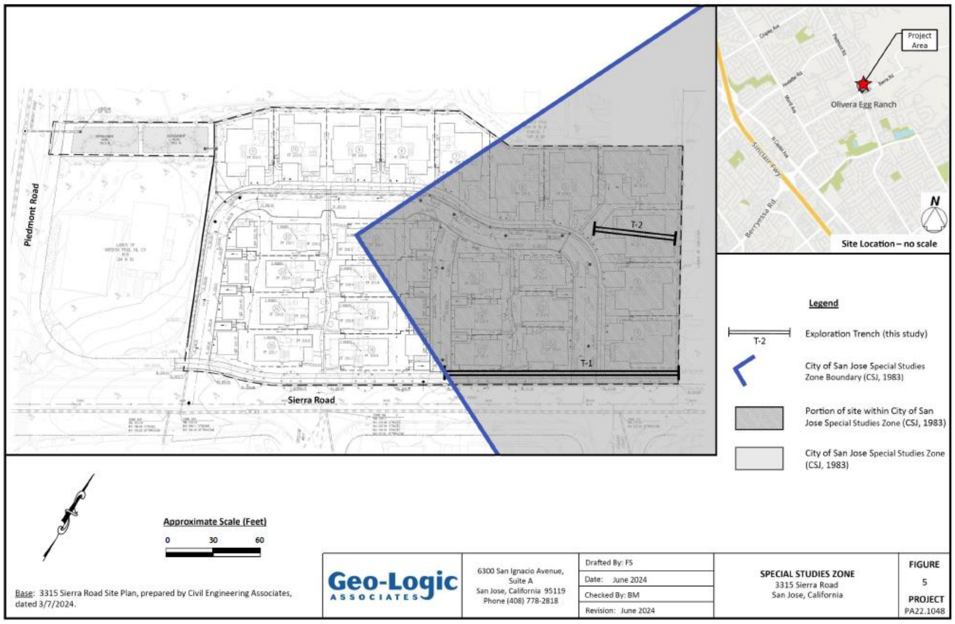
GEOHAZARDS EVALUATION
RESIDENTIAL DEVELOPMENT



3315 SIERRA ROAD
SAN JOSE, CALIFORNIA

JULY 10, 2024
PROJECT PA22.1048.00

SUBMITTED TO:

Robson Homes
2185 The Alameda, Suite 150
San Jose, CA 95126

PREPARED BY:

Geo-Logic Associates
6300 San Ignacio Avenue, Suite A
San Jose, California 95119
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GEOHAZARDS EVALUATION
PROPOSED RESIDENTIAL DEVELOPMENT
3315 SIERRA ROAD
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Table of Contents

1 INTRODUCTION

This report presents the results of our geohazards evaluation for a proposed residential development at 3315 Sierra Road, San Jose, California. The property, with an area of about 2.71 acres, is referred as the “property,” “site,” or “project site” in this report. The Assessor Parcel Number (APN) of the property is 565-10-067. The approximate location of the project site is shown on the Site Plan (Existing Conditions) included as Figure 1 of this report. The base map for Figure 5 shows a layout of the proposed development.

Previously, Geo-Logic Associates (GLA) submitted a Geotechnical Study report for the proposed residential development at this site dated March 28, 2023. Based on a Preliminary Geologic Hazard Review by the City Geologist, Mike Shimamoto (date March 22, 2024), that report did not satisfy the required geologic/seismic hazard evaluation. As such, this Geohazards Evaluation has been conducted by our Certified Engineering Geologist and is specifically designed to address the City requirements including: 1) surface fault rupture beneath the subject parcel; and 2) static and earthquake induced displacement of the Penitencia Creek Landslide Complex (PCLC).

This report presents our findings, conclusions, and geologic recommendations for design and construction of the project. These findings, conclusions, and recommendations are based on information collected during this study and should not be extrapolated to other areas or used for other projects without our review.

1.1 Project Description

The project site is currently occupied by a business with several buildings, a loading dock, a paved parking lot, and several residences. The proposed residential development will include single-family units. Associated improvements will include underground utilities, landscaping, exterior flatwork, driveways, and an on-site street. Site grading will be limited to cuts and fills of generally 1 to 3 feet deep because of the flat-lying topography across the site. The proposed residential buildings will be three-story, wood-framed structures. No basements are planned for the residential units. Retaining walls, if needed, will be free-standing landscaping walls generally a few feet high and not a part of the proposed buildings.

The above project descriptions are based on information provided to us. If the actual project differs from those described above, Geo-Logic Associates (GLA) should be contacted to review our findings, conclusions, and recommendations, and to present any necessary modifications to address the different project development schemes.

1.2 Information Provided

For this study, our client provided us with the following information.

- A drawing titled “3315 Sierra Road Conceptual Grading and Drainage Plan,” prepared by Civil Engineering Associates, dated March 27, 2024.

- Drawings (2 pages) titled "ALTA/NSPS Land Title Survey, 3315 Sierra Road," prepared by Civil Engineering Associates, dated January 18, 2022.

1.3 Purpose and Scope of Services

The purpose of this geohazards evaluation was to explore the surface and subsurface conditions at the project site and to assess the potential for the existence and possible geologic hazard on or adjacent to the site that could possibly affect the proposed project.

The following work was performed in accordance with our proposal date April 17, 2024.

- Field preparation including coordination with our excavation subcontractor, Pearson Exploration of Sebastopol, CA.
- Utility field marking for Underground Services Alert (USA) and clearance of on-site utilities with our private utility location subconsultant, NorCal Underground Locating.
- Review of available pertinent geologic literature, maps and Google Earth aerial photographs.
- Geologic reconnaissance mapping of landslide and fault related geomorphic or distress features evident at the ground surface in the vicinity of the project.
- Submitting a City of San Jose Geologic Testing Form to obtain an environmental clearance for the geologic testing and consult with the City Geologist regarding the proposed exploration plan.
- Localized saw-cutting of asphalt pavement in trench locations.
- Providing and installing temporary hydraulic shoring in accordance with soil classification and Cal-OSHA standards (Competent Person/Pearson Exploration).
- Providing safety fencing and trench plate covers.
- Performing subsurface exploration by means of excavating and logging two trenches by GLA professional geologists; one 216 feet in length and another 75 feet in length and 8 feet deep.
- Selective sampling of soils for possible radiocarbon age-dating.
- Accelerator Mass Spectrometry (AMS) bulk radiocarbon age dating on one soil sample by Beta Analytic laboratories in Florida.
- Compaction testing of native soils trench backfill to a minimum of 90% relative compaction (ASTM D-1557).
- Analysis of the collected data and characterization of potential geologic hazards at this site.
- Communications with City Geologist.

- Project management.
- Preparation of this Geohazards Evaluation report.

2 GEOLOGIC LITERATURE REVIEW

2.1 Regional Geology

The site is located along the base of the foothills in northeastern San Jose, within the Coast Range Geomorphic Province of Northern California. This province is generally characterized by northwest-trending mountain ranges and intervening valleys, which are a reflection of the dominant northwest structural trend of the bedrock in the region. The basement rock in this portion of the province consists predominantly of the Franciscan Complex; a subduction complex of diverse groups of igneous, sedimentary and metamorphic rocks of Cretaceous to Upper Jurassic age (65 to 160 million years old) and to the east, the Coast Range Ophiolite and Great Valley Complex, an Upper to Middle Jurassic age (approximately 145 to 175 million years old) volcanic ophiolite sequence with associated Lower Cretaceous to Upper Jurassic (approximately 100 to 160 million years old) sedimentary rocks. The Coast Range Ophiolite and Great Valley Complex were tectonically juxtaposed with the Franciscan Complex (most likely during subduction accretion of the Franciscan Complex), and these ancient fault boundaries are truncated by a modern right-lateral fault system that includes the San Andreas, Calaveras and Hayward-Rodgers Creek fault zones. Located approximately 16 miles southwest of the site (CGS, 2021), the San Andreas fault defines the westernmost boundary of the local bedrock. In the site vicinity, the Great Valley Sequence, Coast Range Ophiolite and Franciscan Complex are unconformably overlain by Tertiary age (approximately 2.6 to 65 million years old) continental and marine sedimentary and volcanic rocks. These Tertiary age rocks are locally overlain by younger Quaternary (approximately 2.6 million years old to present day) alluvial, colluvial and landslide deposits.

2.2 Site Geology

The site has been mapped by Dibblee (2005), who indicates the study site is underlain by Holocene (<11,700 year-old) alluvium, which generally consists of unconsolidated to semi-consolidated gravels, sand, silts and clays. Dibblee (2005) does not show the presence of faults or landslide deposits on the site proper. Dibble (2005) as well as others, however, do show landslide deposits and large, active landslide complexes on the hillslope east of the site. Most references are in agreement that the boundary of landslide deposits, which forms an obvious geomorphic slope break, is on the order of 200+/- feet east of the eastern site boundary.

2.2 Faulting and Seismicity

The subject site is not located within an Earthquake Fault Zone associated with the Hayward fault zone (Southeastern Extension, as defined by the California Geological Survey, [CGS], 2021) in accordance with the Alquist-Priolo (A-P) Earthquake Fault Zone Act of 1972. The nearest Earthquake Fault Zone study boundary (CGS, 2021) is located approximately 1,500 feet east of the site and the nearest mapped fault trace is located approximately 2,200 feet east of the site. This trace represents the Crosley fault, which is part of the Hayward Fault Zone-Southeastern Extension. Although described as being within an active right-lateral offset fault system, the

Crosley fault has been characterized as a low angle (20 degree dip downward towards the east) thrust fault originating at depth from the near-vertical Hayward fault trace, located farther to the east (approximately 1.4 miles from the site.). The CGS considers the Crosley fault to be active and the U.S. Geological Survey characterizes past movement on the Crosley to be Latest Quaternary (< 15,000 years) with a slip rate of 1-5 millimeters per year.

The City of San Jose (CSJ,1983) has its own Fault Hazard Maps with associated Special Studies Zones, where fault investigations are required for Geologic Clearance for proposed development. Hence, this is one of the main reasons for performing this current study to assess if there is a potential active fault trace on this site. This is the only reference reviewed to date that indicates the possibility of a fault to be on, or in the immediate vicinity of the site. The eastern portion of the site ("study area"; see Figures 2 and 5) is located within the northwest corner of one of these CSJ Special Studies Zones (1983). This particular Special Studies Zone is localized and presumably was established around a short fault splay, geomorphic feature or aerial photo lineament; possibly associated with the Crosley fault farther upslope. It is unclear who delineated this zone for inclusion on the CSJ maps and/or what evidence was utilized to establish this zone. We are unaware of any studies that have previously exposed a fault trace within this City Special Studies Zone. Figure 2, Fault Zones, shows the locations of the CGS and CSJ Special Studies Zones, relative to the site location. A more detailed depiction of the CSJ Special Studies Zone (1983), in reference to the proposed site development and our field exploration locations, is shown on Figure 5. The study area, which includes that portion of the site within the CSJ Special Studies Zone (1983), is shown as the gray cross-hatched area on Figure 5.

In general, the site is located within the seismically active San Francisco Bay Area. Some of the major faults, capable of producing large seismic ground shaking events during the lifetime of the proposed project include the following, as well as their distance and direction from the site:

- Crosley fault (Hayward fault southeastern extension) 0.4 miles east
- Hayward fault 1.4 miles east
- Calaveras fault 3.4 miles east
- San Andreas fault 16 miles southwest

A number of large earthquakes have occurred within the Bay Area region in the historic past. Some of the significant nearby events include the Hayward 1868 (M6.8+), the 1906 San Francisco earthquake (M7.9+), the 1984 Morgan Hill earthquake (M6.2), the 1989 Loma Prieta earthquake (M6.9) and the 2014 South Napa earthquake (M6.0). Future seismic events can be expected to produce strong seismic ground shaking, throughout the Bay Area, as well as this site. The intensity of future shaking will depend on the distance from the site to the earthquake focus, magnitude of the earthquake, and the response of the underlying soil and bedrock. The USGS and the Working Group on California Earthquake Probabilities (2015) indicates that there is a 72% chance of a M6.7 or greater earthquake to occur in this region within the next 30 years.

2.3 Static and Seismic Landslides

Large active to dormant landslide complexes are known to exist in the hills in the site vicinity; however, none of the references reviewed indicates the presence of landslide deposits on the site proper. The CGS (2021) shows the hillside area approximately 225 feet and beyond (east of the site) to be within a Landslide Zone. That is the same area that is shown as a Known Landslide Mass within a CSJ Seismic Hazard Zone. The site is not located within a CSJ Seismic Hazard Zone (current City GIS website). The site, and areas up to 350 feet west of the site (approximately along Piedmont Road), are located within a CSJ Geologic Hazard Zone, which requires some study to determine if there is a potential for ground deformation related to seismic shaking and seismically-induced movement at the toe of known landslide deposits (east of the site). Assessment of this potential effect on site was another factor to be determined for CSJ Geologic Clearance. Figure 3 , City of San Jose Hazard Zones, shows the locations of CSJ Hazard Zones (current City GIS website), as it relates to the location of the overall site. Figure 4, Landslide Maps, shows two references that show the mapped locations of landslide deposits in the site vicinity.

2.4 Other Geologic/Seismic Hazards

The site is not located within any other CGS or CSJ hazards zone including zone of potential liquefaction (hence no potential for lateral spreading) or beyond minimal flooding. According to the CSJ maps, the site is located within a zone characterized by seismic coefficients of S_s: 1.6573, S_1: 0.6524 and PGA: 0.6408.

3 SITE INVESTIGATION

3.1 Field Preparation and Reconnaissance

This initial part of the study consisted of site reconnaissance and field preparation. Field preparation consisted of reviewing existing geologic hazard maps and developing a site exploration plan. Our plan was shared with the City Geologist to gain concurrence on adequacy to properly assess the potential various geologic hazards at the site and to aid in the application for an environmental clearance for the geologic testing prior to commencing the field exploration work. The reconnaissance consisted of laying out and marking the subsurface exploration trenches for Underground Services Alert (USA marking) for public utilities and to assist our private utility locator subconsultant NorCal Underground for onsite utility locating.

In addition, our Certified Engineering Geologist and Staff Geologist performed a geologic reconnaissance to observe and locate surface distress features in the immediate site vicinity that could be indicative of slope movement. Such distress features included crack concrete curbs and sidewalks, warped sidewalks, compressed/buckled storm drain inlet, offset sidewalks slabs and tilted masonry fence columns. The locations of the features are show on Figure 6, Observed Off-Site Surface Distress Features. Photographs of these feature are present on Figure 7, Photos of Off-Site Distress Features. These features appear to be related to slow, continual downhill shallow soil movement either caused by soil creep or possibly minor seasonal movement of landslide deposits. Some of these features, especially within the flatter portion of the site vicinity (i.e. Locations 1, 2, 3, 4, and 5) may also be caused by seasonal shrink/swell movement of the underlying plastic clay soils. No distress features were observed on-site. The closest feature identified during the reconnaissance was located across the street, approximately 40 feet away from the southeast corner of the site, but not directly in-line with the site (i.e. not directly upslope of the site). The closest feature that is most likely due to slope movement and in-line (i.e. directly upslope of the site, Location 7) was located approximately 80 feet east of the eastern site boundary.

3.2 Subsurface Exploration

The subsurface exploration program was conducted to explore geologic earth conditions in the eastern portion of the project site (study area) in order to properly assess the potential existence of faults and/or past ground deformation due to seismically-induced landsliding.

Subsurface exploration consisted of the excavation, logging, and backfill of two exploratory trenches. The trenches were strategically placed to cover the width of the CSJ (1983) Fault Hazard Special Studies Zone (see Figure 5) in the eastern portion of the site (study area). The trenches, one 216 feet in length and another 75 feet in length, were logged by GLA professional geologists. The total length of the exploratory trenches was 291 lineal feet. Trenches were excavated by Pearson Exploration (Sebastopol) to depths of up to 8 feet below existing ground surface with a track excavator equipped with a 30-inch-wide bucket. The trenches were shored for safety with aluminum hydraulic speed shores. The walls of the trenches were cleaned of smeared materials by continuous hand picking by GLA geologists. Once cleaned, the walls of the trenches were logged on a 1" = 5' scale. Logs of the exploratory trenches are presented as Figures 8 (A, B, C) and 9. Trenching operations were performed under the direct supervision of Mr. William McCormick, CEG. The trenches were secured on a daily basis with locking cyclone fencing and covered with steel plates to prevent accidental entry by wildlife or the public.

Upon completion of the logging, the trenches were backfilled with the excavated soil in lifts and each lift was compacted with a compaction wheel on the excavator. Compaction of the lower portion of the trenches was by observation only for safety reason and field density tests were performed on the upper portion of the backfill. Our testing indicates the backfill was compacted to at least 90 percent relative compaction (ASTM D-1557). The results of our field density tests are presented in Appendix A.

To aid in assessing relative recency of faulting or lack of faulting, one sample (1 organic sediment sample) was collected and sent to Beta Analytic Testing Laboratory in Miami, Florida for radiocarbon age dating. The sample were taken from Trench 1 at a depth of 8 feet. Radiocarbon dating was conducted using Accelerator Mass Spectrometry (AMS) techniques* on the bulk organic soils sample. The radiocarbon age-dating result is presented on the log and in Appendix B. The subsurface conditions and age-dating results are further explained in Sections 3.3 and 3.4 below.

3.3 Subsurface Geologic Conditions

As stated above, the purpose of this study was to assess the possible existence of active faults or past ground deformations from landslide movement on this site. We found multiple layers of continuous, near horizontally bedded alluvial soil deposits throughout both trenches. Alluvial soils identified and described on the trench logs generally consisted of gravelly to sandy clays (CL), silty clay, and clay with sand (Cl-CH) with local silty gravelly sands (SM/SP) and gravelly to sandy channel deposits (see Figure 8). A more complete description and graphic depiction of the soils encountered in the trenches are presented on Figures 8 and 9. Gravelly and sandy fill material was encountered locally where the trenches crossed driveways/access roads on site.

We found no evidence indicative of the presence of active faulting or past ground deformation due to landsliding in the subsurface layers on this site.

3.4 Radiocarbon Age Date

Trenches on this site were excavated down to what was considered a practical and safe depth (about 8 feet below ground surface) for personnel entry, logging and soil characterization. During our exploration planning, we anticipated that if an active fault existed on site, we would see indications of fault features or deformation within the upper 8 feet. Since we did not see any evidence for faulting in our trenches, we decided to collect a bulk sediment sample for radiocarbon dating for the lowest exposed layer to determine relative age of the lowest, undisturbed soil layer (Unit 6 in Trench 1). The sample was collected from Station 0+10 in

Trench 1 (see Figure 8). Results of the radiocarbon age-dating yielded an age of 5,670 ± 30 years before present (B.P). Age-dating results are presented in more detail in Appendix B.

4 CONCLUSIONS

4.1 Fault Rupture Hazards

Besides the CSJ (1983) Fault Hazards Maps, no other publications reviewed indicate the potential existence of faulting at this site. As previously stated, it is not known who originally delineated this Special Studies Zone and what evidence was used to identify a potential fault hazard. We are not aware of any previous fault investigations within this short, isolated Special Studies Zone. We suspect that this zone may have been delineated by some limited geomorphic feature or photo lineament, that could have also represented a feature associated with the toe of the large landslide complex in this area. Even if there is a fault associated with this study zone, it is very limited in extent, with a very low probability of generating large enough magnitude earthquakes to result in surface rupture (generally consider to be a minimum M6 magnitude). Study zones are typically delineated as a "box", equidistant around the mapped feature. This site is located in the far northwest corner of the Special Study Zone and would generally not be expected to encounter this short, linear, potential fault feature even if it existed.

Our subsurface exploration did not encounter any fault-related features on this site. Undisturbed alluvial soil layers indicate no faulting within approximately the last 6,000 years.

Taking into account all of the collected data analyzed for this study, it is our professional opinion that it is unlikely that an active or inactive fault exists at depth below this site and that there is a very low potential for ground rupture to adversely affect this site.

4.2 Landslide Hazards

Our study did not identify any features on, under**,** or immediately adjacent to the site that would indicate that there are landslide deposits on the site or that there is any potential for future distress or damage to future development on this due to static and/or seismically-induced landsliding. While none of the literature references or our subsurface exploration indicate**s** the presence of landslide deposits on site, it is well documented that active and dormant landslide deposits exist within a couple hundred feet of the eastern site boundary. Also, there are surficial distress feature associate with slope movement (either soil creep or landslide movement/creep) within at least 80 feet east of the site boundary. As previously stated, the site is located within a CSJ Geologic Hazard Zone, which requires relative assessment of the potential for future site distortion by seismically-induced movement of known landslide masses a couple hundred feet upslope (east) of the site.

In order to provide an evaluation of this hazard potential, we have relied on two sources of data:

1. Subsurface geologic exposures within our trenches.

2. Seismically-induced landslide displacement analysis by others for the nearby Penitencia Creek Landslide Complex (PCLC) for the adjacent Penitencia Water Treatment Plant (PWTP).

