.0 |
| Depth (feet): | 25-26.5 | Final Moisture Content (%): | 23.3 |
| Sample Type: | Mod Cal | Assumed Specific Gravity: | 2.7 |
| Soil Description: | Clay | Initial Void Ratio: | 0.65 |
| Remarks: | Swell= 0.24% upon inundation | | |

**CONSOLIDATION CURVE**
**ASTM D 2435**

**Project Name:** 2655 The Alameda Santa Clara, CA
**Project No.:** TET 22-235E
**Date:** 4/22/2022
**AP No:** 22-0458 **Sheet No:** 1



Boring No. : Tt-2

Sample No.: R-10

Depth (feet): 35-36.5

Sample Type: Mod Cal

Soil Description: Clay w/gravel

Remarks: Swell= 0.02% upon inundation

Initial Dry Unit Weight (pcf): 108.0

Initial Moisture Content (%): 20.6

Final Moisture Content (%): 20.5

Assumed Specific Gravity: 2.7

Initial Void Ratio: 0.56

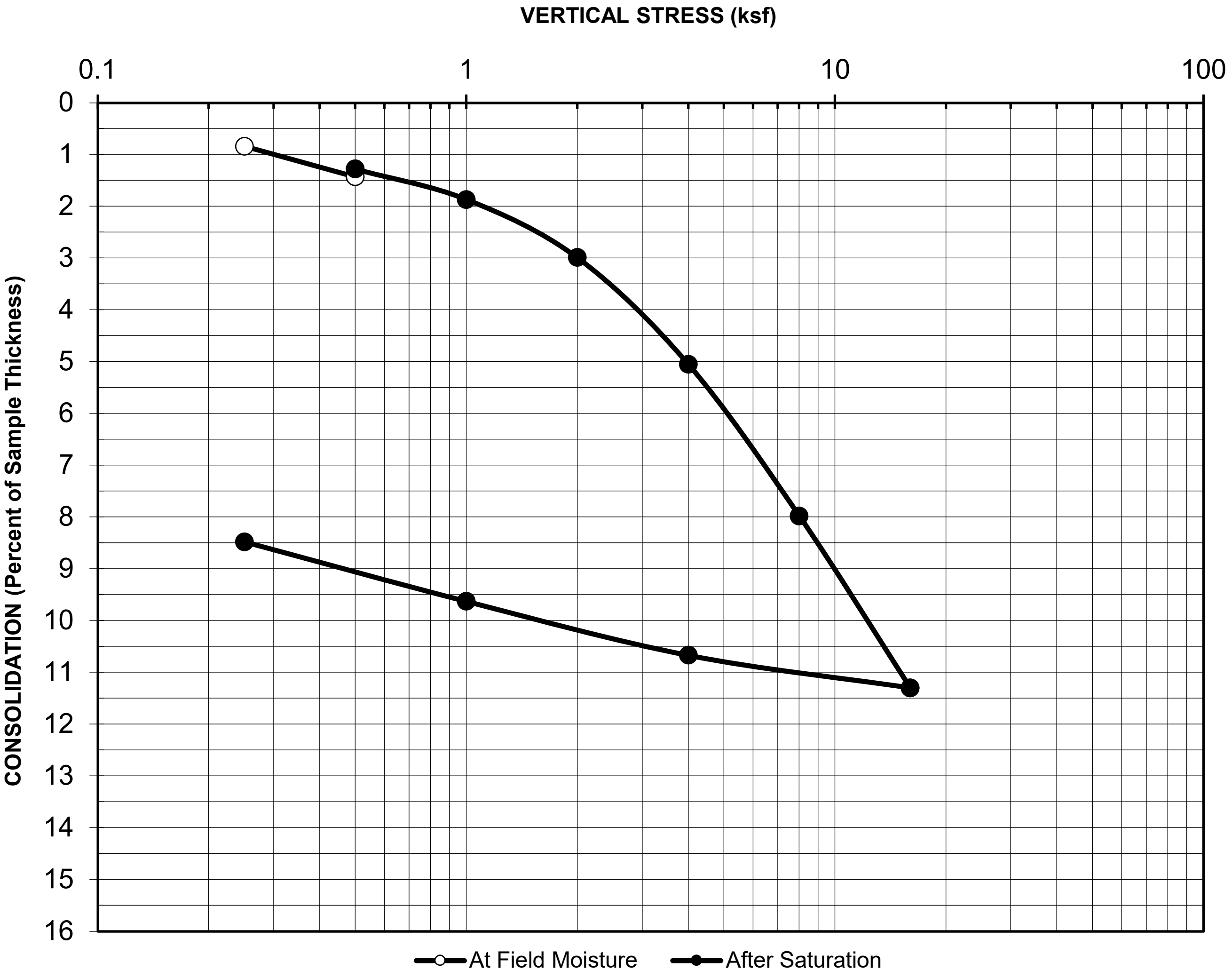
**CONSOLIDATION CURVE**
**ASTM D 2435**

**Project Name:** 2655 The Alameda Santa Clara, CA

**Project No.:** TET 22-235E

**Date:** 4/22/2022

**AP No:** 22-0458 **Sheet No:** 1



| | | | |
|---|---|---|---|
| Boring No. : | Tt-2 | Initial Dry Unit Weight (pcf): | 101.0 |
| Sample No.: | R-12 | Initial Moisture Content (%): | 24.5 |
| Depth (feet): | 45-46.5 | Final Moisture Content (%): | 24.4 |
| Sample Type: | Mod Cal | Assumed Specific Gravity: | 2.7 |
| Soil Description: | Lean Clay | Initial Void Ratio: | 0.67 |
| Remarks: | Swell= 0.15% upon inundation | | |

**CONSOLIDATION CURVE**
**ASTM D 2435**

**Project Name:** 2655 The Alameda Santa Clara, CA
**Project No.:** TET 22-235E
**Date:** 4/22/2022
**AP No:** 22-0458 **Sheet No:** 1

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t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

# EXPANSION INDEX TEST RESULTS

ASTM D 4829

Client Name: Tetra Tech
Project Name: 2655 The Alameda Santa Clara, CA
Project No.: TET 22-235E

AP Job No.: 22-0458
Date: 04/27/22

| Boring No. | Sample No. | Depth (ft) | Soil Description | Molded Dry Density (pcf) | Molded Moisture Content (%) | Init. Degree Saturation (%) | Measured Expansion Index | Corrected Expansion Index |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | | |
| Tt-1 | SK-1 | 0-5 | Clayey Sand | 114.2 | 8.4 | 48.0 | 5 | 4 |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |

## ASTM EXPANSION CLASSIFICATION

| Expansion Index | Classification |
|---|---|
| 0-20 | V. Low |
| 21-50 | Low |
| 51-90 | Medium |
| 91-130 | High |
| >130 | V. High |