Our subsurface explorations show no evidence for distorted or disrupted ground, soil layers or the existence of landslide deposits on the site. The soils exposed in our trenches to a depth of 8 feet are on the order of at least 6,000 years old. This is clearly much older than the generally accepted 200-year recurrence interval for large earthquakes (which would be required to induce landsliding) on the two major fault systems in the Bay Area; the San Andreas and Hayward fault zones. Therefore, by inference, we can conclude that seismically induced landslide movement has not affected the site within the past 6,000 years (at least) and would not likely occur in the future, especially during the expected lifetime of the proposed development on this site.

In addition to the geologic data we have exposed on site, we can draw conclusions from a nearby study at the PWTP performed by Salah-Mars, et al. (Woodward-Clyde, 1995). In general, that study analyzed the potential for earthquake-induced landslide movement at the plant, which is underlain by a more active portion of the massive landslide complex; as compared to the portion of the complex that is directly above and east of this site. They modeled the geometry, deformation from past earthquakes and displacement analysis/estimation for a magnitude M7 event on the Hayward fault. That study concluded that future large earthquakes (M7) on the nearby Hayward fault could result in slope displacements on the order of 1 to 6 feet laterally. A more recent study at the same location by Baune (2016) and Givler, et al. (2019) indicate that large earthquake-induced movement of the PCLC could be on the order of 7.7 feet.

Even if we take the high-end estimate of 7.7 feet of lateral movement, it is reasonable to conclude that no seismically-induced landslide movement would adversely affect development at this site since the toe of the existing landslide complex is located at least 200 feet east of the eastern site boundary.

4.3 Geohazard Implications for Proposed Development

Based on our study, it is our opinion that there are no fault ground rupture hazard or seismically-induced landslide effects that would adversely affect this site, nor preclude development as planned. Results of this study do not change the geotechnical conclusions and geotechnical design recommendations for development (other than a revised development layout) presented in our previous geotechnical report. Therefore, we still consider our previous Geotechnical Report (dated March 28, 2023) to be valid for development and design at this site. For convenience, our 2023 geotechnical report is included as Appendix C.

5 REFERENCES

Baune, Darren, 2016, A Unique Solution to a Unique Problem: Large-Diameter Pipeline Seismic Retrofit Mitigates Landslide Hazards; *delivered at Pipelines 2016, a conference sponsored by ASCE and its Utility Engineering & Surveying Institute.*

California Geological Survey (CGS, 2021) EQ Zapp webpage, Special Studies Zone, Calaveras Quadrangle.

City of San Jose, 1983, Fault Hazard Maps

City of San Jose, 1995, Phase 1A Regional Geologic Study; Plate C, Landslide Map

City of San Jose, (Current), GIS Webpage, Geologic and Seismic Hazard Zones

Dibblee, Thomas.W., 2005, Geologic Map of the Calaveras Quadrangle, Dibblee Geology Center Map #DF-154.

Givler, R., J.N, Baldwin, A. Seifried, W. Godwin, M. Meyers, P. Gregory, 2019, Penitencia Creek Landslide Evaluation and Seismic Retrofit of Large Diameter Water Conveyance Pipelines in San Jose, CA, Conference Proceedings, Association of Environmental and Engineering Geologists (AEG) 2019 Annual Meeting at San Francisco, California.

Salah-Mars, S., R.K. Green, H. Kanakari, L.H. Mejia and K.D. Weaver,1995, Evaluation of Earthquake Induced Slope Displacements, International; Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics 7.

Working Group on California Earthquake Probabilities, 2015, Published as: Field, E.H., Biasi,G.P., Bird, P., Dawson, T.E., Felzer, K.R. Jackson, D.D., Johnson, K.M., Jordan, T.H.,Madden, C. Michael, A.J., Milner, K.R., Page, M.T., Parsons, T., Powers, P.M., Shaw,B.E., Thatcher, W.R., Weldon, R.J. II, and Zeng, Y. (2015), Long-term, time-dependent probabilities for the third uniform California earthquake rupture forecast (UCERF3),Bulletin of the Seismological Society of America.

6 LIMITATIONS

In preparing the findings and professional opinions presented in this report, Geo-Logic Associates (GLA) has endeavored to follow generally accepted principles and practices of the engineering geologic and geotechnical engineering professions in the area and at the time our services were performed. No warranty, express or implied, is provided.

The conclusions and recommendations contained in this report are based, in part, on information that has been provided to us. In the event that the general development concept (i.e. standard residential and minor grading) or general location and type of structures are modified significantly, our conclusions and recommendations shall not be considered valid unless we are retained to review such changes and to make any necessary additions or changes to our recommendations.

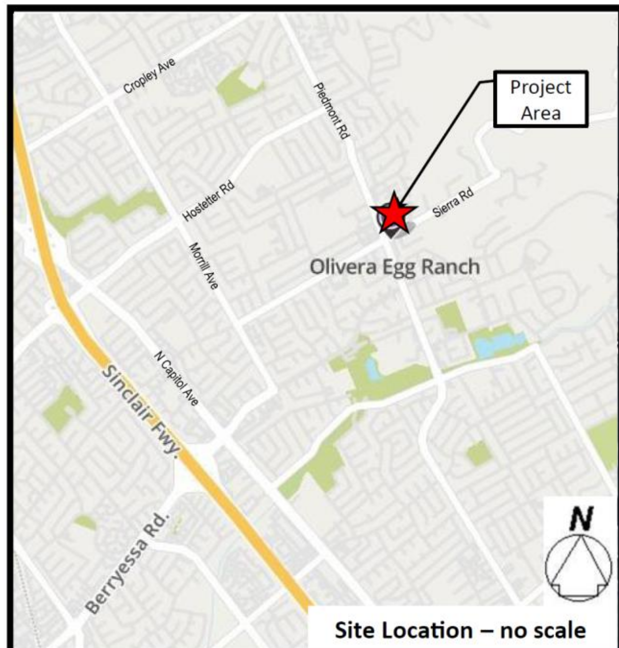
Subsurface exploration is necessarily confined to selected locations and conditions may, and often do, vary between these locations. Should conditions different from those described in this report be encountered during project development, GLA should be consulted to review the conditions and determine whether our recommendations are still valid. Additional exploration, testing, and analysis may be required for such evaluation.

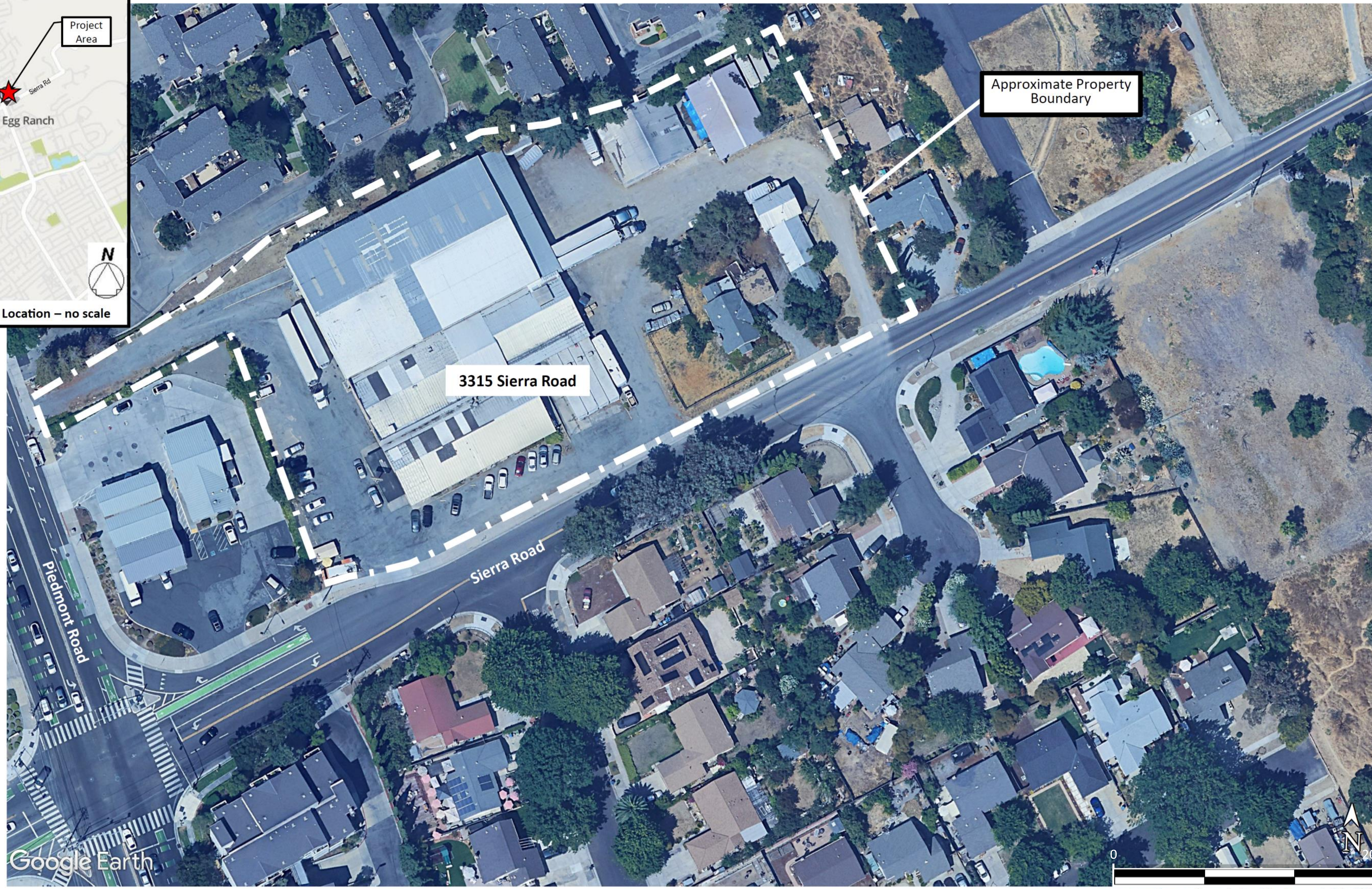
The findings, conclusions, and recommendations in this report are applicable only to the specific project development on this specific site. These data should not be used for other projects, sites, or purposes unless they are reviewed by GLA or a qualified geotechnical professional.

Report prepared by,

Geo-Logic Associates

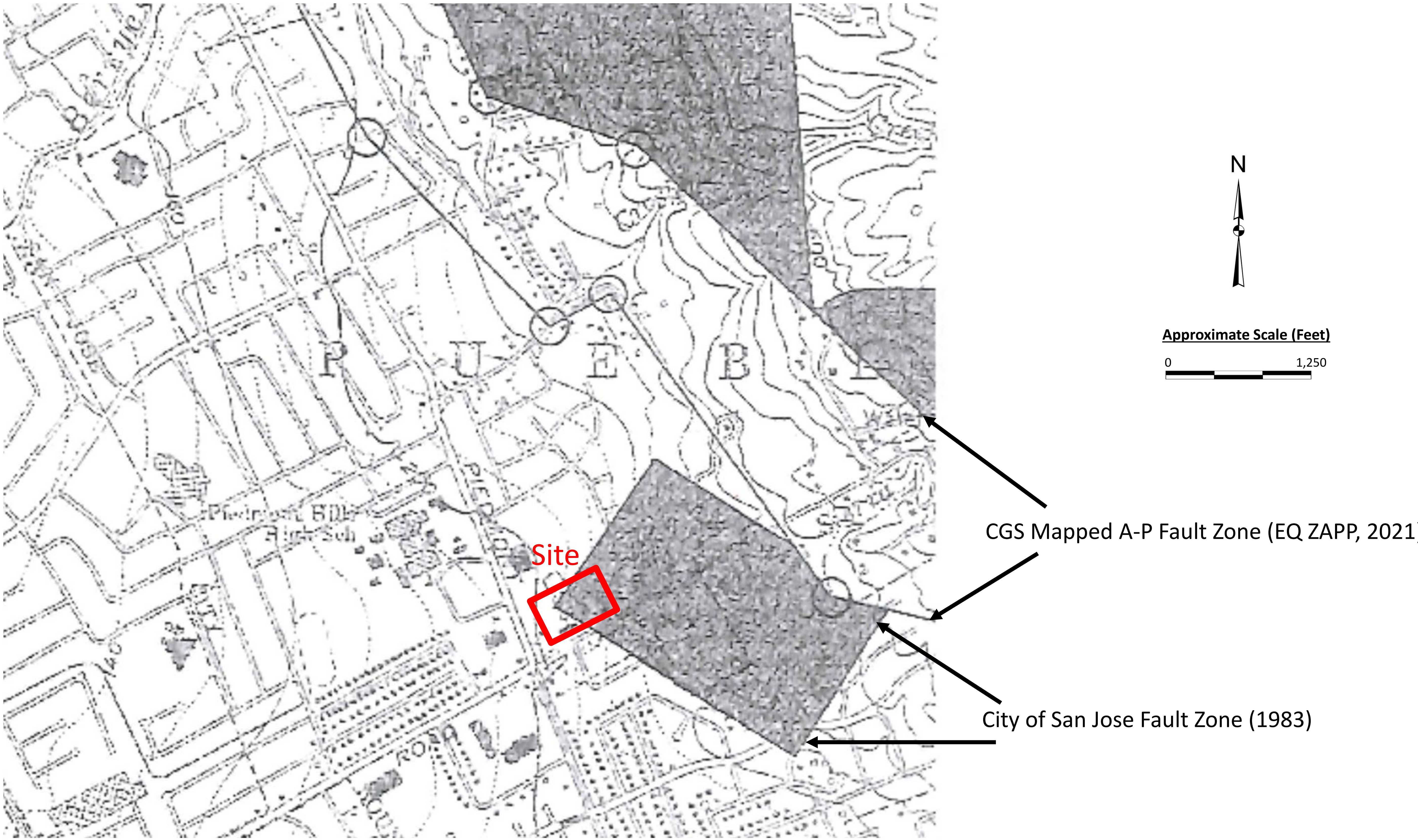
William V. McCormick, CEG 1673	Chalerm (Beeson) Liang, GE 2031
Principal Engineering Geologist	Principal Geotechnical Engineer



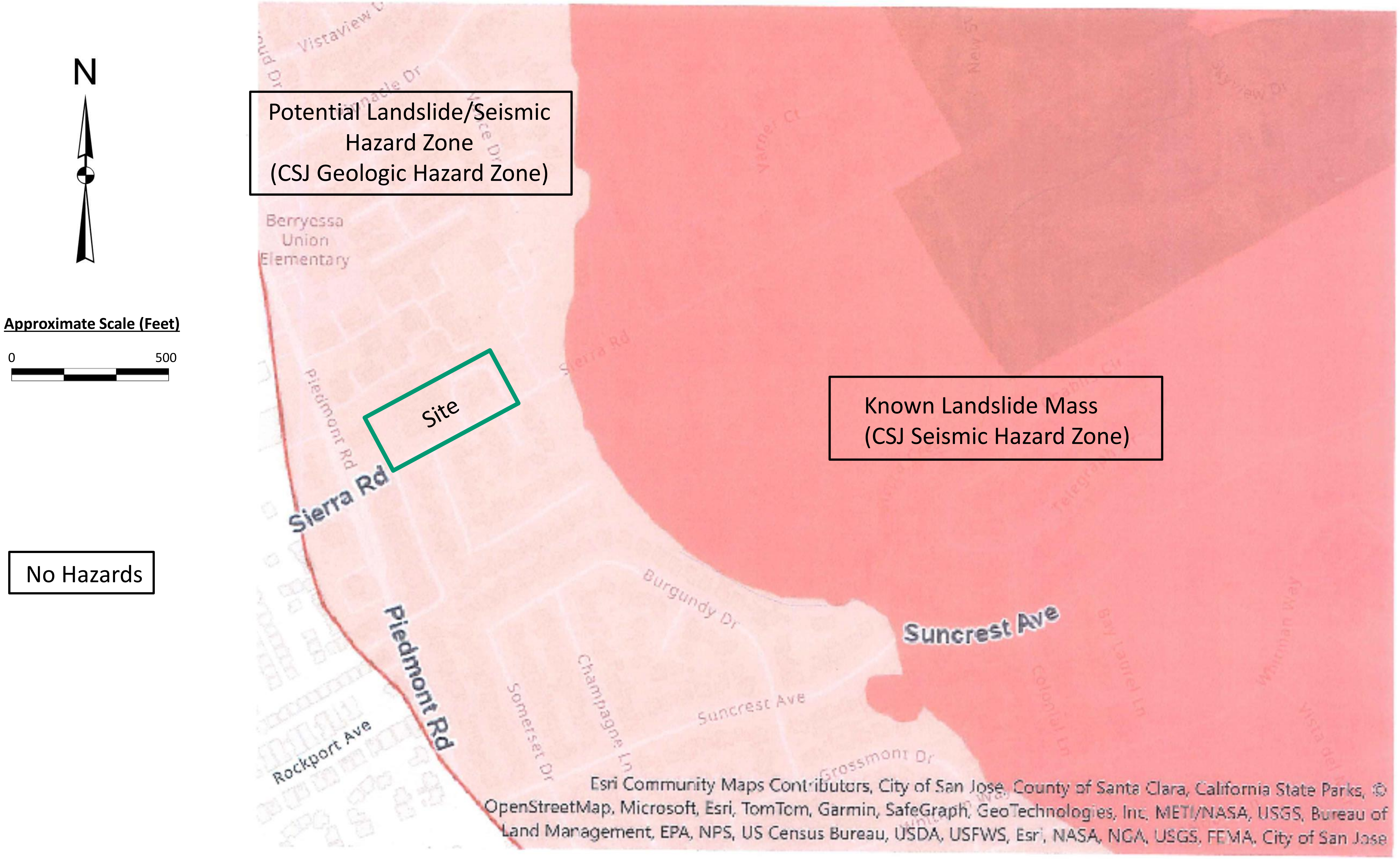


Base: Google Earth, August 30, 2023.

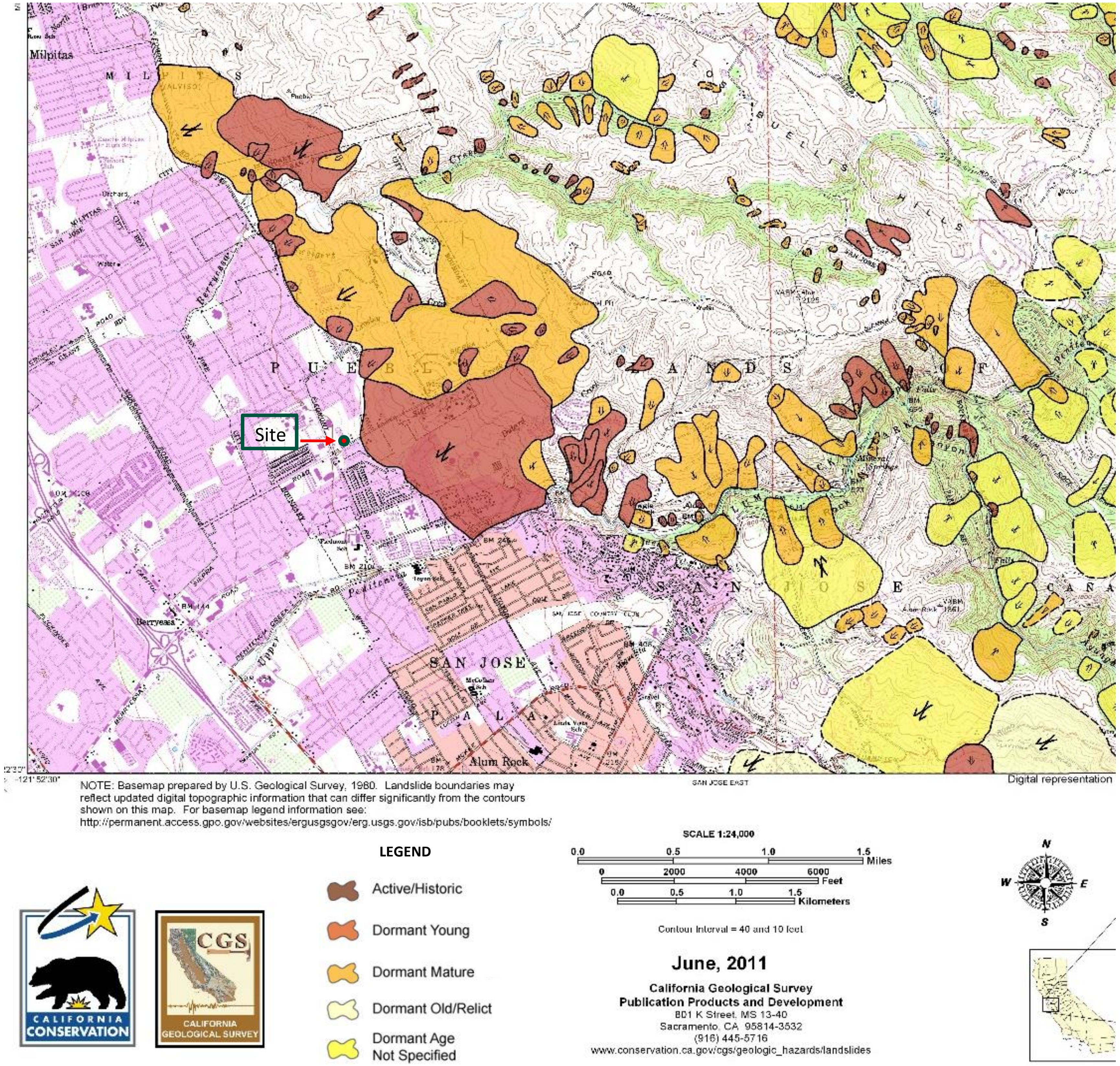
Geo-Logic ASSOCIATES	6300 San Ignacio Avenue, Suite A San Jose, California 95119 Phone (408) 778-2818	Drafted By: FS Date: June 2024 Checked By: BM Revision: June 2024	**SITE PLAN (EXISTING CONDITIONS)** Proposed Residential Development 3315 Sierra Road San Jose, California	**FIGURE** 1 **PROJECT** PA22.1048



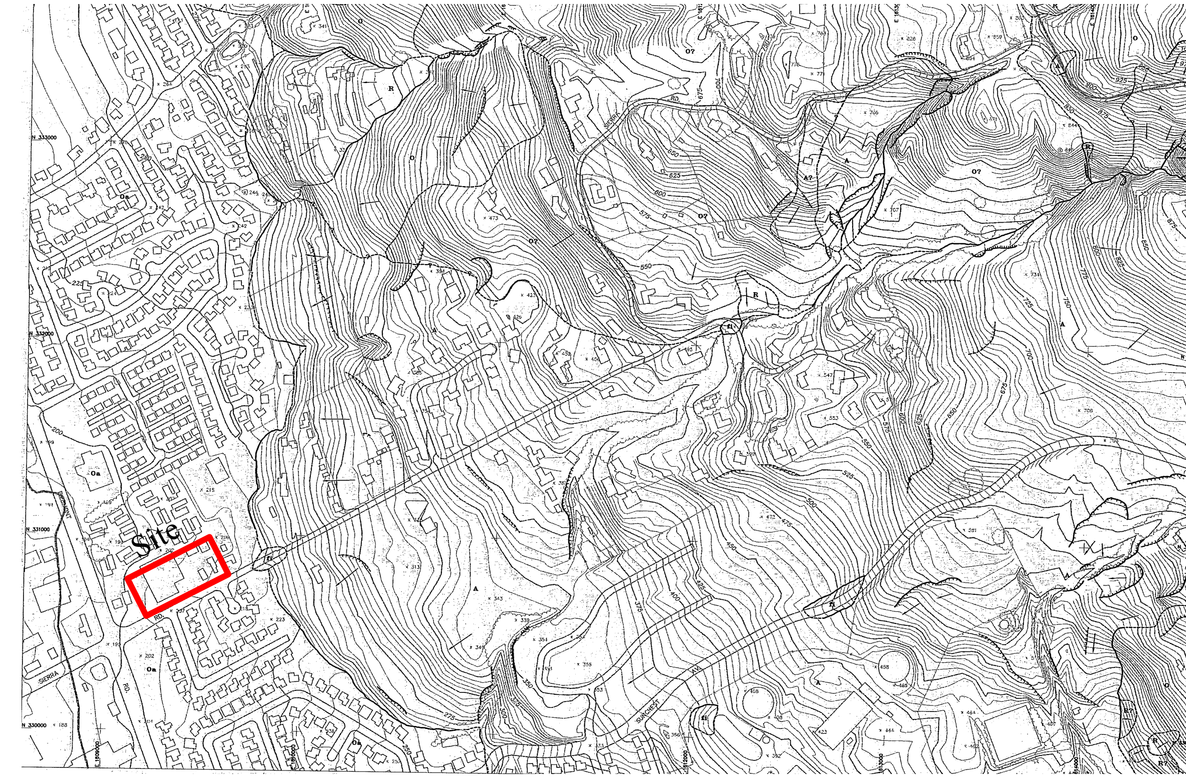
Geo-Logic ASSOCIATES	6300 San Ignacio Avenue, Suite A San Jose, California 95119 Phone (408) 778-2818	Drafted By: FS Date: June 2024 Checked By: BM Revision: June 2024	**FAULT ZONES** 3315 Sierra Road, San Jose, California	**FIGURE** 2 **PROJECT** PA22.1048



Geo-Logic ASSOCIATES	6300 San Ignacio Avenue, Suite A San Jose, California 95119 Phone (408) 778-2818	Drafted By: FS Date: June 2024 Checked By: BM Revision: June 2024	**CITY OF SAN JOSE HAZARD ZONES** 3315 Sierra Road, San Jose, California	**FIGURE** 3 **PROJECT** PA22.1048



Source: Landslide Inventory Map of the Calaveras Reservoir Quadrangle Alameda and Santa Clara Counties, California (CGS 2011)



Source: City of San Jose Phase 1A Regional Geologic Study: Plate C, Landslide Map (7/13/1995)

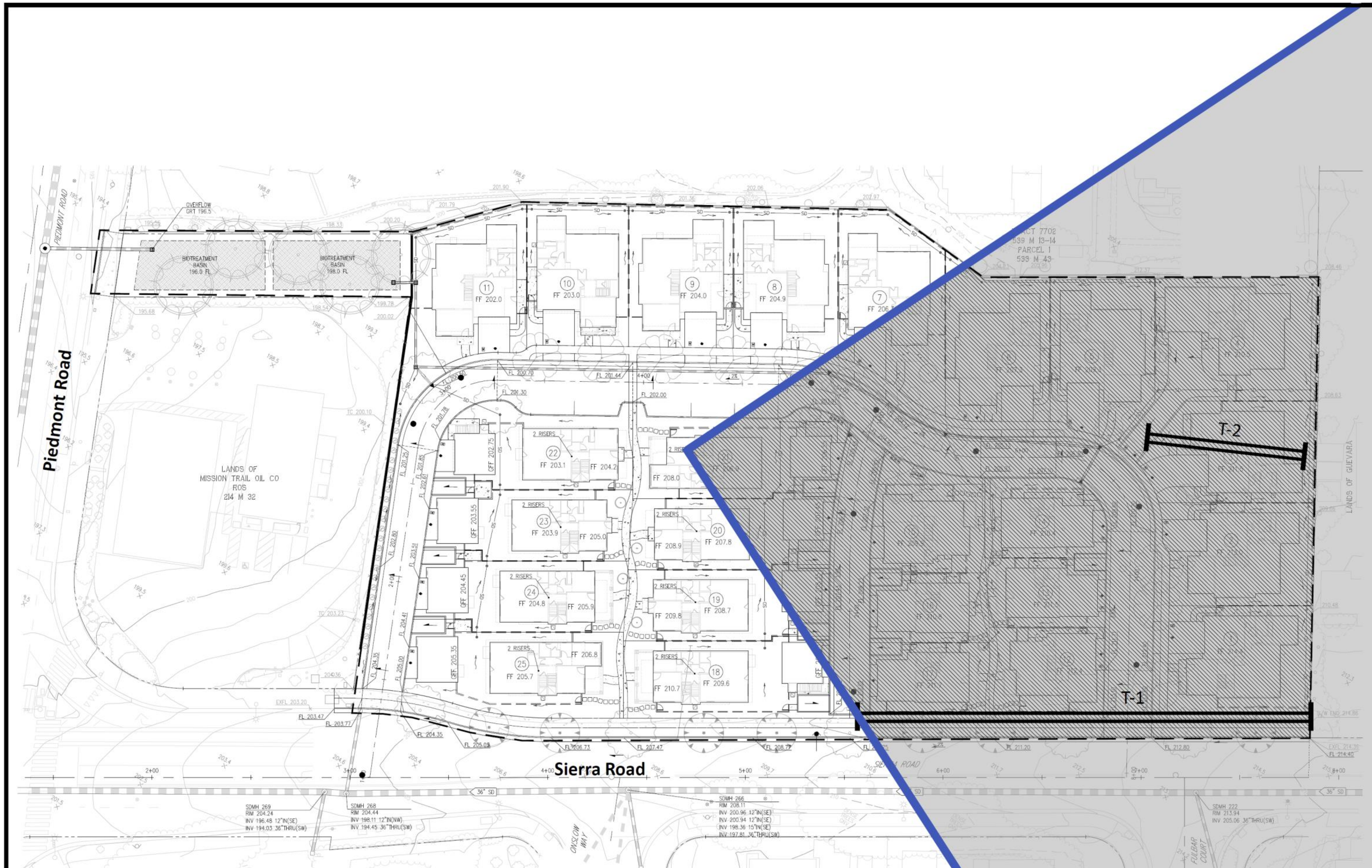
N

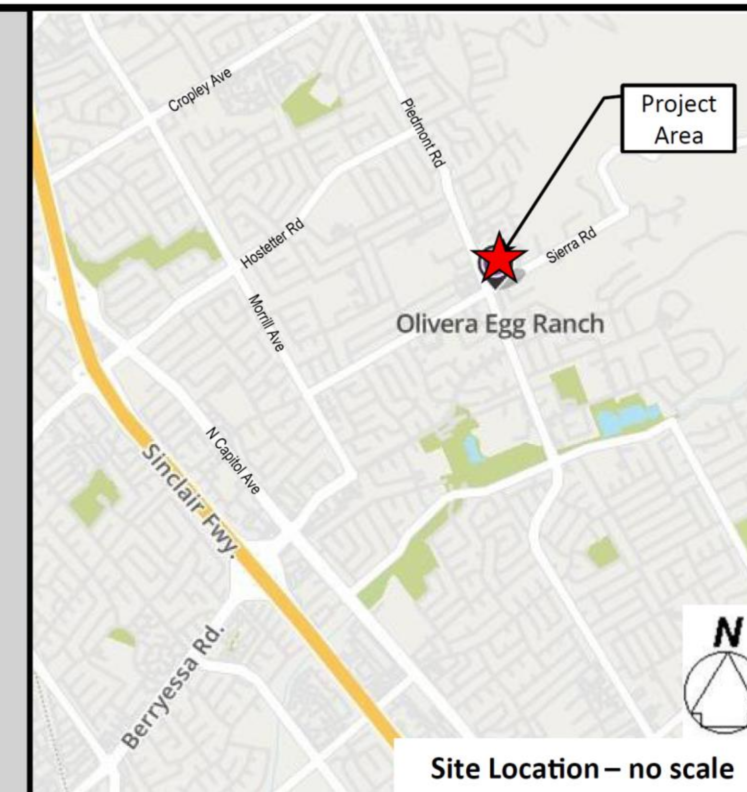
Approximate Scale (Feet)

0 1000

Geo-Logic ASSOCIATES	6300 San Ignacio Avenue, Suite A San Jose, California 95119 Phone (408) 778-2818	Drafted By: FS Date: June 2024 Checked By: BM Revision: June 2024	**REGIONAL LANDSLIDE MAPS** 3315 Sierra Road San Jose, California

FIGURE
4
PROJECT
PA22.1048





Legend

T-2 — Exploration Trench (this study)

City of San Jose Special Studies Zone Boundary (CSJ, 1983)

Portion of site within City of San Jose Special Studies Zone (CSJ, 1983)

City of San Jose Special Studies Zone (CSJ, 1983)

Approximate Scale (Feet)

0 30 60

Base: 3315 Sierra Road Site Plan, prepared by Civil Engineering Associates, dated 3/7/2024.

Geo-Logic ASSOCIATES	6300 San Ignacio Avenue, Suite A San Jose, California 95119 Phone (408) 778-2818	Drafted By: FS Date: June 2024 Checked By: BM Revision: June 2024	**SPECIAL STUDIES ZONE** 3315 Sierra Road San Jose, California	**FIGURE** 5 **PROJECT** PA22.1048

1. Cracked curb
2. Cracked and warped sidewalk
3. Tilted brick column
4. Multiple sidewalk cracks
5. Tilted brick column
6. Down-warped sidewalk
7. Buckled storm drain inlet
8. Distorted sidewalk slabs
9. Warped, depressed sidewalk

See Figure 7 for vicinity photos

Base: Google Earth, August 30, 2023

Approximate Scale (Feet)

0 30 60

N

Geo-Logic ASSOCIATES	6300 San Ignacio Avenue, Suite A San Jose, California 95119 Phone (408) 778-2818	Drafted By: FS Date: June 2024 Checked By: BM Revision: June 2024	OBSERVED OFF-SITE SURFACE DISTRESS FEATURES 3315 Sierra Road, San Jose, California	**FIGURE** 6 **PROJECT** PA22.1048

1

2

3

4

5

6

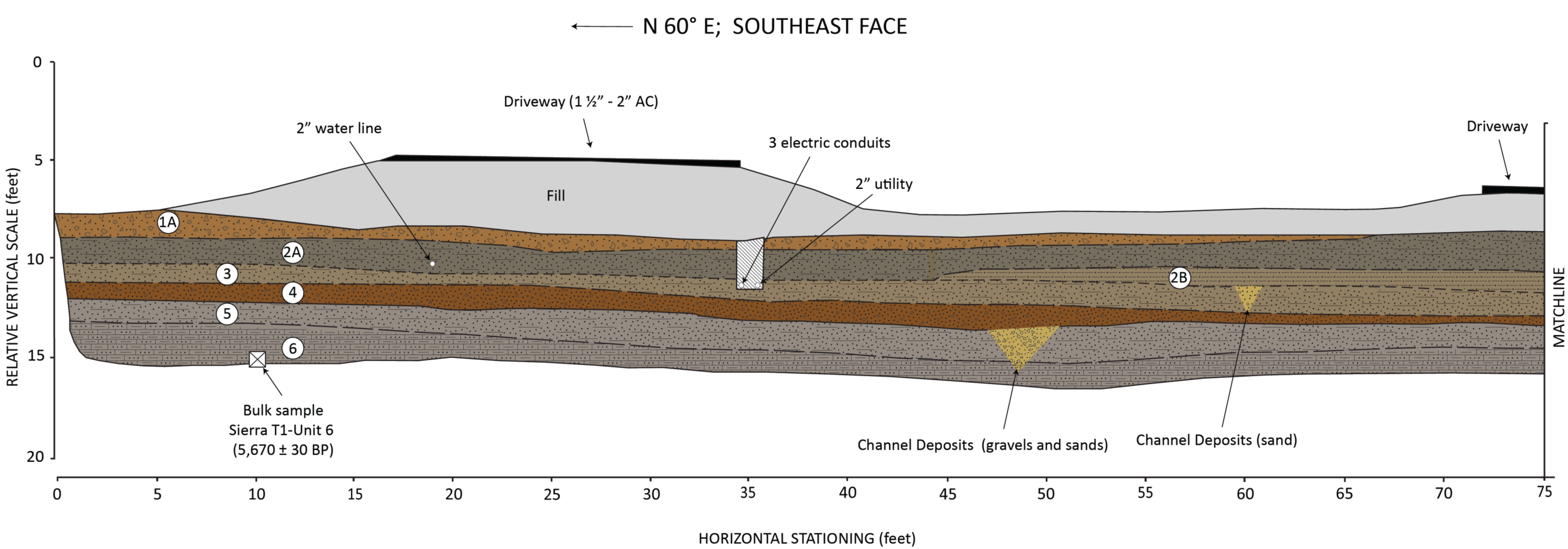
7

8

9

Numbers refer to feature locations shown on Figure 6

Geo-Logic ASSOCIATES	6300 San Ignacio Avenue, Suite A San Jose, California 95119 Phone (408) 778-2818	Drafted By: FS Date: June 2024 Checked By: BM Revision: June 2024	PHOTOS OF OFF-SITE DISTRESS FEATURES 3315 Sierra Road, San Jose, California	FIGURE 7 PROJECT PA22.1048



UNIT DESCRIPTIONS

- **Fill** — Multiple layers of sand and gravel, dry, dense (fill)
- **1A** — GRAVELLY SANDY CLAY (CL) - very dark brown (10YR 2/2), damp, very stiff, porous, gravels are sub-rounded to rounded up to 1/2" - 3"
- **2A** — SILTY CLAY W/ SAND and occasional gravels (CL) - very dark grayish brown (10YR 4/2), damp, hard, slightly porous, gravels are sub-angular to sub-rounded up to 1/4" - 1"
- **2B** — CLAY W/ SAND (CL) - very dark grayish brown (10YR 3/2), damp, stiff, porous, occasional pebbles and gravels, sub-angular to sub-rounded
- **3** — SILTY CLAY W/ SAND and occasional gravels (CL) - very dark grayish brown (10YR 3/2), damp, hard, very slightly porous, scattered, sub-angular gravels up to 1"
- **4** — SILTY CLAY W/ SAND and occasional gravels (CL) - very dark grayish brown (10YR 4/2), damp, hard, slightly porous, gravels are sub-angular to sub-rounded up to 1/4" - 1"
- **5** — CLAY W/ SAND (CL-CH) - very dark gray (5Y 3/1), fine-grained sand, damp, hard, non porous, abundant multi-directional polished surfaces
- **6** — CLAY W/ SAND (CL-CH) - very dark gray (5Y 3/1), fine-grained sand, damp, very hard, non porous, abundant carbonate blebs, occasional angular gravels, chert up to 1/2" - 2", occasional multi-directional polished surfaces

5 2.5 0 5

APPROXIMATE HORIZONTAL AND VERTICAL SCALE: 1"=5'

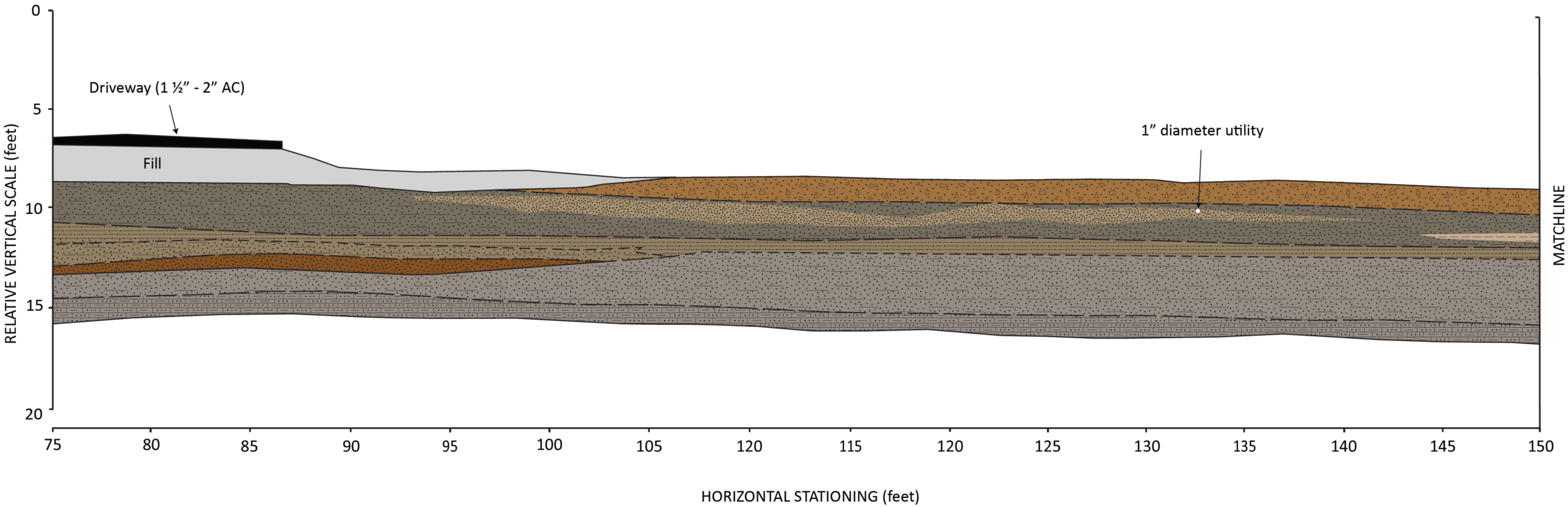
EXPLANATION

———	Geologic Contact (<1" wide)
— — —	Gradational Geologic Contact (1" to 3" wide)
- - - -	Gradational Geologic Contact (>3" wide)
⊠	Bulk Soil Radiocarbon Dating Sample Location
(5,670 ± 30 BP)	Radiocarbon Date of Tested Sample (years before present)

Geo-Logic ASSOCIATES

DATE	LOG OF TRENCH 1	FIGURE
JUNE 2024	3315 SIERRA ROAD, SAN JOSE, CALIFORNIA	8A
Drafted by: FS		PROJECT
Checked by: BM		PA22.1048