TETRA TECH

# EXPANSION INDEX

## ASTM D4829

| | | | |
|---|---|---|---|
| **Job Name:** | Tt Alameda | **Date Sampled:** | 4/20/2022 |
| **Job Number:** | TET-22-235E | **Date Completed:** | 4/28/2022 |
| **Tested By:** | MG | **Sample Identification:** | Tt-1, SPT-7 |
| **Note:** | | **Sample Depth:** | 15-16.5ft |
| **Sample Description:** | Dark Olive Gray CL Native | | |

| SAMPLE PROCESSING | | |
|---|---|---|
| **Percentage Passing #4 Sieve** | | |
| Total Air Dry Weight | *grams* | 999.00 |
| Weight Retained on #4 | *grams* | 0.07 |
| *% Retained | *%* | 0.01 |
| ***% Passing # 4 Sieve** | *%* | **100.0** |

| SAMPLE DIMENSIONS | | |
|---|---|---|
| Sample Height | *inches* | 1.00 |
| Sample Diameter | *inches* | 4.01 |

| MOISTURE CALCS | | INITIAL | FINAL |
|---|---|---|---|
| Tare ID or # | | X33 | X31 |
| Wet Weight of Soil + Tare | *grams* | 256.8 | 436.5 |
| Dry Weight of Soil + Tare | *grams* | 228.3 | 338.9 |
| Weight of Tare | *grams* | 10.5 | 10.5 |
| * Moisture Weight | *grams* | 28.5 | 97.6 |
| * Weight of Dry Soil | *grams* | 217.8 | 328.4 |
| *** Moisture Content** | *%* | **13.1** | **29.7** |

| AFTER REMOLDING | | INITIAL | FINAL |
|---|---|---|---|
| Weight of Ring and Sample | *grams* | 595.5 | 595.5 |
| Weight of Ring | *grams* | 202.2 | 202.2 |
| *Remolded Wet Weight | *grams* | 393.3 | 393.3 |
| *Wet Density | *pcf* | 118.6 | 112.5 |
| *Dry Density | *pcf* | 104.9 | 86.7 |
| Assumed/Measured Specific Gravity | | 2.7 | 2.7 |
| ***Degree of Saturation** | *%* | **58.2** | **85.1** |

| EXPANSION INDEX TEST | | | |
|---|---|---|---|
| **DATE** | **TIME** | **DIAL** | **Δ H %** |
| 4/25/2022 | 10:10 AM | 0.0243 | 0.0 |
| 4/25/2022 | 10:20 AM | 0.0243 | 0.0 |
| | | 0.0344 | 1.0 |
| | | 0.0378 | 1.3 |
| | | 0.0582 | 3.4 |
| | 8:00 PM | 0.0783 | 5.4 |
| 4/26/2022 | 8:30 PM | 0.0783 | 5.4 |
| ***Total** | | **0.0783** | **5.4** |

| EI | Expansion Potential |
|---|---|
| 0 to 20 | Very Low |
| 21 to 50 | Low |
| 51 to 90 | Medium |
| 90 to 130 | High |
| >130 | Very High |

| | |
|---|---|
| **UNCORRECTED EXPANSION INDEX** | 54 |
| **CORRECTED EXPANSION INDEX**<br>For degrees of Saturation ≠ 50%, >40% and <60% | **60** |

**AP Engineering and Testing, Inc.**
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

# CORROSION TEST RESULTS

Client Name: Tetra Tech
Project Name: 2655 The Alameda Santa Clara, CA
Project No.: TET 22-235E

AP Job No.: 22-0458
Date: 04/27/22

| Boring No. | Sample No. | Depth (feet) | Soil Description | Minimum Resistivity (ohm-cm) | pH | Sulfate Content (ppm) | Chloride Content (ppm) |
|---|---|---|---|---|---|---|---|
| | | | | | | | |
| Tt-1 | SK-1 | 0-5 | Clayey Sand | 3,458 | 9.1 | 22 | 20 |
| Tt-2 | SPT-3 | 10-11.5 | Clay | 1,010 | 7.9 | 510 | 25 |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |

NOTES:

Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

# Appendix C

# Seismic Demand

# Alameda

## Latitude, Longitude: 37.346191, -121.937395

| | |
|---|---|
| **Date** | 5/5/2022, 5:13:42 PM |
| **Design Code Reference Document** | ASCE7-16 |
| **Risk Category** | II |
| **Site Class** | E - Soft Clay Soil |

| Type | Value | Description |
|---|---|---|
| $S_S$ | 1.5 | $MCE_R$ ground motion. (for 0.2 second period) |
| $S_1$ | 0.6 | $MCE_R$ ground motion. (for 1.0s period) |
| $S_{MS}$ | null -See Section 11.4.8 | Site-modified spectral acceleration value |
| $S_{M1}$ | null -See Section 11.4.8 | Site-modified spectral acceleration value |
| $S_{DS}$ | null -See Section 11.4.8 | Numeric seismic design value at 0.2 second SA |
| $S_{D1}$ | null -See Section 11.4.8 | Numeric seismic design value at 1.0 second SA |

| Type | Value | Description |
|---|---|---|
| SDC | null -See Section 11.4.8 | Seismic design category |
| $F_a$ | null -See Section 11.4.8 | Site amplification factor at 0.2 second |
| $F_v$ | null -See Section 11.4.8 | Site amplification factor at 1.0 second |
| PGA | 0.508 | $MCE_G$ peak ground acceleration |
| $F_{PGA}$ | 1.192 | Site amplification factor at PGA |
| $PGA_M$ | 0.605 | Site modified peak ground acceleration |
| $T_L$ | 12 | Long-period transition period in seconds |
| SsRT | 2.027 | Probabilistic risk-targeted ground motion. (0.2 second) |
| SsUH | 2.112 | Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration |
| SsD | 1.5 | Factored deterministic acceleration value. (0.2 second) |
| S1RT | 0.756 | Probabilistic risk-targeted ground motion. (1.0 second) |
| S1UH | 0.809 | Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration. |
| S1D | 0.6 | Factored deterministic acceleration value. (1.0 second) |
| PGAd | 0.508 | Factored deterministic acceleration value. (Peak Ground Acceleration) |
| $C_{RS}$ | 0.96 | Mapped value of the risk coefficient at short periods |
| $C_{R1}$ | 0.935 | Mapped value of the risk coefficient at a period of 1 s |

U.S. Geological Survey - Earthquake Hazards Program

# Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

## Input

Edition

Dynamic: Conterminous U.S. 2014 (up…

Latitude

Decimal degrees

37.346191

Longitude

Decimal degrees, negative values for western longitudes

-121.937395

Site Class

180 m/s (D/E boundary)

Spectral Period

Peak Ground Acceleration

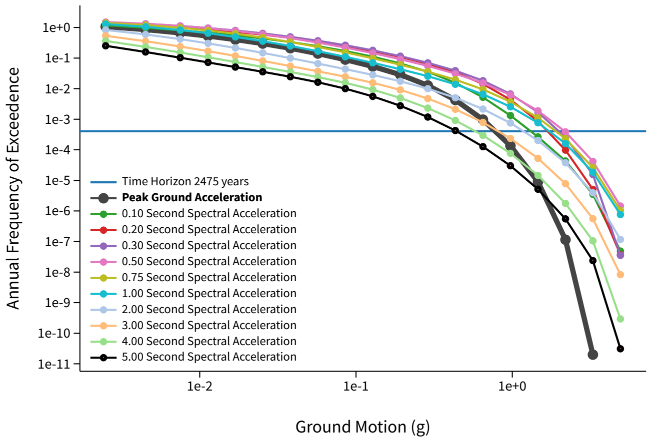
Time Horizon

Return period in years

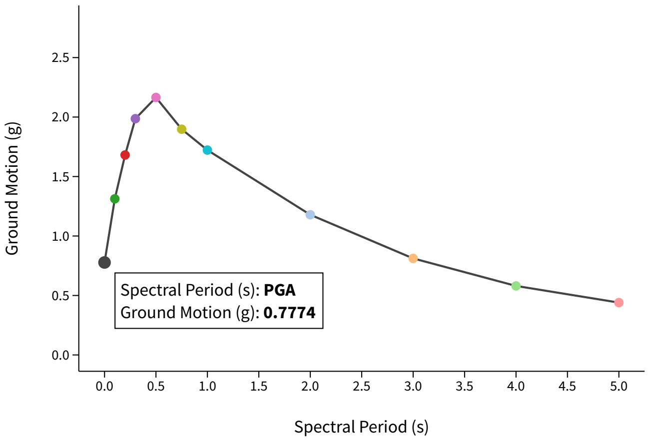
2475

## Hazard Curve

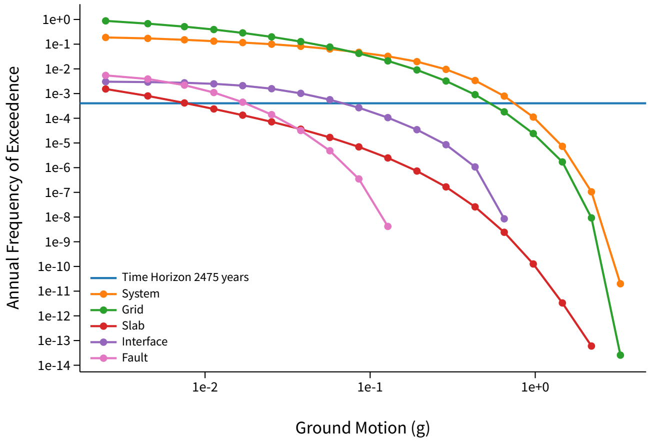
Hazard Curves



Uniform Hazard Response Spectrum



Component Curves for Peak Ground Acceleration

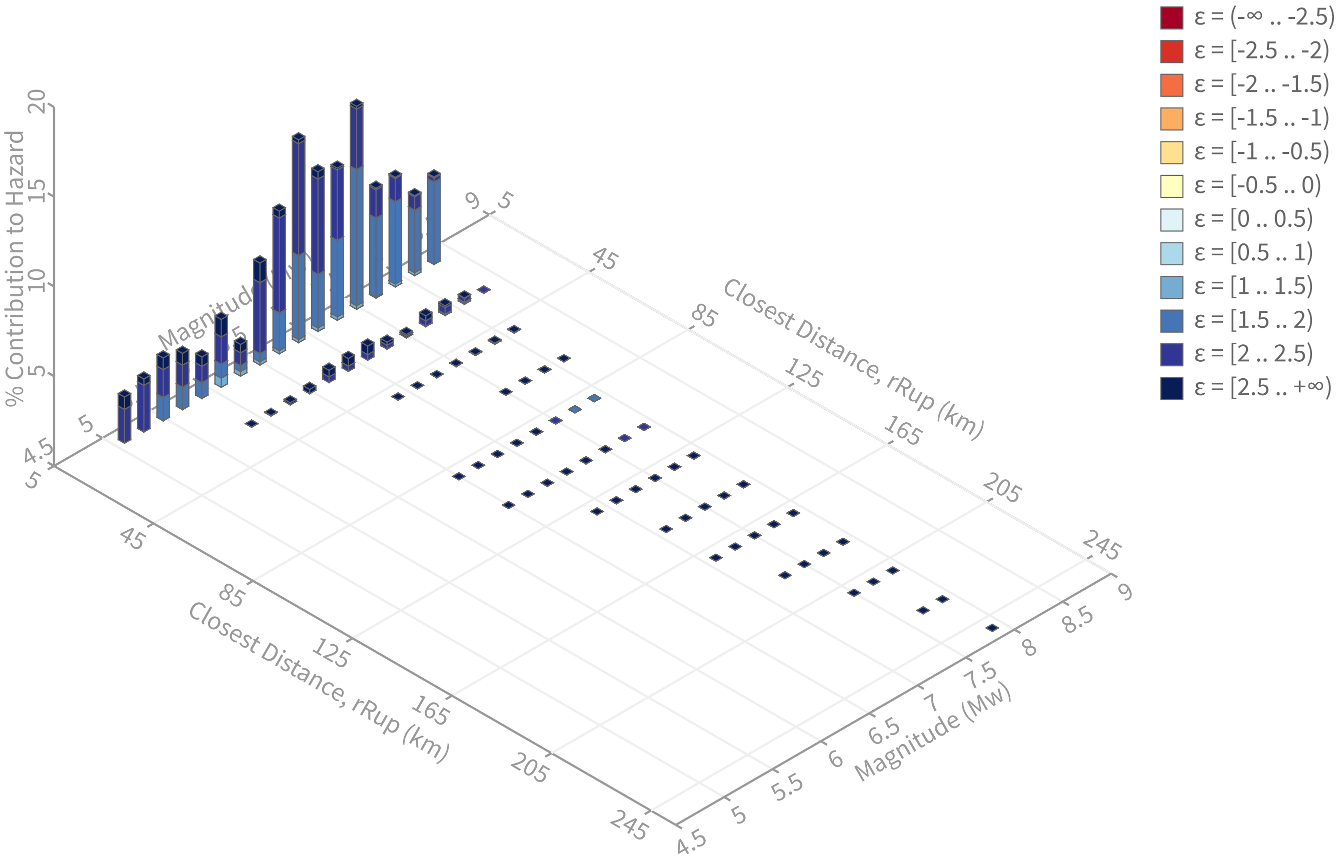


View Raw Data

## ^ Deaggregation

Component

Total



# Summary statistics for, Deaggregation: Total

## Deaggregation targets

**Return period:** 2475 yrs
**Exceedance rate:** 0.0004040404 $yr^{-1}$
**PGA ground motion:** 0.77738075 g

## Recovered targets

**Return period:** 3388.4697 yrs
**Exceedance rate:** 0.00029511848 $yr^{-1}$

## Totals

**Binned:** 100 %
**Residual:** 0 %
**Trace:** 0.06 %