N 60° E; SOUTHEAST FACE



UNIT DESCRIPTIONS

Fill	Multiple layers of sand and gravel, dry, dense (fill)
1A	GRAVELLY SANDY CLAY (CL) - very dark brown (10YR 2/2), damp, very stiff, porous, gravels are sub-rounded to rounded up to 1/2" - 3"
2A	SILTY CLAY W/ SAND and occasional gravels (CL) - very dark grayish brown (10YR 4/2), damp, hard, slightly porous, gravels are sub-angular to sub-rounded up to 1/4" - 1"
2B	CLAY W/ SAND (CL) - very dark grayish brown (10YR 3/2), damp, stiff, porous, occasional pebbles and gravels, sub-angular to sub-rounded
3	SILTY CLAY W/ SAND and occasional gravels (CL) - very dark grayish brown (10YR 3/2), damp, hard, very slightly porous, scattered, sub-angular gravels up to 1"
4	SILTY CLAY W/ SAND and occasional gravels (CL) - very dark grayish brown (10YR 4/2), damp, hard, slightly porous, gravels are sub-angular to sub-rounded up to 1/4" - 1"
5	CLAY W/ SAND (CL-CH) - very dark gray (5Y 3/1), fine-grained sand, damp, hard, non porous, abundant multi-directional polished surfaces
6	CLAY W/ SAND (CL-CH) - very dark gray (5Y 3/1), fine-grained sand, damp, very hard, non porous, abundant carbonate blebs, occasional angular gravels, chert up to 1/2" - 2", occasional multi-directional polished surfaces

EXPLANATION

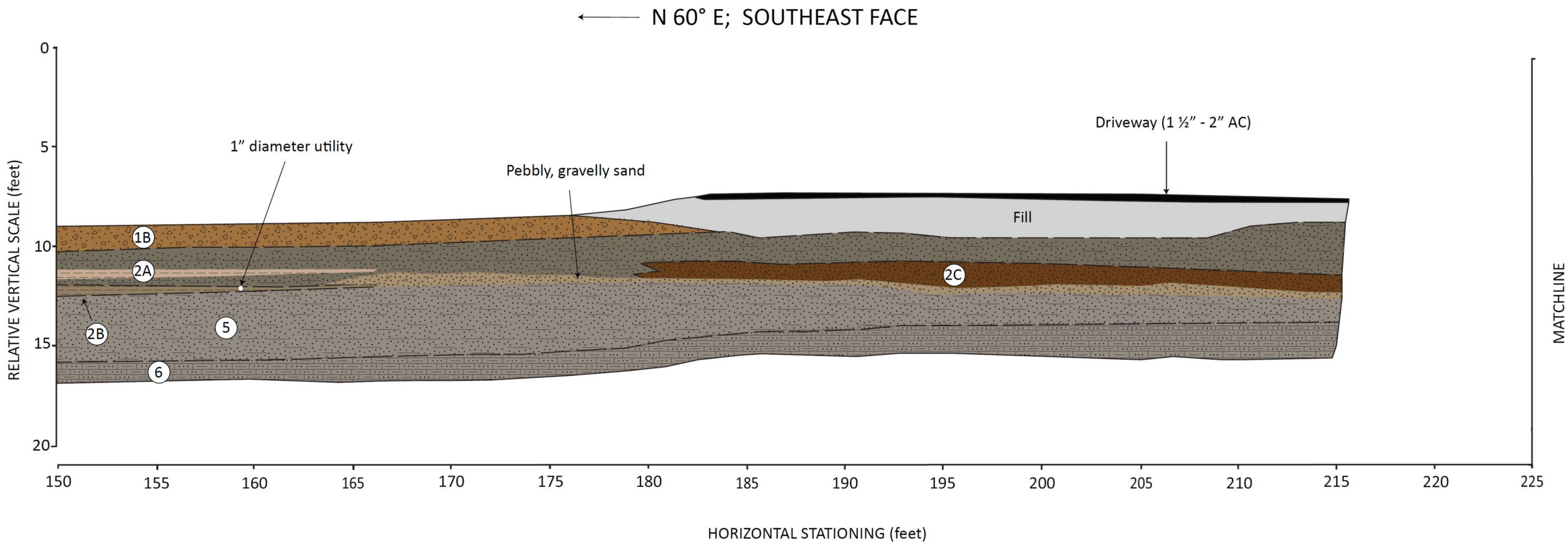
- Geologic Contact (<1" wide)
- Gradational Geologic Contact (1" to 3" wide)
- Gradational Geologic Contact (>3" wide)

5 2.5 0 5

APPROXIMATE HORIZONTAL AND VERTICAL SCALE:
1"=5'

Geo-Logic ASSOCIATES

DATE	LOG OF TRENCH 1	FIGURE
JUNE 2024	3315 SIERRA ROAD, SAN JOSE, CALIFORNIA	8B
Drafted by: FS		PROJECT
Checked by: BM		PA22.1048



UNIT DESCRIPTIONS

Fill – Multiple layers of sand and gravel, dry, dense (fill)

1B – GRAVELLY SANDY CLAY (CL) - very dark brown (10YR 2/2), damp, very stiff, porous, gravels are sub-rounded to rounded up to 1/2" - 3"

2A – SILTY CLAY W/ SAND and occasional gravels (CL) - very dark grayish brown (10YR 4/2), damp, hard, slightly porous, gravels are sub-angular to sub-rounded up to 1/4" - 1"

2B – CLAY W/ SAND (CL) - very dark grayish brown (10YR 3/2), damp, stiff, porous, occasional pebbles and gravels, sub-angular to sub-rounded

2C – SANDY CLAY (CL) - dark brown (10YR 3/3), damp, stiff, slightly porous, pebbles and gravels at base, sub-angular to sub-rounded

5 – CLAY W/ SAND (CL-CH) - very dark gray (5Y 3/1), fine-grained sand, damp, hard, non porous, abundant muli-directional polished surfaces

6 – CLAY W/ SAND (CL-CH) - very dark gray (5Y 3/1), fine-grained sand, damp, very hard, non porous, abundant carbonate blebs, occasional angular gravels, chert up to 1/2" - 2", occasional multi-directional polished surfaces

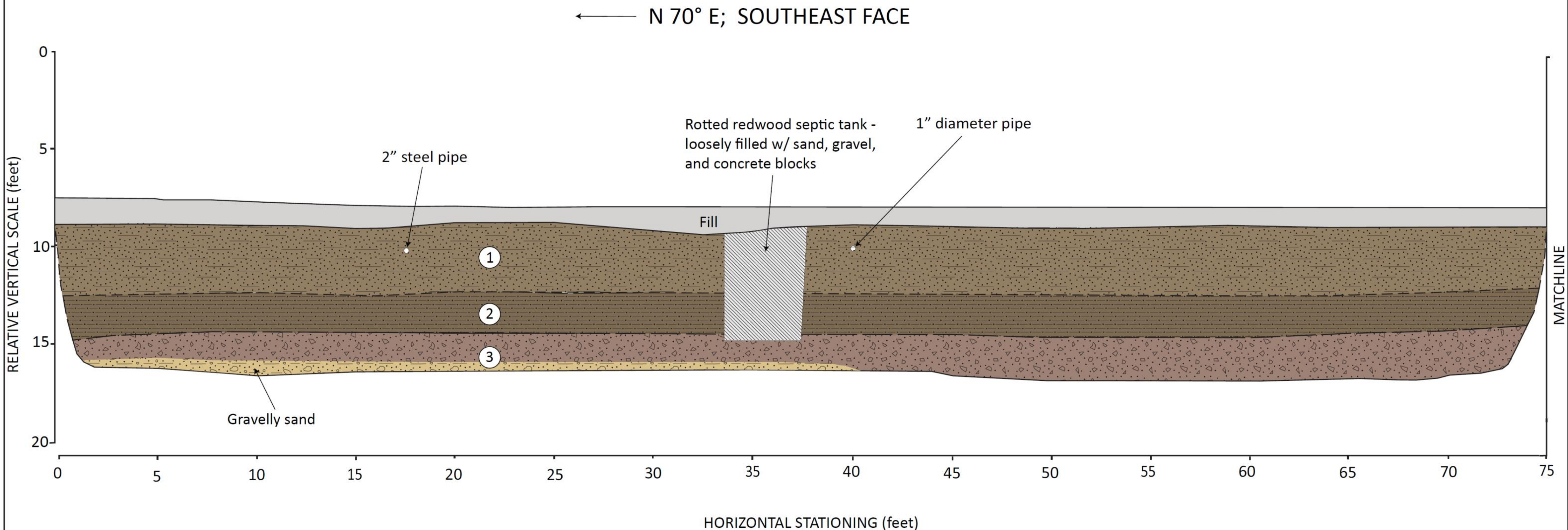
5 2.5 0 5

APPROXIMATE HORIZONTAL AND VERTICAL SCALE:
1"=5'

EXPLANATION

- Geologic Contact (<1" wide)
- Gradational Geologic Contact (1" to 3" wide)
- Gradational Geologic Contact (>3" wide)

Geo-Logic ASSOCIATES	DATE: JUNE 2024	LOG OF TRENCH 1	FIGURE: 8C
	Drafted by: FS	3315 SIERRA ROAD, SAN JOSE, CALIFORNIA	PROJECT: PA22.1048
	Checked by: BM		



UNIT DESCRIPTIONS

Fill — Multiple layers of sand and gravel, dry, dense (fill)

1 — SILTY CLAY W/ SAND (CL) - mottled very dark grayish brown (10YR 2/2), dark grayish brown (10YR 4/2), damp, hard, slightly porous, gravels are sub-angular to angular up to <1"

2 — CLAY W/ SAND (CH) - mottled near base, very dark grayish brown (10YR 3/2), dark grayish brown (10YR 4/2), highly gradational unit, damp, hard, non porous, abundant calcareous blebs, occasional small polished surfaces in various directions, gravels are scatteted, sub-angular to sub-sub-rounded u to <1"

3 — SILTY GRAVELLY SAND (SM/SP) TO SANDY CLAY (CL) - dark brown (10YR 3/3), damp, medium dense to hard, slightly porous, fine to coarse grained-sand, calcareous blebs in upper 6-8", gravels are sub-angular to sub-rounded up to <3" of sandstone and chert

EXPLANATION

- Geologic Contact (<1" wide)
- Gradational Geologic Contact (1" to 3" wide)
- Gradational Geologic Contact (>3" wide)

5 2.5 0 5

APPROXIMATE HORIZONTAL AND VERTICAL SCALE: 1"=5'

Geo-Logic ASSOCIATES	DATE JUNE 2024	LOG OF TRENCH 2	FIGURE 9
	Drafted by: FS	3315 SIERRA ROAD, SAN JOSE, CALIFORNIA	PROJECT PA22.1048
	Checked by: BM		

APPENDIX A

FIELD DENSITY TEST RESULTS OF TRENCH BACKFILL

6300 San Ignacio Avenue, Suite A
San Jose, CA 95119
408-778-2818 T

TABLE A. FIELD DENSITY TEST RESULTS
3315 SIERRA ROAD, SAN JOSE, CALIFORNIA
PA22.1048.00
Date of Work: 5/23/2024 through 5/30/2024

Test #	Date	Approximate Location	Approx. Depth of Fill (ft.)	Approx. Depth Below Finish Grade (ft.)	Lab Max Dry Density (pcf)	Field Moisture (%)	Field Dry Density (pcf)	Relative Compaction (%)	Min. Compaction (%)	Pass / Fail	Retest # / Comment
		Trench 1									
1	5/23/2024	East of easternmost driveway	8	RSG	114.7	18	103	90	90	PASS	
2	5/23/2024	East driveway	8	RSG	114.7	14	110	96	90	PASS	
3	5/23/2024	In front of barn	7	1	114.7	20	108	94	90	PASS	
4	5/29/2024	Between barn driveway and home driveway	5	2	114.7	20	103	90	90	PASS	
		Trench 2									
5	5/29/2024	In front of barn - west	7	1	114.7	14	105	91	90	PASS	
6	5/29/2024	In front of barn - middle	8	RSG	114.7	18	105	91	90	PASS	
7	5/29/2024	In front of barn - east	5	3	114.7	13	105	91	90	PASS	
8	5/29/2024	East	8	RSG	114.7	24	105	91	90	PASS	
9	5/29/2024	East	8	RSG	114.7	19	106	92	90	PASS	
		Trench 1									
10	5/30/2024	Between barn driveway and home driveway	2	RSG	114.7	18	105	91	90	PASS	
11	5/30/2024	East of home driveway	2	RSG	114.7	13	105	92	90	PASS	

RSG = Rough subgrade

APPENDIX B

RESULTS OF BETACAL 5.0 TEST FROM BETA ANALYTIC RADIOCARBON DATING LABORATORY

BetaCal 5.0

Calibration of Radiocarbon Age to Calendar Years

(High Probability Density Range Method (HPD): INTCAL20)

(Variables: d13C = -25.1 o/oo)

Laboratory number **Beta-698933**

Conventional radiocarbon age **5670 ± 30 BP**

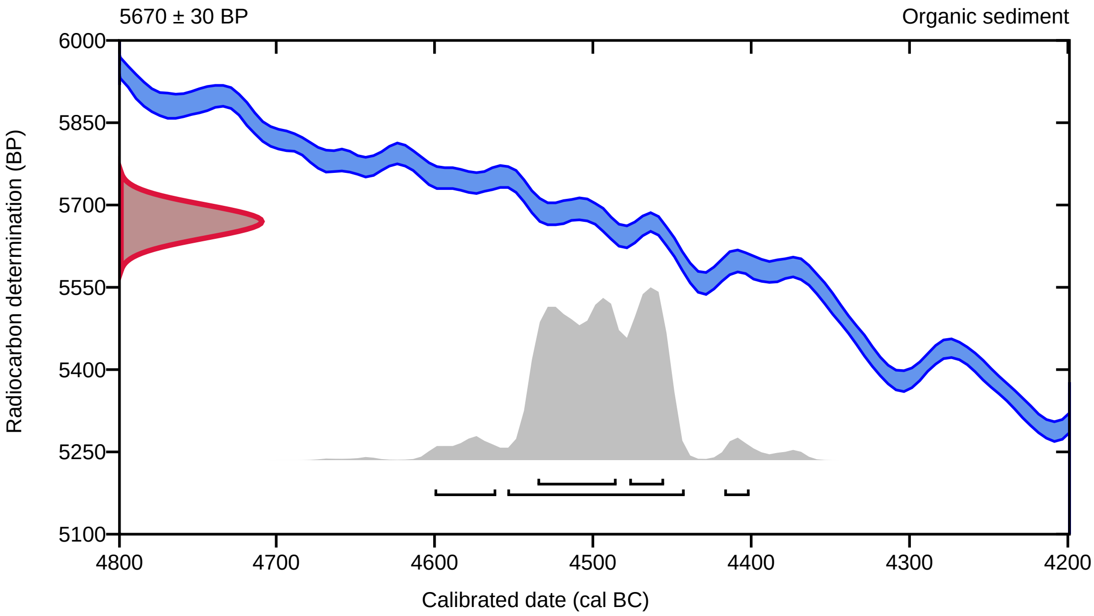
95.4% probability

(89.1%)	4556 - 4444 cal BC	(6505 - 6393 cal BP)
(4.4%)	4602 - 4563 cal BC	(6551 - 6512 cal BP)
(1.9%)	4419 - 4403 cal BC	(6368 - 6352 cal BP)

68.2% probability

(46.4%)	4537 - 4487 cal BC	(6486 - 6436 cal BP)
(21.8%)	4479 - 4457 cal BC	(6428 - 6406 cal BP)

Sierra T1-Unit 6



Database used

INTCAL20

References

References to Probability Method

Bronk Ramsey, C. (2009). Bayesian analysis of radiocarbon dates. Radiocarbon, 51(1), 337-360.

References to Database INTCAL20

Reimer, et al., 2020, Radiocarbon 62(4):725-757.

Beta Analytic Radiocarbon Dating Laboratory

4985 S.W. 74th Court, Miami, Florida 33155 • Tel: (305)667-5167 • Fax: (305)663-0964 • Email: info@betalabservices.com

APPENDIX C

GLA GEOTECHNICAL STUDY REPORT FOR 3315 SIERRA ROAD, SAN JOSE, CA

GEOTECHNICAL STUDY
RESIDENTIAL DEVELOPMENT

3315 SIERRA ROAD
SAN JOSE, CALIFORNIA

MARCH 28, 2023
PROJECT PA22.1048.00

SUBMITTED TO:

Robson Homes
2185 The Alameda, Suite 150
San Jose, CA 95126

PREPARED BY:

Geo-Logic Associates

Geo-Logic Associates
6300 San Ignacio Avenue, Suite A
San Jose, California 95119
(408) 778-2818

GEOTECHNICAL STUDY
PROPOSED RESIDENTIAL DEVELOPMENT
3315 SIERRA ROAD
SAN JOSE, CALIFORNIA

Table of Contents

1 INTRODUCTION

This report presents the results of our geotechnical study for a proposed residential development at 3315 Sierra Road, San Jose, California. The property, with an area of about 2.71 acres, is referred as the “property,” “site,” or “project site” in this report. The Assessor Parcel Number (APN) of the property is 565-10-067. The approximate location of the project site is shown on the Vicinity Map included with Figures 1 and 2 of this report. Figure 1 shows a layout of the proposed development and Figure 2 shows the existing site conditions.

This report presents our findings, conclusions, and geotechnical recommendations for design and construction of the project. These findings, conclusions, and recommendations are based on information collected during this study. The conclusions and recommendations in this report should not be extrapolated to other areas or used for other projects without our review.

1.1 Project Description

The project site is currently occupied by a business with several buildings, a loading dock, a paved parking lot, and several residences. The proposed residential development will include single-family units. Associated improvements will include underground utilities, landscaping, exterior flatwork, driveways, and an on-site street. Site grading will be limited to cuts and fills of generally 1 to 3 feet deep because of the flat-lying topography across the site. The proposed residential buildings will be three-story, wood-framed structures. No basements are planned for the residential units. Retaining walls, if needed, will be free-standing landscaping walls generally a few feet high and not a part of the proposed buildings.

The above project descriptions are based on information provided to us. If the actual project differs from those described above, Geo-Logic Associates (GLA) should be contacted to review our findings, conclusions, and recommendations, and to present any necessary modifications to address the different project development schemes.

1.2 Information Provided

For this study, our client provided us with the following information.

- A drawing titled “3315 Sierra Road Site Plan,” prepared by Civil Engineering Associates, dated February 14, 2023.

1.3 Purpose and Scope of Services

The purpose of this geotechnical study was to explore subsurface conditions at the project site and to provide geotechnical recommendations for design and construction of the proposed improvements. The following work was performed.

1. Performed a site reconnaissance to observe site surface conditions and to mark the locations of our subsurface exploration.

2. Reviewed available geologic and geotechnical information pertinent to the site.

3. Notified Underground Service Alert (USA) for underground utility clearance.

4. Coordinated our field exploration with our client.

5. Subcontracted with a private underground services locator to check the proposed exploration locations for presence of underground utilities.

6. Explored subsurface conditions by means of six exploratory drill holes.

7. Collected a bulk sample of the near-surface soil from the site.

7. Performed laboratory tests on selected soil samples from the drill holes and on the bulk sample to measure pertinent engineering properties of the samples.

8. Performed engineering analysis on the field and laboratory data.

9. Prepared this geotechnical study report.

2 SITE INVESTIGATION

This study consists of a site reconnaissance and a subsurface exploration program. The site reconnaissance was to observe existing site surface conditions. The subsurface exploration program was to explore subsurface earth conditions at the project site. The observed surface and subsurface site conditions are discussed in Section 3 of this report.

2.1 Subsurface Exploration

Our subsurface exploration program consisted of six exploratory drill holes (DH-1 through DH-6) advanced on November 11, 2022. The drill holes were located in the field by referencing to existing site features and pacing; therefore, their locations are approximate. The approximate locations of the drill holes are shown in Figures 1 and 2 of this report.

The drill holes were advanced using a truck-mounted Mobile B-61 drilling rig equipped with 8-inch diameter hollow stem augers to depths of approximately 20 and 45 feet below ground surface (bgs). Soil samples were obtained using a 3-inch O.D. (2½-inch I.D.) split-barrel sampler. Soil samples were obtained by driving the sampler up to 18 inches into the earth material using a 140-pound automatic-trip hammer falling 30 inches. The number of blows required to drive the sampler was recorded for each 6-inch penetration interval. The number of blows required to drive the sampler the last 12 inches, or the penetration interval indicated on the log when harder material was encountered, is shown as blows per foot (blow count) on the drill hole logs.

In the field, our personnel visually classified the materials encountered and maintained a log of each drill hole. Visual classification of soils encountered in our drill holes was made in general accordance with the Unified Soil Classification System (ASTM D-2487 and D-2488). The results of our laboratory tests were used to refine our field classifications. Two Keys to Soil Classification, one for fine grained soils and one for coarse grained soils, are included in Appendix A, together with the logs of the drill holes.