## Mean (over all sources)

**m:** 6.98
**r:** 14.28 km
**$\varepsilon_0$:** 2.06 σ

## Mode (largest m-r bin)

**m:** 7.51
**r:** 14.34 km
**$\varepsilon_0$:** 1.9 σ
**Contribution:** 11.3 %

## Mode (largest m-r-$\varepsilon_0$ bin)

**m:** 7.52
**r:** 14.06 km
**$\varepsilon_0$:** 1.8 σ
**Contribution:** 7.59 %

## Discretization

**r:** min = 0.0, max = 1000.0, Δ = 20.0 km
**m:** min = 4.4, max = 9.4, Δ = 0.2
**ε:** min = -3.0, max = 3.0, Δ = 0.5 σ

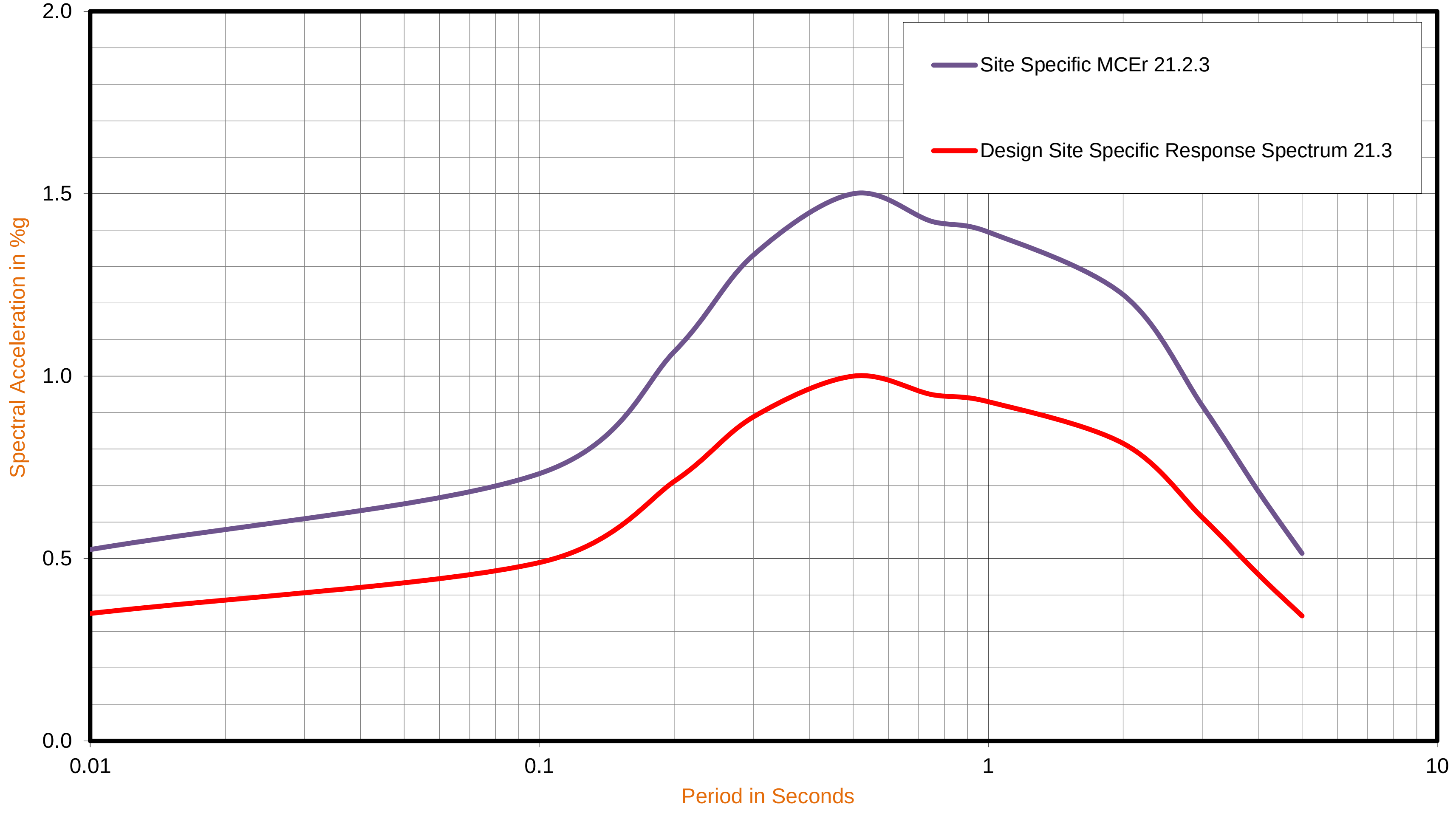
## Epsilon keys

**ε0:** [-∞ .. -2.5)
**ε1:** [-2.5 .. -2.0)
**ε2:** [-2.0 .. -1.5)
**ε3:** [-1.5 .. -1.0)
**ε4:** [-1.0 .. -0.5)
**ε5:** [-0.5 .. 0.0)
**ε6:** [0.0 .. 0.5)
**ε7:** [0.5 .. 1.0)
**ε8:** [1.0 .. 1.5)
**ε9:** [1.5 .. 2.0)
**ε10:** [2.0 .. 2.5)
**ε11:** [2.5 .. +∞]

## Deaggregation Contributors

| Source Set ↳ Source | Type | r | m | $\varepsilon_0$ | lon | lat | az | % |
|---|---|---|---|---|---|---|---|---|
| UC33brAvg_FM31 | System | | | | | | | 41.44 |
| Hayward (So) [0] | | 12.55 | 7.09 | 1.99 | 121.840°W | 37.428°N | 43.13 | 11.73 |
| San Andreas (Peninsula) [1] | | 16.82 | 7.88 | 1.89 | 122.081°W | 37.247°N | 229.27 | 10.41 |
| Calaveras (Central) [9] | | 15.39 | 7.23 | 2.04 | 121.787°W | 37.416°N | 59.68 | 5.29 |
| Hayward (So) [1] | | 12.72 | 6.80 | 2.11 | 121.856°W | 37.441°N | 34.34 | 2.08 |
| Hayward (So) extension [6] | | 15.14 | 6.48 | 2.39 | 121.781°W | 37.400°N | 66.66 | 2.03 |
| Monte Vista - Shannon [4] | | 11.08 | 7.21 | 1.87 | 122.012°W | 37.267°N | 216.70 | 1.54 |
| Hayward (So) [2] | | 16.19 | 6.88 | 2.25 | 121.906°W | 37.487°N | 10.14 | 1.39 |
| UC33brAvg_FM32 | System | | | | | | | 40.87 |
| Hayward (So) [0] | | 12.55 | 7.10 | 1.98 | 121.840°W | 37.428°N | 43.13 | 11.50 |
| San Andreas (Peninsula) [1] | | 16.82 | 7.88 | 1.89 | 122.081°W | 37.247°N | 229.27 | 10.70 |
| Calaveras (Central) [9] | | 15.39 | 7.22 | 2.04 | 121.787°W | 37.416°N | 59.68 | 5.39 |
| Hayward (So) [1] | | 12.72 | 6.80 | 2.11 | 121.856°W | 37.441°N | 34.34 | 2.17 |
| Hayward (So) extension [6] | | 15.14 | 6.48 | 2.39 | 121.781°W | 37.400°N | 66.66 | 1.48 |
| Hayward (So) [2] | | 16.19 | 6.87 | 2.26 | 121.906°W | 37.487°N | 10.14 | 1.44 |
| Monte Vista - Shannon [4] | | 11.08 | 7.18 | 1.88 | 122.012°W | 37.267°N | 216.70 | 1.43 |
| San Andreas (Santa Cruz Mts) [0] | | 19.86 | 7.24 | 2.29 | 122.002°W | 37.175°N | 196.74 | 1.03 |
| UC33brAvg_FM32 (opt) | Grid | | | | | | | 8.85 |
| PointSourceFinite: -121.937, 37.405 | | 8.29 | 5.55 | 2.06 | 121.937°W | 37.405°N | 0.00 | 2.43 |
| PointSourceFinite: -121.937, 37.405 | | 8.29 | 5.55 | 2.06 | 121.937°W | 37.405°N | 0.00 | 2.43 |
| UC33brAvg_FM31 (opt) | Grid | | | | | | | 8.84 |
| PointSourceFinite: -121.937, 37.405 | | 8.29 | 5.55 | 2.06 | 121.937°W | 37.405°N | 0.00 | 2.43 |
| PointSourceFinite: -121.937, 37.405 | | 8.29 | 5.55 | 2.06 | 121.937°W | 37.405°N | 0.00 | 2.43 |

## RISK TARGETED MAXIMUM CONSIDERED RESPONSE SPECTRA (MCEr)
## AND
## DESIGN REPONSE SPECTRA



## DESIGN ACCELERATION PARAMETERS 21.4

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| SDS section 11.4.5 | 1.200 | SDS from 0.9 *max above 0.2 s | 0.900 | SMS Section 11.4.3 | 1.800 | SMS Section 21.4 | 1.350 |
| SD1 Section 11.4.5 | 0.800 | SD1 (maxTsa) | 1.837 | SM1 Section 11.4.3 | 1.200 | SM1 Section 21.4 | 2.756 |

| **FINAL VALUES** | |
|---|---|
| SDS | **0.960** |
| SD1 | **1.837** |
| SMS | **1.440** |
| SM1 | **2.756** |

# Appendix D

# Liquefaction Evaluation

# Summary of Liquefaction and Earthquake-Induced Settlement Analysis

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Project: | TET 22-235E<br>The Alameda Feasibility Study | Boring: | **TT-1** | Engineer: | FC | Date: | 5/6/2022 |

| Liquefaction Evaluation Method | |
|---|---|
| Correction for fines content | Idriss & Boulang. 2008, 2014 |
| Correction for overburden $C_N$ | Idriss & Boulang. 2014 (N1)60cs |
| Cyclic resistance ratio of soil $CRR_{CS}$ | Idriss & Boulang. 2004, 2014 |
| Correction for overburden Kσ | Idriss & Boulang. 2008, 2014 |
| Stress reduction factor $r_D$ | Idriss 1999, I&B 2008,2014 |
| Magnitude scaling factor MSF | Idriss & Boulang. 2014 |
| Dry settlement | Pradel, 1998a,b |
| Liquefaction settlement | Yoshimine et al., 2006 – w/ calibration |

| Liquefaction Analysis Statistics | |
|---|---|
| Total thickness of evaluated profile | 32.5 feet |
| Profile thickness susceptible to liquefaction | 2.5 feet |
| Number of evaluated intervals | 9 |
| Number of potentially liquefiable intervals | 1 |
| **Average Factor of Safety** of sandy intervals | **0.18** |
| Dry sand settlement | 0.00 inches |
| Liquefaction settlement | 1.00 inches |
| **Total earthquake-induced settlement** | **1.00 inches** |

| | |
|---|---|
| Liquefaction behavior Plasticity Index threshold | less or equal to 7 |
| Saturated settlement threshold | less or equal to 70% fines |
| Dry settlement threshold | less or equal to 25% fines |
| Cyclic softening Plasticity Index threshold | greater or equal to 18 |