2.2 Laboratory Testing

Geotechnical laboratory testing was conducted on selected soil samples collected from our drill holes. These tests included moisture content, dry density, Atterberg limits, sieve analysis, and hydrometer. An R-value test was performed on the bulk sample collected from the site. The laboratory test results are presented on the drill hole logs at the corresponding sample depths. Graphic presentations of the results of the Atterberg limits, sieve analysis, and R-value tests are presented on separate sheets in Appendix B.

In addition to geotechnical testing, two selected soil samples were sent to CERCO Analytical for corrosivity analysis. A brief report from CERCO Analytical with the corrosivity test results is included in Appendix B.

3 FINDINGS

3.1 Surface Conditions

The site is currently occupied by the Olivera Egg Ranch. There are several buildings in the central portion of the site, with a loading dock in the northeastern portion of the buildings and a paved parking lot adjacent to the buildings. The eastern portion of the site is occupied by several residential structures, sheds, a paved driveway, grassy areas, and trees. Ground surface across the site is flat-lying, with a gentle down slope from the southeast to the north and west. There is an utility pole in the southeastern portion of the site.

3.2 Subsurface Conditions

In DH-1, a pavement section consisting of roughly ¼ inch of asphalt concrete over roughly 3 inches of base rock was encountered at the ground surface. Below the pavement section, a layer of very stiff to hard sandy clay of intermediate plasticity was encountered to a depth of about 12 feet bgs. This clay is underlain by very stiff to hard clay to a depth of about 27 feet bgs, dense clayey sand with gravel to a depth of about 32 feet bgs, hard clay to a depth of about 43 feet bgs, and dense clayey sand to the maximum explored depth of about 45 feet bgs.

In DH-2, a pavement section consisting of roughly 1 inch of asphalt concrete over roughly 8 inches of base rock was encountered at the ground surface. Below the pavement section, a layer of fill consisting of medium dense clayey sand with gravel was encountered to a depth of about 2 feet bgs. The fill is underlain by stiff to very stiff sandy clay to medium dense clayey sand to a depth of about 4 feet bgs, very stiff sandy clay to a depth of about 8 feet bgs, stiff to very stiff clay to a depth of about 19.5 feet bgs, and medium dense clayey sand with gravel to the maximum explored depth of about 20 feet bgs.

In DH-3, a pavement section consisting of roughly 2 inches of asphalt concrete over roughly 6 inches of base rock was encountered at the ground surface. Below the pavement section, a layer of very stiff to hard sandy clay was encountered to a depth of about 12 feet bgs, underlain by a layer of hard clay to the maximum explored depth of about 20 feet bgs.

In DH-4, a pavement section consisting of roughly 2 inches of asphalt concrete over roughly 11 inches of base rock was encountered at the ground surface. Below the pavement section, a layer of very stiff to hard sandy clay was encountered to a depth of about 8 feet bgs. This clay is underlain by dense clayey sand with gravel to a depth of about 12 feet bgs, very stiff to hard clay to a depth of about 19.5 feet bgs, and medium dense clayey sand with gravel to the maximum explored depth of about 20 feet bgs.

In DH-5, a layer of stiff to hard sandy clay was encountered to a depth of about 19.5 feet bgs, underlain by a layer of medium dense clayey sand with gravel to the maximum explored depth of about 20 feet bgs.

In DH-6, a layer of hard sandy clay was encountered to a depth of about 14.5 feet bgs, underlain by a layer of medium dense to dense clayey sand with gravel to the maximum explored depth of about 20 feet bgs.

3.3 Groundwater

Groundwater was not encountered in our six drill holes for this study, the deepest of which extended to about 45 feet bgs. Our review of Plate 1.2, "Depth to historically high ground water and locations of boreholes used in this study, Calaveras Reservoir 7.5-minute Quadrangle, California," Seismic Hazard Zone Report 048, prepared by California Geological Survey, Department of Conservation, 2001, indicates that historically high groundwater level at the site was greater than 50 feet.

It should be noted that fluctuations in the groundwater level may occur due to seasonal variations in rainfall and temperature, water level in nearby creeks, pumping from wells, regional groundwater recharge program, irrigation, or other factors that were not evident at the time of our study.

3.4 Variations in Subsurface Conditions

Our interpretations of soil and groundwater conditions, as described in this report, are based on information obtained from subsurface exploration and laboratory testing for this study. Our conclusions and recommendations are based on these interpretations. Please realize the site has undergone different phases of development and grading. Therefore, it is likely that undisclosed variations in subsurface conditions exist at the site, particularly old foundations, abandoned utilities, and localized areas of deep and loose fill.

Careful observations should be made during construction to verify our interpretations. Should variations from our interpretations be found, we should be notified to evaluate whether any revisions should be made to our recommendations.

4 SEISMIC CONSIDERATIONS

4.1 Earthquake Faulting

The San Francisco Bay area is seismically dominated by the active San Andreas Fault system, the tectonic boundary between the northward moving Pacific Plate (west of the fault) and the North American Plate (east of the fault). This movement is distributed across a complex system of generally strike-slip, right-lateral, and subparallel faults.

Potential sources of significant earthquake ground shaking at the site include several active and potentially active faults in the Greater San Francisco Bay area, as well as faults farther afield. The faults were first compiled on the State's Fault Activity Map (Jennings, 1974; Jennings and Bryant, 2010). This map has now been integrated into the US Geological Survey's Quaternary Fault and Fold Database and made available as a .kmz "drape" over Google Earth terrain files.

The distance to a seismic source (fault) is defined by the Next Generation Attenuation (NGA) relationships as the closest distance to the seismogenic zone, be it in the subsurface or at the surface; distances may therefore differ from distances measured on the ground surface. The distances shown on the table below are for reference only, as they are horizontal distances from the site to the surface trace of the seismic source, and not necessarily the closest distance to a (dipping) seismogenic zone. These distances were measured using the US Geological Survey's Quaternary Fault and Fold Database, with major faults listed in approximate order of distance from the site; not all sources are listed in the summary table below.

Fault Name	Approximate Distance	Orientation from Site
Hayward (southeast extension)	¾ km	Northeast
Calaveras (central segment)	5½ km	Northeast
Monte Vista	21 km	Southwest
San Andreas	27 km	Southwest
Sargent	30½ km	Southwest
San Gregorio	49½ km	Southwest

4.2 Site Class for Seismic Design

To evaluate the site class for seismic design for this project site, we reviewed published shear wave velocity in the upper 30 meters (V_{S30}) information from the U.S. Geologic Survey A Compilation of V_{S30} Values in the United States website and evaluated the drill hole information for this study. The published V_{S30} value for three sites near the subject project site ranges between 234 and 345 meters per second, generally corresponds to a site class D. Our evaluation of the drill hole information, following the procedures outlined in Section 20.4 of ASCE 7-16, also suggests a site class D. Therefore, a site class D is considered appropriate for the project site.

4.3 Ground Accelerations

According to the 2022 California Building Code (CBC) and American Society of Civil Engineers (ASCE) Standard 7-16, the spectral response acceleration at any period can be taken as the lesser of the spectral response accelerations from the probabilistic and deterministic ground motion approaches. The U.S. Seismic Design Maps tool available at the Structural Engineers Association of California (SEAOC) website was used for this purpose to retrieve seismic design parameter values for design of buildings at the subject site. Two levels of ground motions are considered in the Application: Risk-targeted Maximum Considered Earthquake (MCE_R) and Design Earthquake (DE), with both probabilistic and deterministic values defined in terms of maximum-direction rather than geometric-mean, horizontal spectral acceleration (S_a). The probabilistic MCE_R spectral response accelerations are represented by a 5 percent damped acceleration response spectrum having a 1 percent probability of collapse within a 50-year period and in the direction of the maximum horizontal response. The probabilistic Design Earthquake (DE) S_a value at any period can be taken as two-thirds of the MCE_R S_a value at the same period.

Using the Seismic Design Maps application at the SEAOC website, a site Class D, and the latitude and longitude of the site (latitude 37.40064º N, longitude -121.84628º W), the calculated geometric mean peak ground acceleration adjusted for site class effects (PGA_M) for the MCE_G (Geometric Mean Maximum Considered Earthquake) is 1.01g.

4.4 Seismicity

The Working Group on California Earthquake Probabilities' (WGCEP) estimates of the probabilities of major earthquakes are now in their sixth iteration, with the greatest changes in approach being the inclusion of multifold rupture scenarios, in the progressive consideration of more potential seismic sources, the possibility of earthquakes on unrecognized faults, and the inclusion of the notion of fault "readiness". Current estimates (WGCEP, 2014) for the San Francisco region indicate a 72% probability of a large (magnitude 6.7 or greater) earthquake in the San Francisco Bay area as a whole over the 30-year period beginning in 2014; this overall probability is greater than the previous (WGCEP, 2007) probability of 63%, due mainly to the inclusion of multi-fault rupture scenarios. The estimate for the Calaveras fault alone is 14.4% (revised up from the 7% presented by WGCEP, 2007); for the (northern) San Andreas fault alone, 27.4% (revised upward from the WGCEP (2007) value of 21%); and for the Hayward fault, 45.3% (revised upward from the WGCEP (2007) value of 31%).

4.5 Liquefaction

Soil liquefaction is a phenomenon in which saturated granular soils, and certain fine-grained soils, lose their strength due to the build-up of excess pore water pressure during cyclic loading, such as that induced by earthquakes. Soils most susceptible to liquefaction are saturated, clean, loose, fine-grained sands and non-plastic silts. Certain gravels, plastic silts, and clays are also susceptible to liquefaction. The primary factors affecting soil liquefaction include: 1) intensity and duration of seismic shaking; 2) soil type; 3) relative density of granular soils; 4) moisture content and plasticity of fine-grained soils; 5) overburden pressure; and 6) depth to groundwater.

The project site is not located in a California Geological Survey (CGS) Earthquake Zones of Required Investigation nor a County of Santa Clara liquefaction hazard zone. In our opinion, the potential for liquefaction is low because of the lack of groundwater in the upper 50 feet at the site and the medium dense to dense relative density of the granular soils.

4.6 Dynamic Compaction of Granular Soils

Dynamic compaction of granular soils is settlement of unsaturated sand and gravel soils above the groundwater table as a result of seismic shaking from an earthquake. Based on our analysis, the estimated total settlement from dynamic compaction is small, less than about ¼ inch. Potential differential settlement is about one-half of the estimated total settlement.

4.7 Seismic Design Parameters

Design of the proposed structures should comply with design for structures located in seismically active areas. Structures should be designed in accordance with the requirements of governing jurisdictions and applicable building codes. GLA evaluated ASCE 7-16 seismic design parameters for the site using the SEAOC U.S. Design Maps application. The table below lists the seismic design parameters for the site. Note that, in accordance with Section 11.4.8 of ASCE 7-16, a ground motion hazard analysis is required because the Mapped Spectral Acceleration at 1.0-second Period (S_1) value for the site is larger than 0.2g, unless the exceptions in Section 11.4.8 are met. This should be verified by the structural engineer.

Seismic Design Parameter	Value[1]
Site Class	D
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.7
Mapped Spectral Acceleration at 0.2-second Period, S_s	2.18g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.843g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	2.18g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	1.433g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.453g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.955g
Long-period Transition Period, T_L	12 sec.
Note: 1. The F_V, S_{M1} and S_{D1} values provided in the table above assume that the exceptions in Section 11.4.8 of ASCE 7-16 are met.	

5 CONCLUSIONS AND DISCUSSION

Based on our geotechnical evaluation, it is our opinion the project site may be developed as discussed in this report, provided our geotechnical recommendations are incorporated in the design and construction of the project. Our opinions, conclusions, and recommendations are based on our understanding of the proposed development, data review, properties of soils encountered in subsurface exploration, laboratory test results, and engineering analyses. Geotechnical considerations for this project are discussed below.

5.1 Ground Rupture

The project site is not located in an Alquist-Priolo Earthquake Fault Zone. Because no active or potentially active faults are known to cross the site, the risk of fault rupture through the project site is low.

5.2 Seismic Shaking

The site is in an area of high seismicity. Based on general knowledge of site seismicity, it should be anticipated that, during the design life of the improvements, the site will be subject to high intensity ground shaking from at least one severe earthquake (magnitude 7 to 8+). It is also anticipated that the site will periodically experience small to moderate magnitude earthquakes. The proposed improvements should be designed accordingly using applicable building codes and experience of the design professionals.

5.3 Liquefaction and Dynamic Compaction

The project site is not located in a California Geological Survey (CGS) Earthquake Zones of Required Investigation liquefaction hazard zone nor a County of Santa Clara liquefaction hazard zone. In our opinion, the potential for liquefaction is low because of the lack of groundwater in the upper 50 feet at the site and the medium dense to dense relative density of the granular soils.

Based on our analysis, the estimated total settlement from dynamic compaction is small, less than about ¼ inch. Potential differential settlement is about one-half of the estimated total settlement.

5.4 Expansion Potential of Surficial Soils

The results of two Atterberg limits tests performed on two near-surface clay samples from our drill holes indicate the clays have intermediate plasticity with plasticity indices of 24 and 29, which generally correspond to medium to high expansion potential.

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 rainfall, landscape irrigation, perched groundwater, drought or other factors. Changes in soil moisture may result in unacceptable settlement or heave of structures, concrete slabs and pavements supported on these materials. Depending on the extent and location below finished subgrade, these soils could have a detrimental effect on the proposed construction.

To reduce its potential impact on the proposed structures, the upper 30 inches of soil below design grade in the proposed building and concrete slab-on-grade areas should be moisture conditioned with controlled compaction per the "Geotechnical Recommendations" section of this report. The post-tensioned slab foundations for the proposed structures should be designed using the recommended parameters in this report to accommodate the potential effect of soil expansion. Exterior concrete slabs should be constructed on a layer of "non-expansive" fill over moisture conditioned subgrade soil.

5.5 Existing Improvements

Existing improvements at the site include miscellaneous structures, underground utilities, fences, pavements, and isolated trees. Prior to construction, the designated existing structures and improvements should be removed and the resulting excavations should be properly backfilled with engineered fill under the observation and testing of the geotechnical engineer.

The properties adjoining the project site have been developed. Design and construction of project improvements should consider the neighboring structures and improvements to avoid undermining or adversely impacting these existing structures and improvements.

6 GEOTECHNICAL RECOMMENDATIONS

6.1 Earthwork

6.1.1 Site Preparation, Clearing and Stripping

Prior to grading, construction areas should be cleared of designated structures and foundations, obstructions, deleterious materials, debris, abandoned or designated utility lines, designated trees, and other below grade obstacles encountered during the site clearing operation. Tree stumps should be grubbed. Roots with diameter of about 1 inch or larger or length of about 3 feet or longer should be removed. Depressions, excavations, and holes that extend below the planned finish grades should be cleaned and backfilled with engineered fill compacted to the requirements given under the section of "Engineered Fill Placement and Compaction."

After clearing, vegetated areas should be stripped to sufficient depth to remove vegetation and organic-laden topsoil. Organic laden soils are defined as soils with more than 3 percent by weight of organic content. Stripped material may be stockpiled for use in landscape areas if approved by the project landscape architect; otherwise, it should be removed from the site. Typical stripping depth would be about 3 to 6 inches in vegetated areas. The actual stripping depth should be determined in the field by the geotechnical engineer at the time of construction.

6.1.2 Excavation, Temporary Construction Slopes, and Shoring

Excavations are expected for demolition, cuts to achieve design grades, trenching to construct new underground improvements, and foundation excavations. Excavation walls in clayey soil and less than 5 feet in height should be able to stand near vertical with minimal bracing, provided proper moisture content in the soil is maintained. Granular (sand and gravel) soils, typically have little or no cohesion, will require more extensive bracing or laying back because they are prone to sudden collapse. Excavations and temporary construction slopes should be constructed in accordance with the current CAL-OSHA safety standards and local jurisdiction. The stability and safety of excavations, braced or unbraced, is the responsibility of the contractor. Care should be exercised when excavating in the proximity of existing structures and improvements.

Contractors are responsible for the design, installation, maintenance, and removal of temporary shoring and bracing systems. The presence of existing improvements must be incorporated in the design of the shoring and bracing systems.

Trench excavations adjacent to existing or proposed foundations should be above an imaginary plane having an inclination of 1½:1 (horizontal to vertical) extending down from the bottom edge of the foundations. If achieving this is not possible, GLA should be contacted to evaluate options to protect the existing improvements.

6.1.3 Subgrade Preparation

After site clearing and stripping, the soil subgrades should be prepared as recommended below.

Building and concrete slab-on-grade areas: Soils in building and concrete slab-on-grade areas should be over-excavated to at least 18 inches below design pad grade, but not less than 12 inches below existing grade. The soil surfaces exposed by over-excavation should be scarified to a depth of 12 inches, moisture-conditioned, and compacted in accordance with the recommendations given in the "Engineered Fill Placement and Compaction" section below. In structure areas to receive concrete slabs-on-grade or foundations, subgrade preparation should extend at least 5 feet horizontally beyond the limits of the proposed structures and any adjoining flatwork, unless it is restricted by existing improvements.

Pavement areas: Soils in pavement areas should be scarified to at least 12 inches below pavement subgrade level, moisture-conditioned, and compacted in accordance with the recommendations given in the "Engineered Fill Placement and Compaction" section below. Subgrade preparation should extend at least 3 feet beyond the back of the curbs or pavements.

Prepared soil subgrades should be non-yielding when proof-rolled by a fully-loaded water truck or similar weight equipment. Moisture conditioning of subgrade soils should consist of adding water if the soils are too dry and allowing the soils to dry if the soils are too wet. After the subgrades are properly prepared, the areas may be raised to design grades by placement of engineered fill.

Wet soils should be anticipated during and after rainy months and in existing building and pavement areas. Where encountered, unstable, wet, or soft soil will require processing before compaction can be achieved. If construction schedule does not allow for air-drying, other means such as lime or cement treatment of the soil or excavation and replacement with suitable material may be considered. Geotextile fabrics may also be used to help stabilize the subgrade. The method to be used should be determined at the time of construction based on the actual site conditions. We recommend obtaining unit prices for subgrade stabilization during the construction bid process.

6.1.4 Exterior Slabs-on-Grade

To reduce the potential effects of soil expansion on exterior slabs-on-grade, the following two options may be considered.

“Non-expansive” Fill: Construct exterior concrete slabs-on-grade on a section of “non-expansive” fill over properly moisture-conditioned and compacted subgrade soil. The “non-expansive” fill layer should be at least 6 inches thick below the bottom of the slabs. “Non-expansive” fill should meet the recommendations in the “Materials for Engineered Fill” section and should be compacted per the "Engineered Fill Placement and Compaction" section below.

Moisture-conditioned Soil Subgrade: Construct exterior concrete slabs-on-grade on properly moisture-conditioned subgrade soil. Prior to construction of the slabs, the moisture content of the subgrade soil should be tested by the geotechnical engineer. If the moisture content of the soil is less than 3 percent above the laboratory optimum moisture content, the subgrade should be scarified and moisture conditioned to between 3 and 6 percent above the laboratory optimum value, and be compacted to between 87 and 92 percent relative compaction. The prepared subgrade should be protected from drying prior to placing concrete for the slabs.

6.1.5 Materials for Engineered Fill

In general, on-site soils with an organic content of less than 3 percent by weight, free of deleterious materials or hazardous substances, and meeting the gradation requirements below may be used as engineered fill except where special material (such as "non-expansive" fill, capillary break material, etc.) is recommended.

Engineered fill material should not contain rocks or lumps larger than 3 inches in greatest dimension, should not contain more than 15 percent of the material larger than 1½ inches, and should contain at least 20 percent passing the No. 200 sieve. In addition to these requirements, import fill should have a low expansion potential as indicated by Plasticity Index of 15 or less (per ASTM D4318) or Expansion Index of less than 20 (per ASTM D4829).

Import fills should be approved by the geotechnical engineer prior to delivery to the site. At least 5 working days prior to importing to the site, a representative sample of each proposed import fill should be delivered to our laboratory for evaluation. Import fills should be tested and approved for the intended site use per the California Department of Toxic Substances Control (DTSC) guidelines.

6.1.6 Engineered Fill Placement and Compaction

Engineered fill should be placed in horizontal lifts each not exceeding 8 inches in thickness, moisture conditioned to the required moisture content, and mechanically compacted to the recommendations below. Relative compaction or compaction is defined as the in-place dry density of the compacted soil divided by the laboratory maximum dry density as determined by ASTM Test Method D1557, latest edition, expressed as a percentage. Moisture conditioning of soils should consist of adding water to the soils if they are too dry and allowing the soils to dry if they are too wet.

Engineered fills consisting of on-site clay soils should be compacted to between 87 and 92 percent relative compaction at moisture content between 3 and 6 percent above the laboratory optimum value. Engineered fills consisting of soils of low expansion potential, including "non-expansive" fill, should be compacted to at least 90 percent relative compaction at moisture content between 1 and 3 percent above the laboratory optimum value. In pavement areas, the upper 8 inches of subgrade soil should be compacted to at least 95 percent relative compaction. Aggregate base in vehicle pavement areas should be compacted at slightly above the optimum moisture content to at least 95 percent relative compaction.

6.1.7 Utility Trench Backfill

Backfilling of utility trenches in public right-of-way areas should comply with the City of San Jose standard specifications and details.

Backfilling of utility trenches in private areas may consist of bedding material extending from the bottom of the trench to about 1 foot above the top of pipe, and on-site or imported backfill material above the bedding to the proposed finish subgrade. Bedding may consist of free-draining sand (less than 5% passing a No. 200 sieve), lean concrete, or sand cement slurry. Sand, if used as bedding, should be compacted to at least 90 percent relative compaction. Backfill material may consist of on-site or imported soil, and should be compacted per recommendations in the "Engineered Fill Placement and Compaction" section above.

The backfill material should be placed in lifts each not exceeding 6 inches in uncompacted thickness. Thicker lifts may be used if the contractor can demonstrate that the recommended level of compaction can be achieved with the compaction equipment and procedures used. Compaction should be performed by mechanical means only. Water jetting or flooding to attain compaction of backfill should not be permitted.

6.1.8 Considerations for Soil Moisture and Seepage Control

Subgrade soil and engineered fill should be compacted at moisture content meeting our recommendations. Consideration should be given to reducing the potential for water infiltration from the exterior to under the building through utility lines crossing the building perimeter. In utility lines crossing beneath perimeter foundations, permeable backfill should be terminated at least 1 foot outside of the perimeter foundation. Impermeable material, such as concrete or clay soil, should be used for the entire trench depth to act as a seepage cutoff.

Where concrete slabs or pavements abut against landscaped areas, the base rock layer and subgrade soil should be protected against saturation. Water if allowed to seep into the subgrade soil or pavement section could reduce the service life of the improvements. Methods that may be considered to reduce infiltration of water include: 1) subdrains installed behind curbs and slabs in landscape areas; 2) vertical cut-offs, such as a deepened curb section, or equivalent, extending at least 2 inches into the subgrade soil; and 3) use of a drip or controlled irrigation system for landscape watering.

6.1.9 Wet Weather Construction

If site grading and construction is to be performed during the winter rainy months, the owner and contractors should be fully aware of the potential impact of wet weather. Rainstorms can cause delay to construction and damage to previously completed work by saturating compacted pads or subgrades, or flooding excavations.

Earthwork during rainy months will require extra effort and caution by the contractors. The contractors are responsible for protecting their work to avoid damage by rainwater. Standing pools of water should be pumped out immediately. Construction during wet weather conditions should be addressed in the project construction bid documents and specifications. We recommend the contractors submit a wet weather construction plan outlining procedures they will employ to protect their work and to minimize damage to their work by rainstorms.

6.2 Building Foundations

6.2.1 General

The proposed residential structures may be supported on post-tensioned (PT) slab foundations, bearing on properly moisture-conditioned and compacted soil. Minor structures, such as landscaping retaining walls, may be supported on conventional footing or drilled pier foundations. General recommendations for design of these foundations are presented below.

The Geotechnical Engineer should review the foundation plans and details before construction and observe the foundation excavations during construction to evaluate if the foundation excavations extend into suitable bearing material. Prior to placement of concrete, foundation excavations should be cleaned of loose soils. If unsuitable soils are encountered in the foundation excavations, the soils should be removed as recommended by the geotechnical engineer and replaced with approved material such as compacted engineered fill or lean concrete.

Foundation excavations should not be allowed to dry before placement of concrete. If visible cracks appear in the foundation excavations, the excavations should be thoroughly water conditioned beginning at least 2 days prior to placement of concrete to close all cracks. It is also important that the base of the foundation excavations not be allowed to become excessively wet, resulting in soft soils. Water should not be allowed to pond in the bottom of the excavations. Areas that become water damaged should be over-excavated to a firm base. The foundation excavations should be monitored by our representative for compliance with appropriate moisture control and to confirm the adequacy of the bearing materials.

To maintain the desired support, the bottom of foundations and other structural improvements adjacent to below-ground improvements, including utility trenches and bio-retention facilities, should be below an imaginary plane having an inclination of 1.5 horizontal to 1 vertical and extending upward from the bottom edge of the adjacent utility trenches or structures. If the footings are closer than the recommended distance, contact our office for recommendations.

6.2.2 Post-tensioned Slabs

The proposed residential buildings may each be supported on a post-tensioned (PT) slab foundation bearing on properly moisture-conditioned and compacted subgrade soil. Preparation of soil subgrade, moisture conditioning, and compaction of soil and engineered fill should be as recommended in the "Earthwork" section of this report. At least one week prior to PT slab construction, the moisture content of the subgrade soil should be evaluated. If the soil's moisture content is lower than the recommended value of at least 3 percent above the laboratory optimum moisture content, water should be added to bring the soil's moisture content to above the recommended value.

The following parameters may be used with the 2004 PTI "Design of Post-Tensioned Slabs-on-Ground, Third Edition" manual for design of the PT slabs.

Parameters for Design of Post-tensioned Slabs on On-site Expansive Soil	
Parameters	**PT on On-site Soil**
e_m (center lift)	8.8 feet
e_m (edge lift)	4.5 feet
y_m (center lift)	1.0 inch
y_m (edge lift)	1.6 inch

Allowable soil bearing pressure = 1,500 psf for dead plus live loads, with a one-third increase when including transient loads, such as wind or seismic.

A deepened edge, at least 6 inches wide, should be constructed along the perimeter of the PT slabs. The deepened edge should extend to at least 18 inches below the bottom of the PT slabs (see Figure 3). The deepened edge can help reduce moisture infiltration to under the PT slabs.

When interior building grades are higher than the exterior grades, the perimeter foundation elements should be designed to resist the lateral soil pressure and surcharge loads acting on the foundations. The bottom of the perimeter foundations should extend at least 18 inches below the lowest adjacent finish grades, excluding landscaping soils which are typically not compacted and should not be considered for structural support.

We understand the PT slabs will be constructed on 1 to 2 inches of sand over a 15-mil visqueen vapor barrier over compacted subgrade soil. Sand has been used for protection of the vapor barrier during construction and to allow dissipation of concrete mix water during curing. The use of sand, or equivalent material, should be determined by the project structural engineer or architect. A lower water-cement ratio (0.45 to 0.5) will help reduce the permeability of the concrete and, hence, vapor transmission through the slabs.

Settlements are expected to be primarily elastic. Post-construction total and differential settlements of the PT slabs are anticipated to be less than 1 inch and ½ inch, respectively.

6.2.3 Conventional Footings

Footings, continuous and isolated, may be used to support minor structures and landscaping retaining walls. Footings should bear on undisturbed on-site soil and/or properly compacted engineered fill. Preparation of soil subgrade, moisture conditioning, and compaction of soil and engineered fill should be as recommended in the "Earthwork" section of this report.

Footings may be designed for a net allowable bearing pressure of 3,000 pounds per square foot due to dead plus live loads, with a one-third increase when including transient loads such as wind or seismic. The footing bottom should be at least 18 inches below pad grade or lowest adjacent finish grade, whichever provides a deeper embedment. Foundations should be at least 12 inches wide and should be reinforced as determined by the project structural engineer.

Resistance to lateral loads may be developed from a combination of friction between the bottom of foundations and the supporting subgrade, and by passive resistance acting against the vertical sides of the foundations. Footings bearing on native soil or engineered fill may be designed using an ultimate friction coefficient of 0.25 between the foundations and supporting subgrade, and an ultimate passive resistance of 250 pounds per cubic foot (pcf, equivalent fluid weight) acting against the embedded sides of the foundations. The passive pressure can be assumed to act starting at the top of the lowest adjacent grade in paved areas. In unpaved areas, the passive pressure can be assumed to act starting at a depth of 1 foot below grade. It should be noted that the passive resistance value discussed above is only applicable where the concrete is placed directly against undisturbed soil or engineered fills. Voids created by the use of forms should be backfilled with property compacted engineered fill or with concrete.

Total post-construction settlement of the foundations under the building loads is anticipated to be up to about 1 inch, with up to about ½ inch of differential settlement over a distance of about 30 feet.

6.2.4 Drilled Pier Foundations

Drilled, cast-in-place, reinforced concrete piers should be designed to derive their vertical supporting capacity from "skin friction" using an allowable adhesion value of 500 psf between the pier shafts and the surrounding earth materials. This value is for dead plus live vertical loads, and may be increased by one-third when including transient loads, such as wind or seismic. The upper 1 foot of the pier and end bearing capacity of the piers should be ignored. Piers should have a diameter of 12 inches or greater. Center to center spacing of the piers should be at least 3 pier diameters. Reinforcement in the piers should be determined by the structural engineer.

Resistance to lateral loads may be calculated based on an ultimate passive soil pressure of 300 pcf (equivalent fluid weight) acting against 2 pier diameters, for level ground surface in front of the piers in the direction of load application. It should be noted that passive resistance is only applicable where the concrete is placed directly against undisturbed soil or engineered fill.

The presence of granular soils should be considered in the design and construction of the foundation piers because granular soils are prone to caving if the holes are not cased. Steel casing should be provided to keep the pier holes open.

6.3 Concrete Slabs-on-Grade

The interior building slabs will be post-tensioned concrete slabs.

Exterior concrete slabs-on-grade for this project will be limited to driveways and exterior flatwork such as patios and walkways. Concrete for driveways should be at least 6 inches thick and should be constructed on a 4-inch minimum thick section of Class 2 Aggregate Base over properly prepared subgrade soil, compacted as recommended in the “Earthwork” section of this report. Concrete for exterior patios and walkways should be at least 4 inches thick and should be constructed either on a layer of “non-expansive” fill at least 6 inches thick or on properly moisture-conditioned and compacted subgrade soil. Preparation of subgrade soil should be as recommended in the “Earthwork” section of this report. Design of reinforcement, joint spacing, etc. for concrete slabs is the responsibility of the design engineer.

If desired, exterior concrete slabs-on-grade may be cast free from adjacent foundations or other non-heaving edge restraints. This may be accomplished by using a strip of 1/2-inch asphalt-impregnated felt divider material between the slab edges and the adjacent structural elements. Frequent construction or control joints should be provided in all concrete slabs where cracking is objectionable. Continuous reinforcing or dowels at the construction and control joints will also aid in reducing uneven slab movements.

6.4 Retaining Walls

Retaining walls for this project are expected to be free-standing landscaping walls not a part of the proposed buildings, and we have anticipated these walls will have exposed height of about 3 to 5 feet. Retaining walls should be designed to resist lateral earth pressure and surcharge forces acting on the walls. Lateral pressures will depend on the degree of movement the walls are allowed (or desired), type of backfill, ground slope behind the walls, magnitude of external loads, and subsurface drainage provisions.

For static loading conditions, retaining walls may be designed using at-rest or active soil pressure. At-rest soil pressure should be used for walls where movements at the top of walls are restrained or undesirable. Wall movements could cause settlement of backfill and structures supported on the backfill. Active soil pressure may be used for retaining walls where the top of walls is free to deflect and resulting movement of the backfill is acceptable. The at-rest and active soil pressures given below are for level backfill surface and for both drained and undrained backfill conditions.

Condition	Drained Backfill(1)	Undrained Backfill(2)
At-rest	65 pcf	95 pcf
Active	45 pcf	85 pcf

Notes:
1. Lateral soil pressures for drained backfill may be used above groundwater level with subsurface drainage provided behind walls.
2. Lateral soil pressures for undrained backfill should be used below groundwater level or where subsurface drainage is not provided behind walls.
3. Contact our office if walls are to be designed for seismic surcharge pressure.

Pressures due to static external loads should be added to the soil pressures recommended above in the wall design. For uniform vertical load at the ground surface, the additional lateral pressure on the walls should be calculated as a uniform pressure equal to the magnitude of the vertical load multiplied by a factor. For level backfill slope, the factor is 0.38 for active soil condition and 0.52 for at-rest soil condition. For other slope inclinations and other types of surcharge loads, such as vehicle loads, point loads, strip loads, consult our office for specific recommendations.

Foundations for retaining walls may consist of footings designed using the recommendations in the Foundations Section of this report.

Wall Drains (drained backfill condition): For walls designed using lateral soil pressures under drained backfill condition, a subsurface drain should be installed behind each wall extending from the wall bottom to about 1 foot below finished grade. The drain should consist of a 12-inch minimum wide blanket of drainage material consisting of either Class 2 Permeable material (Caltrans Standard Specifications, Section 68) or clean, 1/2 to 3/4-inch maximum size crushed rock or gravel. If crushed rock or gravel is used, it should be encapsulated in a geotextile filter fabric, such as Mirafi 140N or equivalent. Filter fabric is optional if Class 2 Permeable material is used. The top 1 foot below finish grade should be backfilled with compacted clayey soil to reduce infiltration of surface water.

A 4-inch minimum diameter, perforated, schedule 40 PVC (or equivalent) pipe should be installed (with perforations facing down) along the base of each wall on a 2-inch thick bed of drain rock, regardless whether drain rock or pre-fabricated drainage panel is used. The pipes should be sloped to drain by gravity to a proper collection system and be discharged at a proper outlet as designed by the project Civil Engineer.

Wall Compaction: Backfill against retaining walls should be compacted as discussed in the "Earthwork" Section of this report. Over-compaction should be avoided because increased compaction effort can result in lateral pressures significantly higher than those recommended above. Backfill placed within 3 feet of the walls should be compacted with hand-operated equipment.

6.5 Vehicle Pavements

Vehicle pavements for this project will include interior street, primarily serving automobiles and light pickup trucks, with occasional heavy vehicles, such as delivery and garbage trucks. It the pavements are constructed prior to completion of construction, the pavements will be subject to construction traffic including heavy delivery and concrete truck, and construction equipment.

An R-value of 6 was measured on a bulk sample of soil collected from the site. For design purposes, an R-value of 6 was used to calculate the pavement sections tabulated below using the Caltrans pavement section design procedures.

DESIGN TRAFFIC INDEX	HOT MIX ASPHALT (inches)	CLASS 2 AGGREGATE BASE (inches)	TOTAL (inches)
5.0	3.0	9.5	12.5
5.5	3.5	10.5	14.0
6.0	4.0	11.5	15.5
6.5	4.0	13.5	17.5
7.0	4.5	14.5	19.0

Pavement sections should be constructed on soil subgrades that have been prepared as outlined in the "Earthwork" section of this report. The upper 8 inches of soil subgrade in pavement areas should be compacted to a minimum of 95 percent relative compaction. The full section of aggregate base and aggregate subbase should be compacted to a minimum of 95 percent relative compaction. Evaluation of relative compaction should be based on ASTM D1557, latest edition. The Class 2 Aggregate Base material should conform to Section 26 of the Caltrans Standard Specifications and the Class 2 Aggregate Subbase material should conform to Section 25 of the Caltrans Standard Specifications.

6.6 Surface Drainage

Engineering design of grading and drainage is the responsibility of the project civil engineer. Sufficient surface drainage should be provided to direct water away from buildings, foundations, concrete slabs-on-grade and pavements, and towards suitable collection and discharge facilities. Ponding of surface water should be avoided by establishing positive drainage away from all improvements.

7 PLAN REVIEW, EARTHWORK AND FOUNDATION OBSERVATION

Post-report geotechnical services by Geo-Logic Associates (GLA), typically consisting of pre-construction design consultations and reviews and construction observation and testing services, are necessary for GLA to confirm the recommendations contained in this report. This report is based on limited sampling and investigation, and by those constraints may not have discovered local anomalies or other varying conditions that may exist on the project site. Therefore, this report is only preliminary until GLA can confirm that actual conditions in the ground conform to those anticipated in the report. Accordingly, as an integral part of this report, GLA recommends post-report, construction related geotechnical services to assist the project team during design and construction of the project. GLA requires that it perform these services if it is to remain as the project Geotechnical Engineer-of-record.

During design, GLA can provide consultation and supplemental recommendations to assist the project team in design and value engineering, especially if the project design has been modified after completion of our report. It is impossible for us to anticipate every design scenario and use of construction materials during preparation of our report. Therefore, retaining GLA to provide post-report consultation will help address design changes, answer questions and evaluate alternatives proposed by the project designers and contractors.

Prior to issuing project plans and specifications for construction bidding purposes, GLA should review the grading, drainage and foundation plans and the project specifications to determine if the intent of our recommendations has been incorporated in these documents. We have found that such a review process will help reduce the likelihood of misinterpretation of our recommendations which may cause construction delay and additional cost.

Construction phase services can include, among other things, the observation and testing during site clearing, stripping, excavation, mass grading, subgrade preparation, fill placement and compaction, backfill compaction, foundation construction and pavement construction activities.

Geo-Logic Associates would be pleased to provide cost proposals for follow-up geotechnical services. Post-report geotechnical services may include additional field and laboratory services.

8 LIMITATIONS

In preparing the findings and professional opinions presented in this report, Geo-Logic Associates (GLA) has endeavored to follow generally accepted principles and practices of the engineering geologic and geotechnical engineering professions in the area and at the time our services were performed. No warranty, express or implied, is provided.

The conclusions and recommendations contained in this report are based, in part, on information that has been provided to us. In the event that the general development concept or general location and type of structures are modified, our conclusions and recommendations shall not be considered valid unless we are retained to review such changes and to make any necessary additions or changes to our recommendations. To remain as the project Geotechnical Engineer-of-record, GLA must be retained to provide geotechnical services as discussed under the Post-report Geotechnical Services section of this report.

Subsurface exploration is necessarily confined to selected locations and conditions may, and often do, vary between these locations. Should conditions different from those described in this report be encountered during project development, GLA should be consulted to review the conditions and determine whether our recommendations are still valid. Additional exploration, testing, and analysis may be required for such evaluation.

Should persons concerned with this project observe geotechnical features or conditions at the site or surrounding areas which are different from those described in this report, those observations should be reported immediately to GLA for evaluation.

It is important that the information in this report be made known to the design professionals involved with the project, that our recommendations be incorporated into project drawings and documents, and that the recommendations be carried out during construction by the contractor and subcontractors. It is not the responsibility of GLA to notify the design professionals and the project contractors and subcontractors.

The findings, conclusions, and recommendations in this report are applicable only to the specific project development on this specific site. These data should not be used for other projects, sites, or purposes unless they are reviewed by GLA or a qualified geotechnical professional.