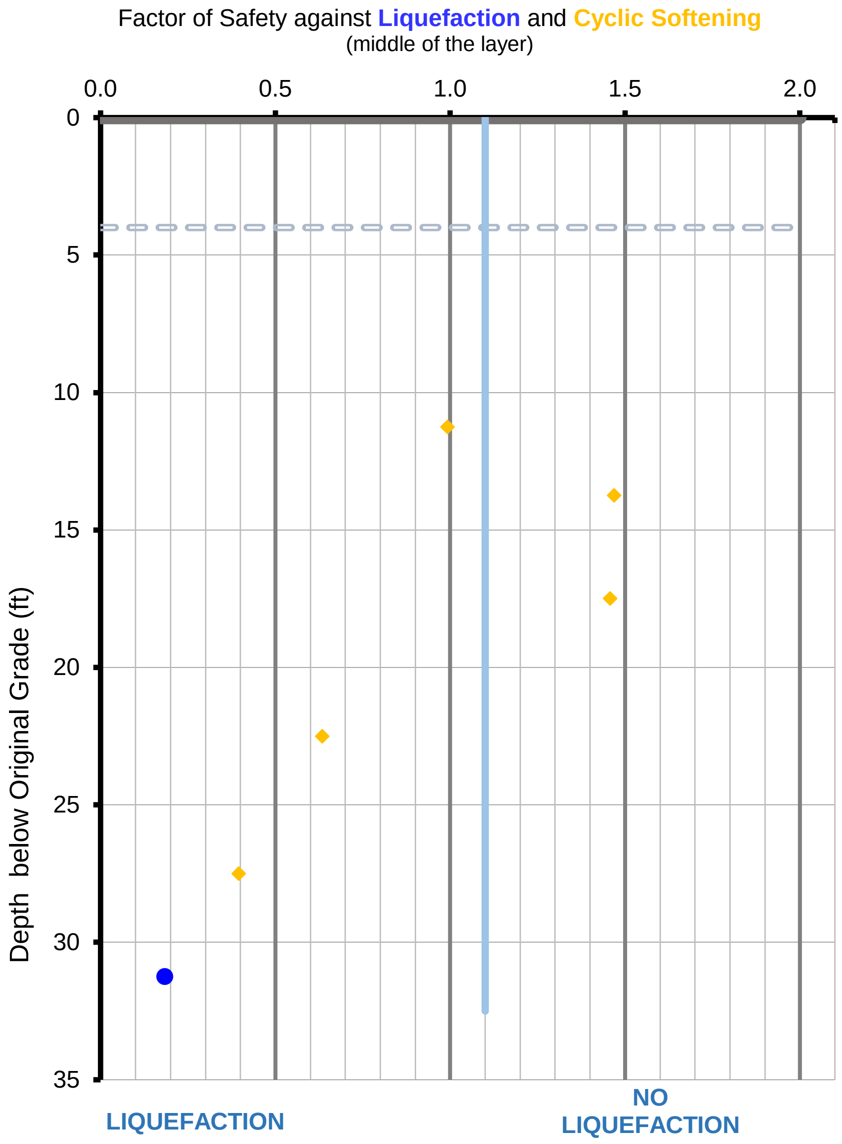
| Profile | | | Earthquake loading | |
|---|---|---|---|---|
| In-Situ Groundwater depth | 30.00 | feet | M | 7.51 |
| DESIGN Groundwater depth | 4.00 | feet | PGA | 0.605 |
| DESIGN Excavation depth | 0.00 | feet | | |
| DESIGN Surcharge (fill) | 0.00 | feet | | |

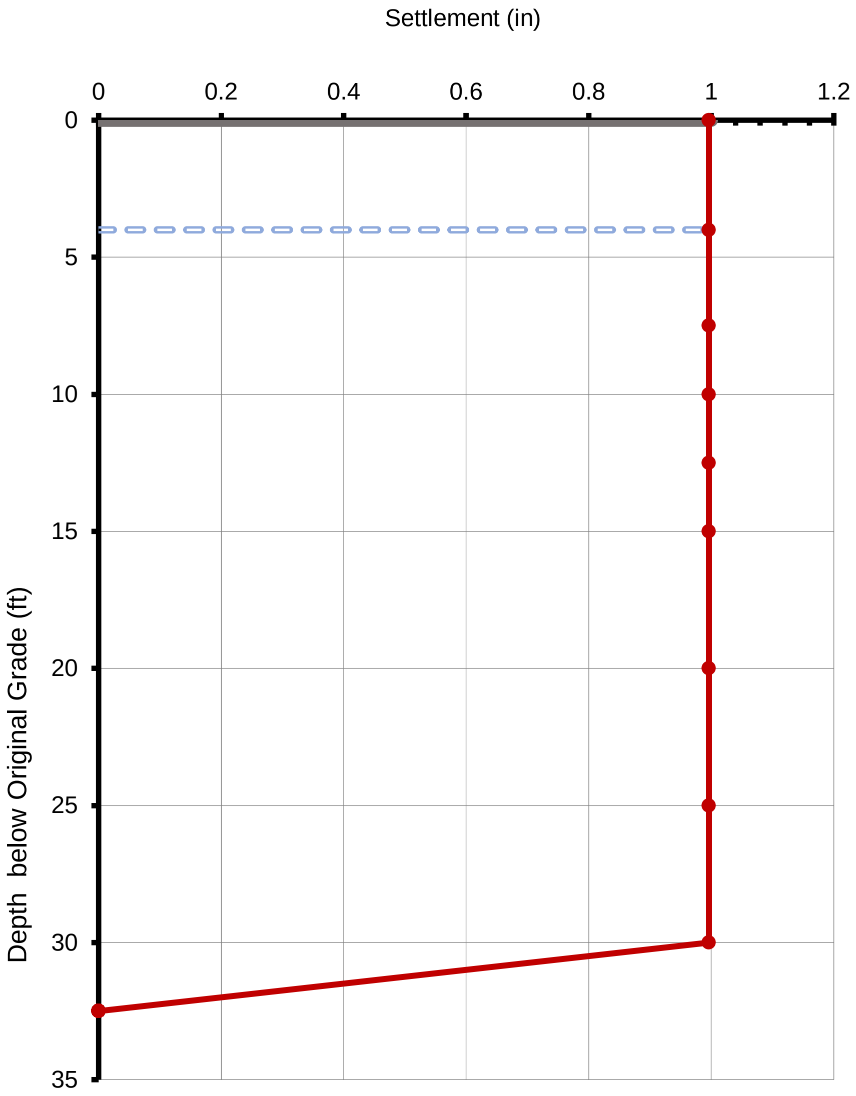
| Checks | |
|---|---|
| Groundwater depth check | OK |
| Design groundwater/excavation depth check | OK |
| Fines correction method compatibility | OK |
| Idris & Boulanger, 2004 method for $C_N$ | not used |
| Cetin 2009 settlement method | not used |

*Version v2 2022-03*

| Depth to Layer Top | Layer Thickness | Total Unit Weight | | Fines % | Plasticity Index | Considered Blowcounts | | | Factor of Safety | | Liquefaction potential rationale | Layer Settlement | Cumulative Settlement |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | In-situ | Design | | | SPT-N | N1,60 | N1,60,cs | Liquefaction | Cyclic softening | | | |
| feet | feet | pcf | pcf | % | – | bpf | bpf | bpf | | | – | in | in |
| 0 | 4 | 112.7 | 120.0 | 35 | 10 | 8.0 | 15.3 | 20.8 | – | – | - no groundwater | 0.00 | 1.00 |
| 4 | 3.5 | 112.7 | 120.0 | 80 | 10 | 8.0 | 15.0 | 20.5 | – | – | - clay-like behaviour | 0.00 | 1.00 |
| 7.5 | 2.5 | 112.7 | 120.0 | 80 | 10 | 11.2 | 17.2 | 22.8 | – | – | - clay-like behaviour | 0.00 | 1.00 |
| 10 | 2.5 | 112.7 | 120.0 | 91 | 24 | 3.0 | 5.0 | 10.5 | – | 0.99 | - clay-like behaviour | 0.00 | 1.00 |
| 12.5 | 2.5 | 112.7 | 120.0 | 91 | 24 | 6.3 | 9.3 | 14.8 | – | 1.47 | - clay-like behaviour | 0.00 | 1.00 |
| 15 | 5 | 121.1 | 120.0 | 91 | 24 | 8.0 | 11.7 | 17.2 | – | 1.46 | - clay-like behaviour | 0.00 | 1.00 |
| 20 | 5 | 118.7 | 120.0 | 81 | 24 | 4.9 | 6.2 | 11.8 | – | 0.63 | - clay-like behaviour | 0.00 | 1.00 |
| 25 | 5 | 123.6 | 120.0 | 91 | 24 | 4.0 | 4.5 | 10.0 | – | 0.39 | - clay-like behaviour | 0.00 | 1.00 |
| 30 | 2.5 | 123.6 | 120.0 | 35 | n/plastic | 4.2 | 4.7 | 10.2 | 0.18 | – | - liquefieable (FS < 1.1) | 1.00 | 1.00 |

| | | |
|---|---|---|
| **TT-1** | Design excavation depth | 0.00 feet |
| | Design groundwater depth | 4.00 feet |





# Summary of Liquefaction and Earthquake-Induced Settlement Analysis

| Project: | TET 22-235E<br>The Alameda Feasibility Study | Boring: | **TT-2** | Engineer: | FC | Date: | 5/6/2022 |
|---|---|---|---|---|---|---|---|

| Liquefaction Evaluation Method | |
|---|---|
| Correction for fines content | Idriss & Boulang. 2008, 2014 |
| Correction for overburden $C_N$ | Idriss & Boulang. 2014 (N1)60cs |
| Cyclic resistance ratio of soil $CRR_{CS}$ | Idriss & Boulang. 2004, 2014 |
| Correction for overburden Kσ | Idriss & Boulang. 2008, 2014 |
| Stress reduction factor $r_D$ | Idriss 1999, I&B 2008,2014 |
| Magnitude scaling factor MSF | Idriss & Boulang. 2014 |
| Dry settlement | Pradel, 1998a,b |
| Liquefaction settlement | Yoshimine et al., 2006 – w/ calibration |

| Liquefaction Analysis Statistics | |
|---|---|
| Total thickness of evaluated profile | 55 feet |
| Profile thickness susceptible to liquefaction | 7.5 feet |
| Number of evaluated intervals | 13 |
| Number of potentially liquefiable intervals | 2 |
| **Average Factor of Safety** of sandy intervals | **0.45** |
| Dry sand settlement | 0.00 inches |
| Liquefaction settlement | 1.46 inches |
| **Total earthquake-induced settlement** | **1.46 inches** |

| | |
|---|---|
| Liquefaction behavior Plasticity Index threshold | less or equal to 7 |
| Saturated settlement threshold | less or equal to 70% fines |
| Dry settlement threshold | less or equal to 25% fines |
| Cyclic softening Plasticity Index threshold | greater or equal to 18 |

| Profile | | Earthquake loading | |
|---|---|---|---|
| In-Situ Groundwater depth | 30.00 feet | M | 7.51 |
| DESIGN Groundwater depth | 4.00 feet | PGA | 0.605 |
| DESIGN Excavation depth | 0.00 feet | | |
| DESIGN Surcharge (fill) | 0.00 feet | | |

| Checks | |
|---|---|
| Groundwater depth check | OK |
| Design groundwater/excavation depth check | OK |
| Fines correction method compatibility | OK |
| Idris & Boulanger, 2004 method for $C_N$ | not used |
| Cetin 2009 settlement method | not used |