Report prepared by,

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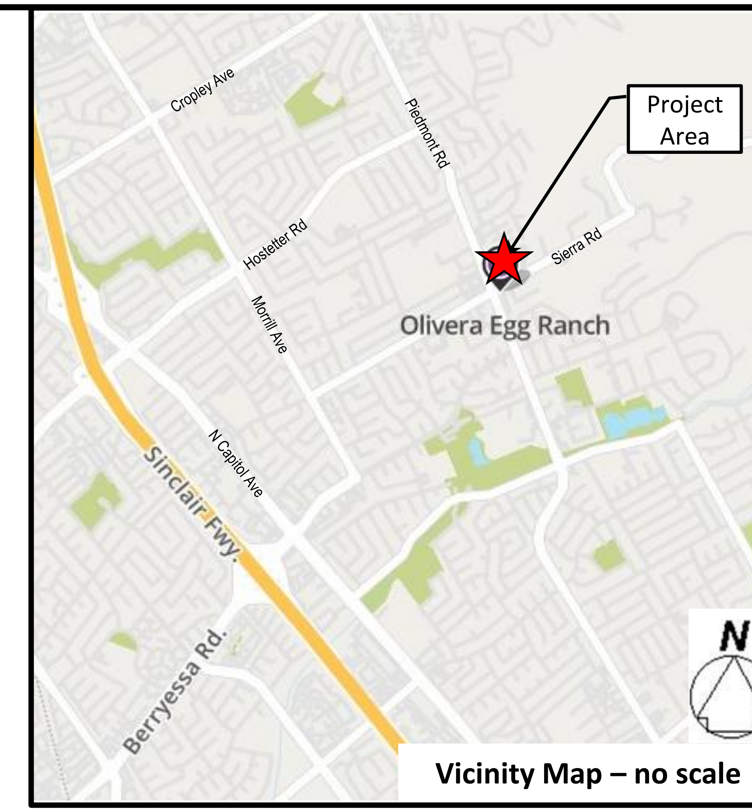
Chalerm (Beeson) Liang
GE 2031



Legend

Exploratory drill hole

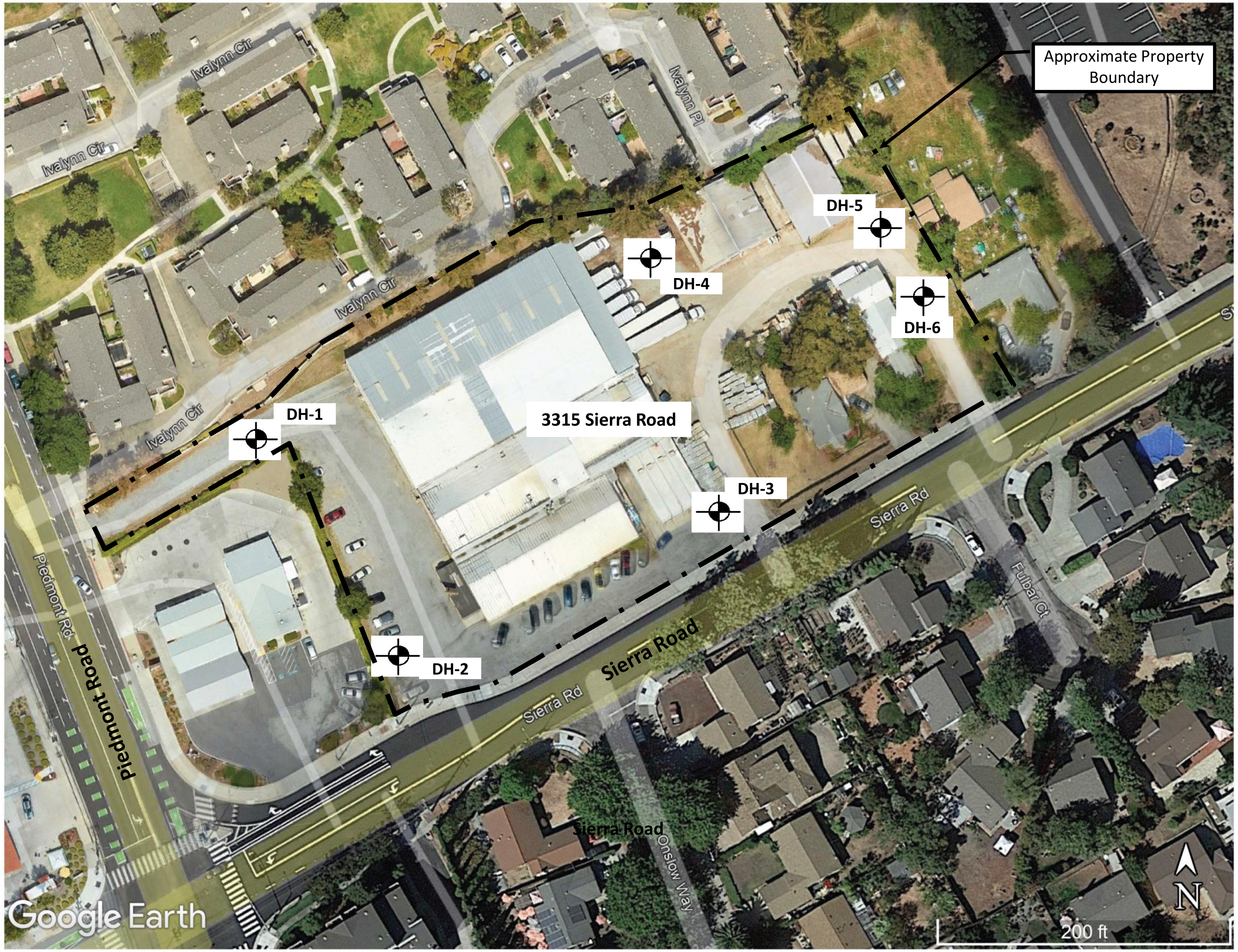
N

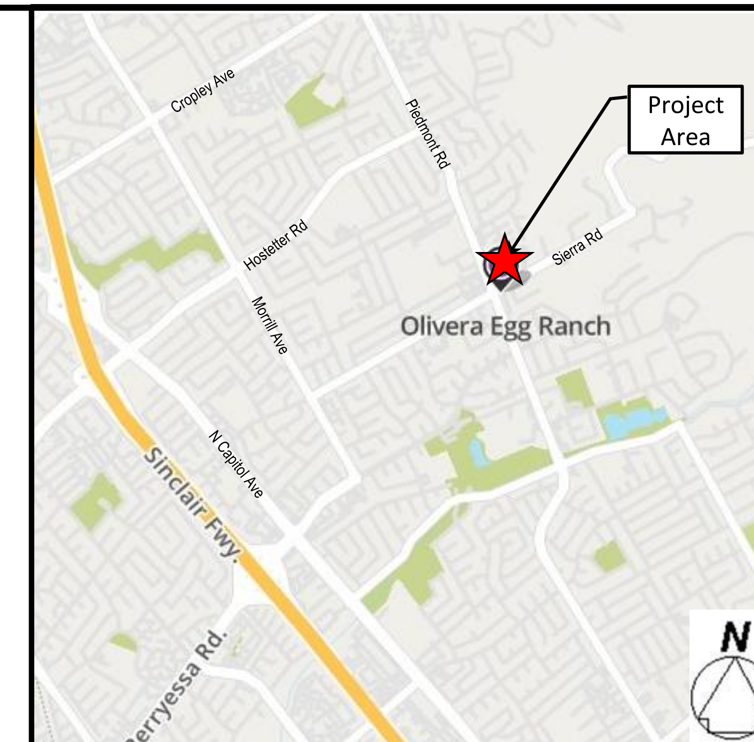
Approximate Scale (Feet)

0 50 100 150

Base: 3315 Sierra Road Site Plan, prepared by Civil Engineering Associates, dated 2/14/2023..

Geo-Logic ASSOCIATES	6300 San Ignacio Avenue, Suite A San Jose, California 95119 Phone (408) 778-2818	Drafted By: Date: March 2023 Checked By: Revision:	**SITE PLAN (PROPOSED DEVELOPMENT)** Proposed Residential Development 3315 Sierra Road San Jose, California	**FIGURE** 1 **PROJECT** PA22.1048





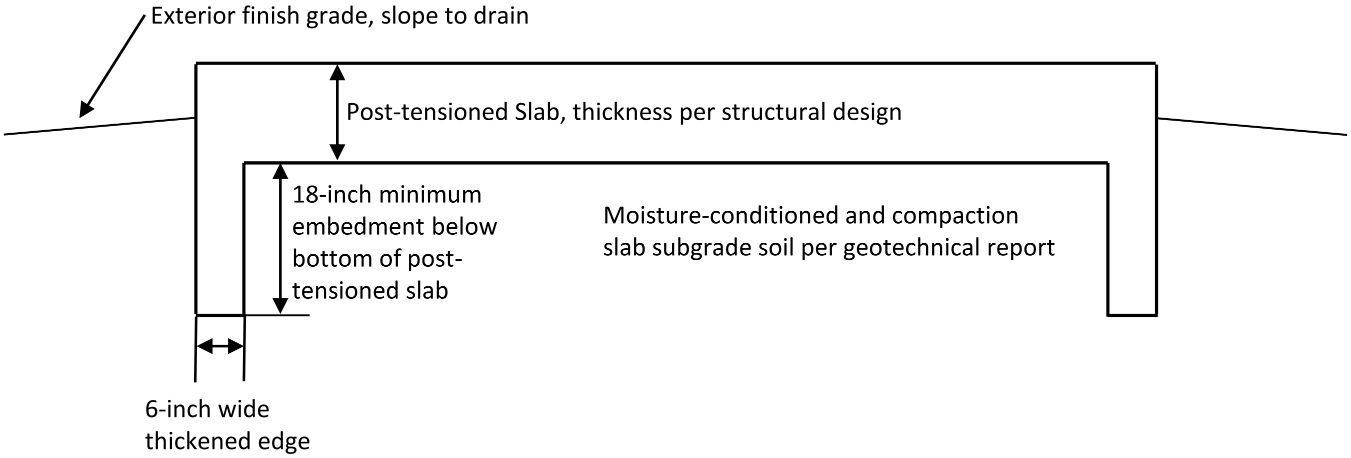
Vicinity Map – no scale

Legend

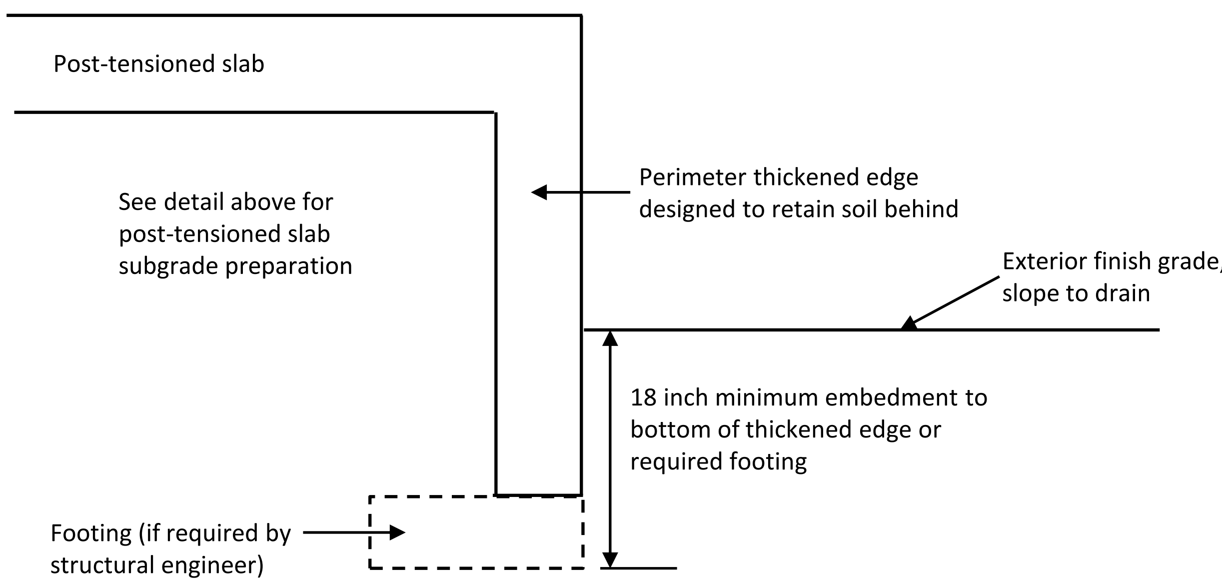
Exploratory drill hole

Base: Google Earth.

Geo-Logic ASSOCIATES	6300 San Ignacio Avenue, Suite A San Jose, California 95119 Phone (408) 778-2818	Drafted By: Date: March 2023 Checked By: Revision:	**SITE PLAN (EXISTING CONDITIONS)** Proposed Residential Development 3315 Sierra Road San Jose, California	**FIGURE** 2 **PROJECT** PA22.1048



Subgrade Preparation and Thickened Edge for Post-tensioned Slab Foundations



Foundation Embedment for Post-tensioned Slabs with Differential Grades

Note:

1. Refer to geotechnical report for detailed recommendations.

Schematic Only – Not to Scale

Geo-Logic ASSOCIATES
6300 San Ignacio Avenue, Suite A
San Jose, California 95119
Phone (408) 778-2818

Compiled by:	Date: March 2023
Reviewed by:	Revision:

POST-TENSIONED SLAB TYPICAL SECTION

Proposed Residential Development
3315 Sierra Road
San Jose, California

FIGURE 3

PROJECT PA22.1048

APPENDIX A

KEYS TO SOIL CLASSIFICATION

AND

DRILL HOLE LOGS

KEY TO SOIL CLASSIFICATION - FINE GRAINED SOILS

(50% OR MORE IS SMALLER THAN NO. 200 SIEVE SIZE)

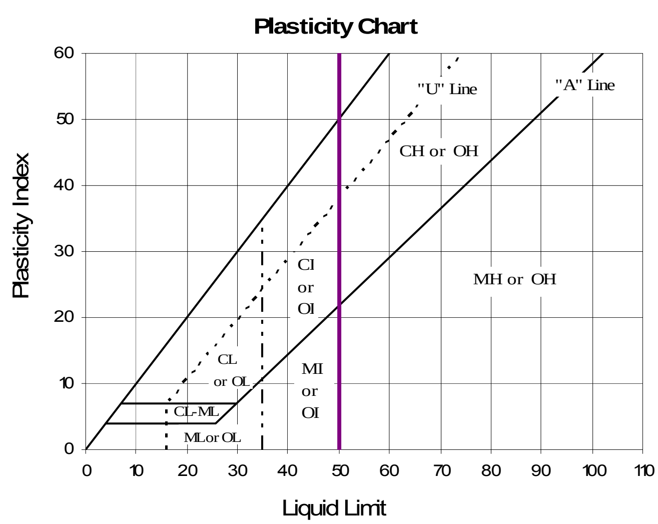
(modified from ASTM D2487 to include fine grained soils with intermediate plasticity)

MAJOR DIVISIONS			GROUP SYMBOLS	GROUP NAMES
SILTS AND CLAYS (Liquid Limit less than 35) Low Plasticity	Inorganic	PI < 4 or plots below “A” line	ML	Silt, Silt with Sand or Gravel, Sandy or Gravelly Silt, Sandy or Gravelly Silt with Sand or Gravel
	Inorganic	PI > 7 or plots on or above “A” line	CL	Lean Clay, Lean Clay with Sand or Gravel, Sandy or Gravelly Lean Clay, Sandy or Gravelly Lean Clay with Sand or Gravel
	Inorganic	PI between 4 and 7	CL-ML	Silty Clay, Silty Clay with Sand or Gravel, Sandy or Gravelly Silty Clay, Sandy or Gravelly Silty Clay with Sand or Gravel
	Organic	See footnote 3	OL	Organic Silt (below “A” Line) or Organic Clay (on or above “A” Line) (1,2)
SILTS AND CLAYS (35 ≤ Liquid Limit < 50) Intermediate Plasticity	Inorganic	PI < 4 or plots below “A” line	MI	Silt, Silt with Sand or Gravel, Sandy or Gravelly Silt, Sandy or Gravelly Silt with Sand or Gravel
	Inorganic	PI > 7 or plots on or above “A” line	CI	Clay, Clay with Sand or Gravel, Sandy or Gravelly Clay, Sandy or Gravelly Clay with Sand or Gravel
	Organic	See footnote 3	OI	Organic Silt (below “A” Line) or Organic Clay (on or above “A” Line) (1,2)
SILTS AND CLAYS (Liquid Limit 50 or greater) High Plasticity	Inorganic	PI plots below “A” line	MH	Elastic Silt, Elastic Silt with Sand or Gravel, Sandy or Gravelly Elastic Silt, Sandy or Gravelly Elastic Silt with Sand or Gravel
	Inorganic	PI plots on or above “A” line	CH	Fat Clay, Fat Clay with Sand or Gravel, Sandy or Gravelly Fat Clay, Sandy or Gravelly Fat Clay with Sand or Gravel
	Organic	See note 3 below	OH	Organic Silt (below “A” Line) or Organic Clay (on or above “A” Line) (1,2)

1. If soil contains 15% to 29% plus No. 200 material, include “with sand” or “with gravel” to group name, whichever is predominant.
2. If soil contains ≥30% plus No. 200 material, include “sandy” or “gravelly” to group name, whichever is predominant. If soil contains ≥15% of sand or gravel sized material, add “with sand” or “with gravel” to group name.
3. Ratio of liquid limit of oven dried sample to liquid limit of not dried sample is less than 0.75.

CONSISTENCY	UNCONFINED SHEAR STRENGTH (KSF)	STANDARD PENETRATION (BLOWS/FOOT)
VERY SOFT	< 0.25	< 2
SOFT	0.25 – 0.5	2 – 4
FIRM	0.5 – 1.0	5 – 8
STIFF	1.0 – 2.0	9 – 15
VERY STIFF	2.0 – 4.0	16 – 30
HARD	> 4.0	> 30

MOISTURE	CRITERIA
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp, but no visible water
Wet	Visible free water, usually soil is below the water table



GEO-LOGIC ASSOCIATES

KEY TO SOIL CLASSIFICATION – COARSE GRAINED SOILS
(MORE THAN 50% IS LARGER THAN NO. 200 SIEVE SIZE)
(modified from ASTM D2487 to include fines with intermediate plasticity)

MAJOR DIVISIONS			GROUP SYMBOLS	GROUP NAMES[1]
GRAVELS (more than 50% of coarse fraction is larger than No. 4 sieve size)	Gravels with less than 5% fines	Cu ≥ 4 and 1 ≤ Cc ≤ 3	GW	Well Graded Gravel, Well Graded Gravel with Sand
		Cu < 4 and/or 1 > Cc > 3	GP	Poorly Graded Gravel, Poorly Graded Gravel with Sand
	Gravels with 5% to 12% fines	ML, MI or MH fines	GW-GM	Well Graded Gravel with Silt, Well Graded Gravel with Silt and Sand
			GP-GM	Poorly Graded Gravel with Silt, Poorly Graded Gravel with Silt and Sand
		CL, CI or CH fines	GW-GC	Well Graded Gravel with Clay, Well Graded Gravel with Clay and Sand
			GP-GC	Poorly Graded Gravel with Clay, Poorly Graded Gravel with Clay and Sand
	Gravels with more than 12% fines	ML, MI or MH fines	GM	Silty Gravel, Silty Gravel with Sand
		CL, CI or CH fines	GC	Clayey Gravel, Clayey Gravel with Sand
		CL-ML fines	GC-GM	Silty Clayey Gravel; Silty, Clayey Gravel with Sand
SANDS (50% or more of coarse fraction is smaller than No. 4 sieve size)	Sands with less than 5% fines	Cu ≥ 6 and 1 ≤ Cc ≤ 3	SW	Well Graded Sand, Well Graded Sand with Gravel
		Cu < 6 and/or 1 > Cc > 3	SP	Poorly Graded Sand, Poorly Graded Sand with Gravel
	Sands with 5% to 12% fines	ML, MI or MH fines	SW-SM	Well Graded Sand with Silt, Well Graded Sand with Silt and Gravel
			SP-SM	Poorly Graded Sand with Silt, Poorly Graded Sand with Silt and Gravel
		CL, CI or CH fines	SW-SC	Well Graded Sand with Clay, Well Graded Sand with Clay and Gravel
			SP-SC	Poorly Graded Sand with Clay, Poorly Graded Sand with Clay and Gravel
	Sands with more than 12% fines	ML, MI or MH fines	SM	Silty Sand, Silty Sand with Gravel
		CL, CI or CH fines	SC	Clayey Sand, Clayey Sand with Gravel
		CL-ML fines	SC-SM	Silty, Clayey Sand; Silty, Clayey Sand with Gravel

US STANDARD SIEVES 3 Inch ¾ Inch No. 4 No. 10 No. 40 No. 200

	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES & BOULDERS	GRAVELS		SANDS			SILTS AND CLAYS

RELATIVE DENSITY (SANDS AND GRAVELS)	STANDARD PENETRATION (BLOWS/FOOT)
Very Loose	0 - 4
Loose	5 – 10
Medium Dense	11 – 30
Dense	31 - 50
Very Dense	50+

1. Add "with sand" to group name if material contains 15% or greater of sand-sized particle. Add "with gravel" to group name if material contains 15% or greater of gravel-sized particle.

MOISTURE	CRITERIA
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp, but no visible water
Wet	Visible free water, usually soi is below the water table

DATE: 11/11/2022	**LOG OF EXPLORATORY DRILL HOLE**	**DH- 1**

PROJECT NAME:	3315 Sierra Road, San Jose, CA	**PROJECT NUMBER:**	PA22.1048
DRILL RIG:	Mobile B-61, 140-lb auto hammer	**LOGGED BY:**	FS
HOLE DIAMETER:	8-inch hollow stem auger	**HOLE ELEVATION:**	---
SAMPLER:	D = 3" OD, 2½" ID Split-spoon X = 2½" OD, 2" ID Split-spoon I = Standard Penetrometer (2" OD SPT) S = Slough in sample	**GROUND WATER DEPTH:**	**Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
PAVEMENT (±0.25" AC over ±3" AB)												
SANDY CLAY: Dark yellowish brown (10YR 4/4), moist, very stiff	Cl	1										
		2	S D D	26	2.25			20				
		3										
hard		4	S D D	23	4.5+ 4.5+			17		103		
		5										
		6										
		7										
		8										
		9	S D D	27	4.5+ 4.5+			18		104		
		10										
		11										
CLAY: Yellowish brown (10YR 5/6), moist, hard	Cl	12										
		13										
		14	S D D	42	4.5+ 4.5+			17		111		
		15										
		16										
		17										
		18										
very stiff to hard, with caliche veins		19	S D D	32	4.0 3.5			19		105		
		20										

DATE: 11/11/2022	LOG OF EXPLORATORY DRILL HOLE	DH- 1

PROJECT NAME: 3315 Sierra Road, San Jose, CA **PROJECT NUMBER:** PA22.1048

DRILL RIG: Mobile B-61, 140-lb auto hammer **LOGGED BY:** FS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAY (continued)	CI											
		21										
		22										
		23										
hard		24	S D D	30	4.5+ 4.5+			16		114		
		25										
		26										
		27										
CLAYEY SAND with GRAVEL: Dark yellowish brown (10YR 4/4), moist, dense; fine to coarse sand, with fine to coarse gravel, sand and gravel are angular to sub-angular	SC	28										
		29	S D D	38		12		7		111		
		30										
		31										
		32										
CLAY: Dark yellowish brown (10YR 4/4), moist, hard, with caliche	CI	33										
		34	S D D	50	4.5+ 4.5+			18		112		
		35										
		36										
		37										
		38										
		39	S D D	78	4.5+ 4.5+			12		120		
		40										

DATE: 11/11/2022	LOG OF EXPLORATORY DRILL HOLE	DH- 1
PROJECT NAME: 3315 Sierra Road, San Jose, CA	PROJECT NUMBER: PA22.1048	
DRILL RIG: Mobile B-61, 140-lb auto hammer	LOGGED BY: FS	
HOLE DIAMETER: 8-inch hollow stem auger	HOLE ELEVATION: ---	
SAMPLER: D = 3" OD, 2½" ID Split-spoon; X = 2½" OD, 2" ID Split-spoon; I = Standard Penetrometer (2" OD SPT); S = Slough in sample	GROUND WATER DEPTH: Initial: --- Final: ---	

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAY (continued)	CI	41										
		42										
CLAYEY SAND: Yellowish brown (10YR 5/6), moist, dense; fine to coarse angular to sub-angular sand	SC	43										
		44	SDD	48		26		12		107		
BOTTOM OF HOLE = 45 FEET **No Groundwater Encountered**		45										
		46										
		47										
		48										
		49										
		50										
		51										
		52										
		53										
		54										
		55										
		56										
		57										
		58										
		59										
		60										

DATE: 11/11/2022	LOG OF EXPLORATORY DRILL HOLE	DH- 2

PROJECT NAME:	3315 Sierra Road, San Jose, CA	**PROJECT NUMBER:**	PA22.1048
DRILL RIG:	Mobile B-61, 140-lb auto hammer	**LOGGED BY:**	FS
HOLE DIAMETER:	8-inch hollow stem auger	**HOLE ELEVATION:**	---
SAMPLER:	D = 3" OD, 2½" ID Split-spoon X = 2½" OD, 2" ID Split-spoon I = Standard Penetrometer (2" OD SPT) S = Slough in sample	**GROUND WATER DEPTH:**	**Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
PAVEMENT (±1" AC over ±8" AB)												
FILL, CLAYEY SAND with GRAVEL: Dark brown (10YR 3/4), moist, medium dense	SC	1	S									
			D					10		95		
		2		12								
SANDY CLAY to CLAYEY SAND: Dark yellowish brown (10YR 4/4), moist, stiff to very stiff clay to medium dense sand; fine to coarse sand	CI/ SC		D			50	41	14	24	96		
		3										
		4	S									
SANDY CLAY: Dark yellowish brown (10YR 4/4), moist, very stiff	CI		D	6	2.25							
			D		3.5			18		93		
		5										
		6										
		7										
		8										
CLAY: Dark yellowish brown (10YR 4/4), moist, very stiff	CI		S									
		9	D	10	3.25							
			D		2.5			21		101		
		10										
		11										
		12										
		13										
		14	S									
stiff to very stiff			D	20	2.0							
			D		2.0			22		104		
		15										
		16										
very stiff												
CLAYEY SAND with GRAVEL: Dark yellowish brown (10YR 4/4), moist, medium dense; fine to coarse sand , with fine gravel; sand and gravel are angular to subangular		17										
		18										
		19	S									
			D	30	4.0							
BOTTOM OF HOLE @ 20 FEET	SC		D					14		115		
No Groundwater Encountered		20										

GEO-LOGIC ASSOCIATES	**PAGE:** 1 of 1

DATE: 11/11/2022	**LOG OF EXPLORATORY DRILL HOLE**	**DH- 3**

PROJECT NAME: 3315 Sierra Road, San Jose, CA **PROJECT NUMBER:** PA22.1048

DRILL RIG: Mobile B-61, 140-lb auto hammer **LOGGED BY:** FS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
PAVEMENT (±2" AC over ±6" AB)												
SANDY CLAY: Dark yellowish brown (10YR 4/4), moist, hard	CI	1	S									
		2	D D	23	4.5 4.5			18				
		3										
very stiff		4	S D D	17	3.25 3.5			21		98		
		5										
		6										
		7										
		8										
very stiff to hard		9	S D D	24	3.25 4.5+			18		110		
		10										
		11										
CLAY: Dark yellowish brown (10YR 4/4), moist, hard	CI	12										
		13										
		14	S D D	28	4.5+			17		111		
		15										
		16										
		17										
		18										
		19	S D D	24	4.0 4.5			18		110		
BOTTOM OF HOLE @ 20 FEET		20										
No Groundwater Encountered												

GEO-LOGIC ASSOCIATES **PAGE:** 1 of 1

DATE: 11/11/2022	LOG OF EXPLORATORY DRILL HOLE	DH- 4

PROJECT NAME: 3315 Sierra Road, San Jose, CA **PROJECT NUMBER:** PA22.1048

DRILL RIG: Mobile B-61, 140-lb auto hammer **LOGGED BY:** FS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
PAVEMENT (±2" AC over ±11" AB)												
		1										
SANDY CLAY: Very dark brown (10YR 2/2), moist, very stiff to hard	CI	2	S									
		3	D	23	3.5			18		109		
			D		4.0							
hard		4	S									
			D	24	4.5							
		5	D		4.5+			17		110		
		6										
		7										
CLAYEY SAND with GRAVEL: Dark yellowish brown (10YR 4/4), moist, dense; fine to coarse sand, with fine to coarse gravel, sand and gravel are angular to subangular	SC	8										
		9	S									
			D	34								
		10	D			26		12		105		
		11										
CLAY: Dark yellowish brown (10YR 4/4), moist, hard	CI	12										
		13										
		14	S									
			D	33	4.5							
		15	D		4.5+			20		109		
		16										
very stiff		17										
CLAYEY SAND with GRAVEL: Dark yellowish brown (10YR 4/4), moist, medium dense; fine to coarse sand, with fine gravel, sand and gravel are angular to subangular		18										
		19	S									
			D	24	2.5							
BOTTOM OF HOLE @ 20 FEET	SC	20	D			17		15		109		
No Groundwater Encountered												

GEO-LOGIC ASSOCIATES **PAGE:** 1 of 1

DATE: 11/11/2022	LOG OF EXPLORATORY DRILL HOLE	DH- 5

PROJECT NAME: 3315 Sierra Road, San Jose, CA **PROJECT NUMBER:** PA22.1048

DRILL RIG: Mobile B-61, 140-lb auto hammer **LOGGED BY:** FS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
SANDY CLAY: Black (10YR 2/1), moist, hard	Cl											
		1	S									
		2	D D	28	4.25 4.25			20		103		
		3										
dark yellowish brown (10YR 4/4)		4	S									
		5	D D	26	4.0 4.0			20		103		
		6										
		7										
		8										
very stiff		9	S D D	24	3.5 2.75			18		109		
		10										
		11										
		12										
		13										
dark grayish brown (10YR 4/2), hard		14	S D D	30	4.5+ 4.5+			18		108		
		15										
		16										
stiff		17										
CLAYEY SAND with GRAVEL: Dark yellowish brown (10YR 4/4), moist, medium dense; fine to coarse sand, with fine gravel, sand and gravel are angular to subangular		18										
		19	S D D	12	1.5			19		100		
BOTTOM OF HOLE @ 20 FEET	SC	20										
No Groundwater Encountered												

GEO-LOGIC ASSOCIATES **PAGE:** 1 of 1

DATE: 11/11/2022	LOG OF EXPLORATORY DRILL HOLE	DH- 6

PROJECT NAME: 3315 Sierra Road, San Jose, CA **PROJECT NUMBER:** PA22.1048

DRILL RIG: Mobile B-61, 140-lb auto hammer **LOGGED BY:** FS

HOLE DIAMETER: 8-inch hollow stem auger **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
SANDY CLAY: Black (10YR 2/1), moist, hard	Cl											
		1	S									
		2	D D	26	4.5+	60	47	15	29	103		
		3										
dark yellowish brown (10YR 4/4)		4	S D D	28	4.5+ 4.5+			12		100		
		5										
		6										
		7										
		8										
		9	S D D	22	4.5+ 4.5+			19		105		
		10										
		11										
		12										
		13										
		14	S D D	32	4.5+			13		114		
CLAYEY SAND with GRAVEL: Dark yellowish brown (10YR 4/4), moist, medium dense to dense; fine to coarse sand, with fine to coarse gravel, sand and gravel are angular to subangular	SC	15										
		16										
		17										
		18										
		19	S D D	41				10		113		
BOTTOM OF HOLE @ 20 FEET		20										
No Groundwater Encountered												

APPENDIX B

LABORATORY TEST RESULTS

ATTERBERG LIMITS
Summary Report
ASTM D-4318

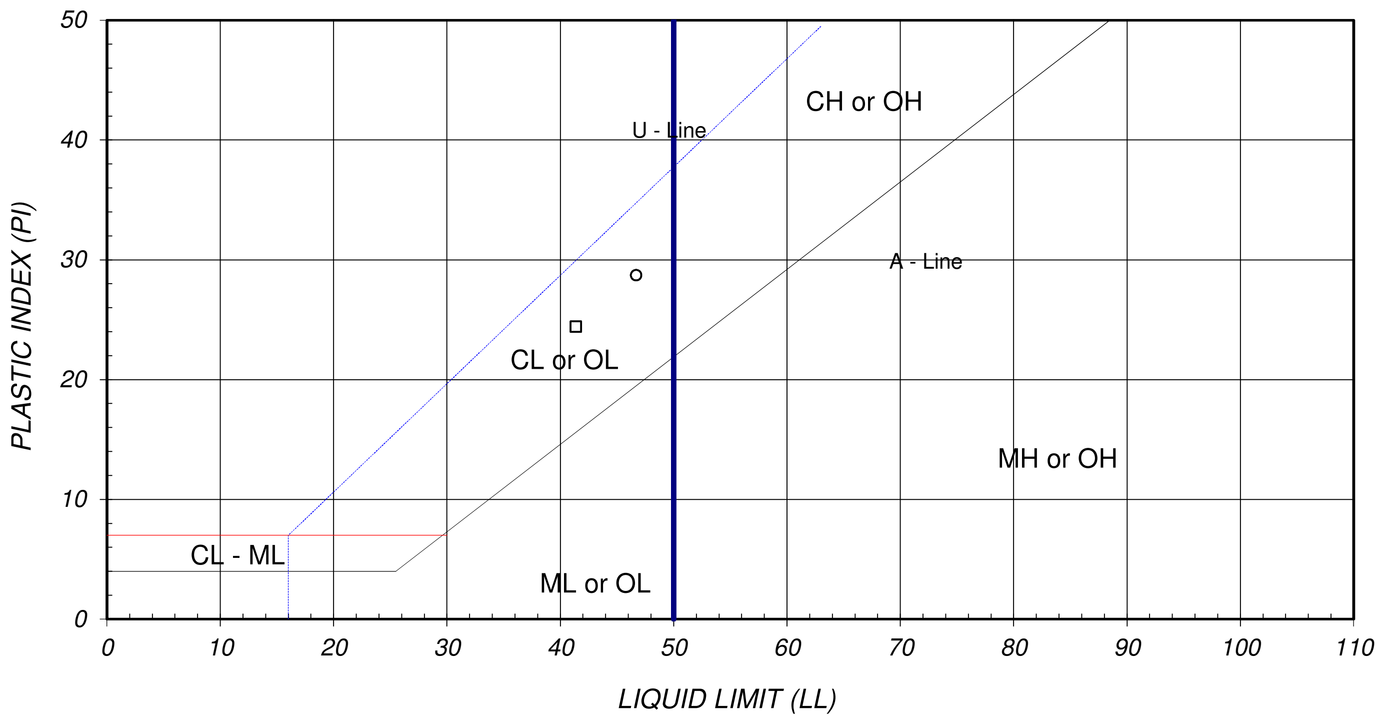
Geo-Logic ASSOCIATES

Client :	Project No:	Lab Log No.:
Robson Homes LLC	PA22.1048.00	4933

Project Name:	Report Date:
3315 Sierra Road San Jose	November 28, 2022

LSN	SYMBOL	SAMPLE IDENTIFICATION	SAMPLE DESCRIPTION	LIQUID LIMIT	PLASTIC LIMIT	PLASTIC INDEX
4933A	□	DH-2 @ 2-2.5	brown sandy lean clay (CL)	41	17	24
4933B	○	DH-6 @ 2-2.5	brown sandy lean clay (CL)	47	18	29

PLASTICITY CHART



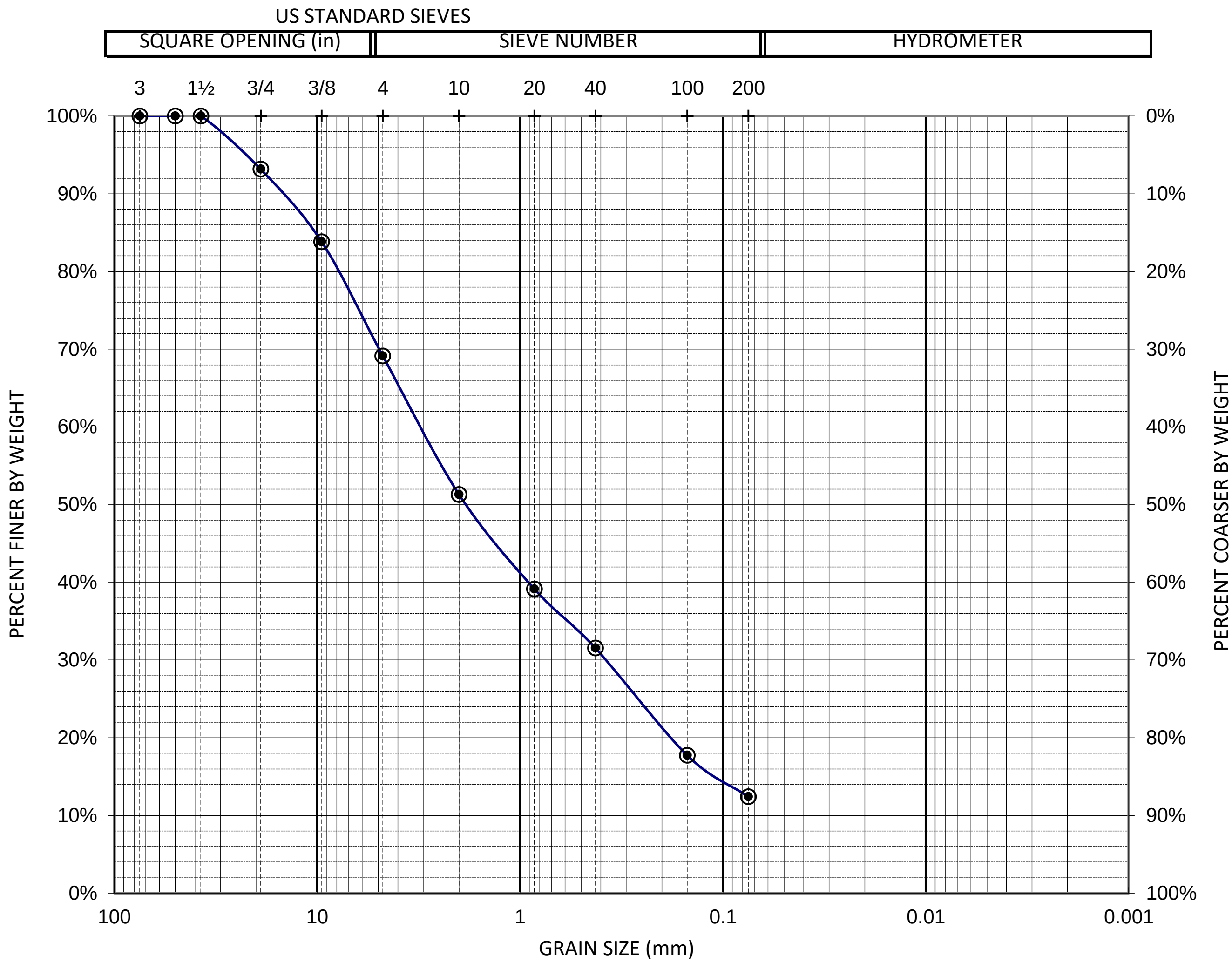
This testing is based upon accepted industry practice as well as the test method listed. These results apply only to the samples supplied and tested for the above referenced job.

GRAIN SIZE TEST RESULTS

PROJECT NAME	3315 Sierra Road					**PROJECT No.**	PA22.1048
DRILL HOLE No.	DH-1	**DEPTH (ft)**	29	**SAMPLE**	0	**DATE OF TEST**	11/16/2022

SOURCE/QUARRY: ---

DESCRIPTION OF SOIL: Clayey Sand with Gravel



	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES	GRAVEL		SAND			SILT & CLAY
	30.9%		56.7%			12.4%

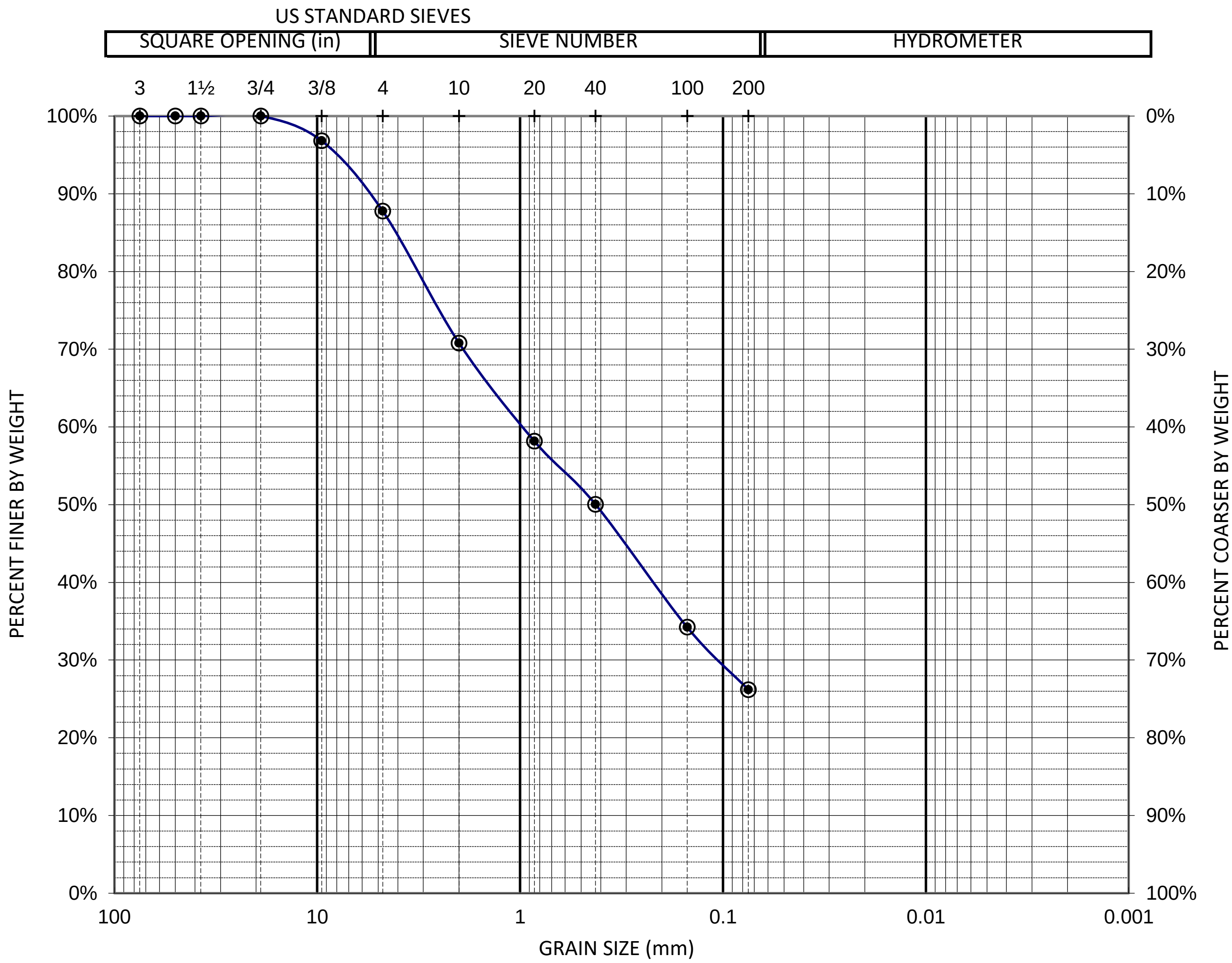
REMARKS:

GRAIN SIZE TEST RESULTS

PROJECT NAME	3315 Sierra Road					**PROJECT No.**	PA22.1048
DRILL HOLE No.	DH-1	**DEPTH (ft)**	44.5	**SAMPLE**	0	**DATE OF TEST**	11/16/2022

SOURCE/QUARRY: ---

DESCRIPTION OF SOIL: Clayey Sand



	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES	GRAVEL		SAND			SILT & CLAY
	12.2%		61.6%			26.2%

REMARKS:

GEO-LOGIC ASSOCIATES | **FIGURE** **B-3**

RevOct20070

PARTICLE SIZE ANALYSIS

Geo-Logic ASSOCIATES