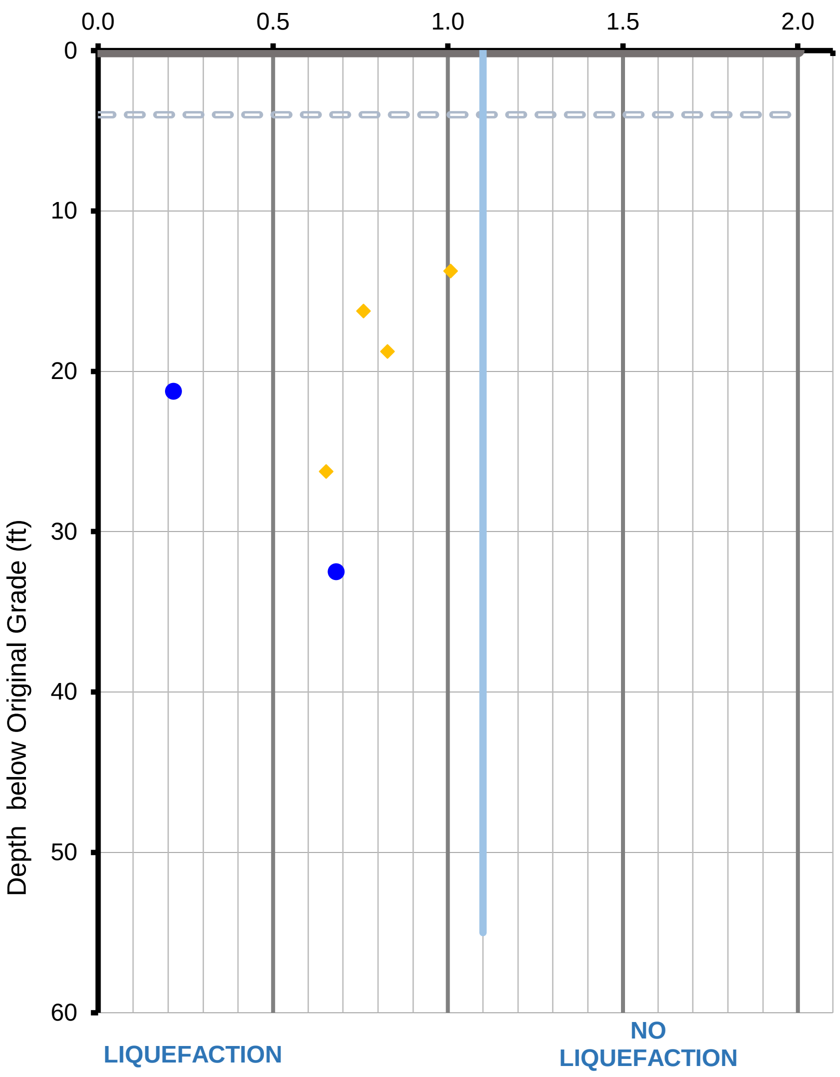
*Version v2 2022-03*

| Depth to Layer Top | Layer Thickness | Total Unit Weight | | Fines % | Plasticity Index | Considered Blowcounts | | | Factor of Safety | | Liquefaction potential rationale | Layer Settlement | Cumulative Settlement |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | In-situ | Design | | | SPT-N | N1,60 | N1,60,cs | Liquefaction | Cyclic softening | | | |
| feet | feet | pcf | pcf | % | – | bpf | bpf | bpf | | | – | in | in |
| 0 | 4 | 112.7 | 120.0 | 35 | n/plastic | 16.1 | 30.8 | 36.3 | – | – | - no groundwater | 0.00 | 1.46 |
| 4 | 3.5 | 112.7 | 120.0 | 91 | 10 | 16.1 | 27.1 | 32.6 | – | – | - clay-like behaviour | 0.00 | 1.46 |
| 7.5 | 5 | 112.7 | 120.0 | 91 | 10 | 4.0 | 6.2 | 11.7 | – | – | - clay-like behaviour | 0.00 | 1.46 |
| 12.5 | 2.5 | 112.7 | 120.0 | 91 | 26 | 4.2 | 6.3 | 11.8 | – | 1.01 | - clay-like behaviour | 0.00 | 1.46 |
| 15 | 2.5 | 112.7 | 120.0 | 91 | 26 | 4.0 | 5.5 | 11.0 | – | 0.76 | - clay-like behaviour | 0.00 | 1.46 |
| 17.5 | 2.5 | 121.1 | 120.0 | 91 | 26 | 4.9 | 6.9 | 12.4 | – | 0.83 | - clay-like behaviour | 0.00 | 1.46 |
| 20 | 2.5 | 118.7 | 120.0 | 70 | 5 | 5.0 | 6.6 | 12.2 | 0.22 | – | - liquefieable (FS < 1.1) | 0.89 | 1.46 |
| 22.5 | 7.5 | 123.6 | 120.0 | 70 | 26 | 6.3 | 7.4 | 13.0 | – | 0.65 | - clay-like behaviour | 0.00 | 0.56 |
| 30 | 5 | 123.6 | 120.0 | 5 | n/plastic | 24.0 | 29.3 | 29.3 | 0.68 | – | - liquefieable (FS < 1.1) | 0.56 | 0.56 |
| 35 | 5 | 130.5 | 120.0 | 65 | 8 | 4.9 | 5.2 | 10.8 | – | – | - clay-like behaviour | 0.00 | 0.00 |
| 40 | 5 | 130.5 | 120.0 | 65 | 8 | 5.0 | 5.1 | 10.7 | – | – | - clay-like behaviour | 0.00 | 0.00 |
| 45 | 5 | 125.7 | 120.0 | 65 | 8 | 7.0 | 7.0 | 12.6 | – | – | - clay-like behaviour | 0.00 | 0.00 |
| 50 | 5 | 125.7 | 120.0 | 65 | 8 | 8.0 | 7.8 | 13.4 | – | – | - clay-like behaviour | 0.00 | 0.00 |

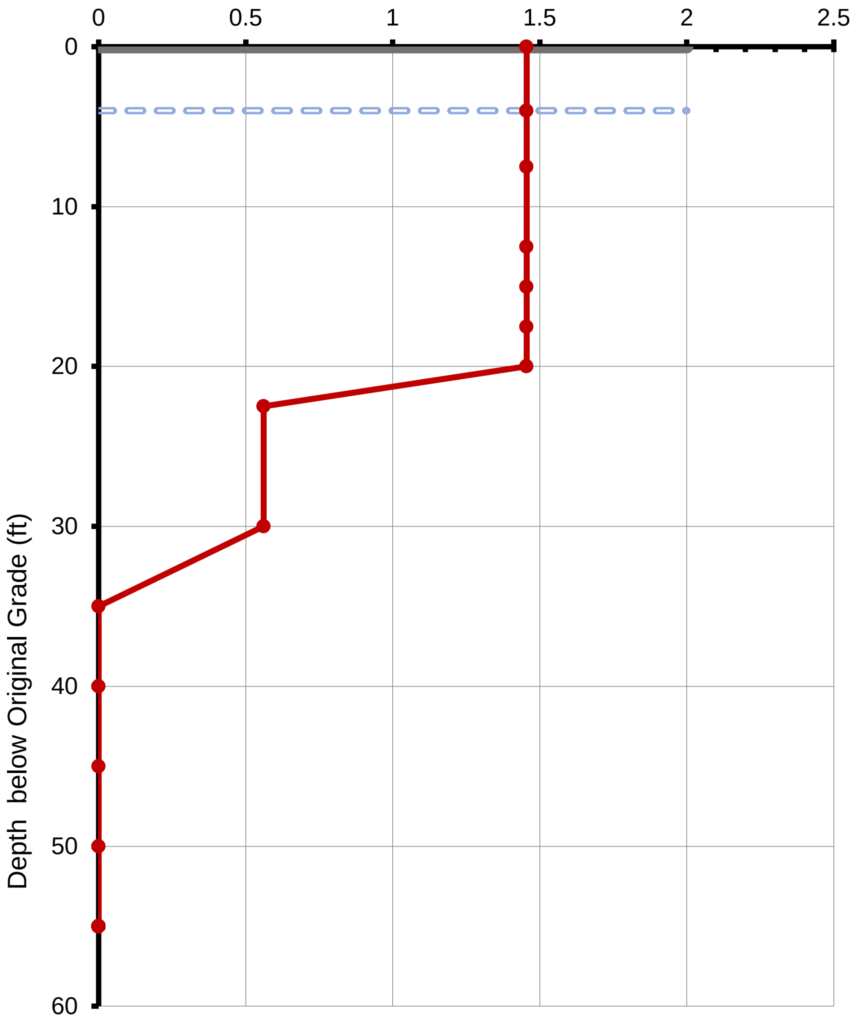
**TT-2**

Design excavation depth 0.00 feet
Design groundwater depth 4.00 feet

Factor of Safety against **Liquefaction** and **Cyclic Softening**
(middle of the layer)



Settlement (in)



## Sensitivity Analyses of Fine-Grained Materials

| Sample No. | Sample Depth (ft) | Groundwater Depth (ft) | Assumed Total Unit Weight above GWT (pcf) | assumed Total Unit Weight below GWT (pcf) | USCS Classification | Sample Moisture Content (%) | Dry Unit Weight at this depth (pcf) | Liquid Limit | Plastic Limit | Assumed Specific Gravity | Plasticity Index | Total Unit Weight (pcf) | Saturated Moisture Content (%) | Liquidity Index | Approximate Effective Vertical Stress (atm) | Sensitivity from Peck, Mesri (1996) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Tt-1 SPT-7 | 15 | 4 | 115 | 118 | CL | 34 | 84 | 42 | 18 | 2.65 | 24 | 112.6 | 36.5 | 0.77 | 0.51 | 5.50 |
| TT-2 SPT-5 | 7 | 4 | 121 | 121 | CL | 26 | 96 | 50 | 24 | 2.65 | 26 | 121.0 | 27.3 | 0.13 | 0.31 | 1.94 |
|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |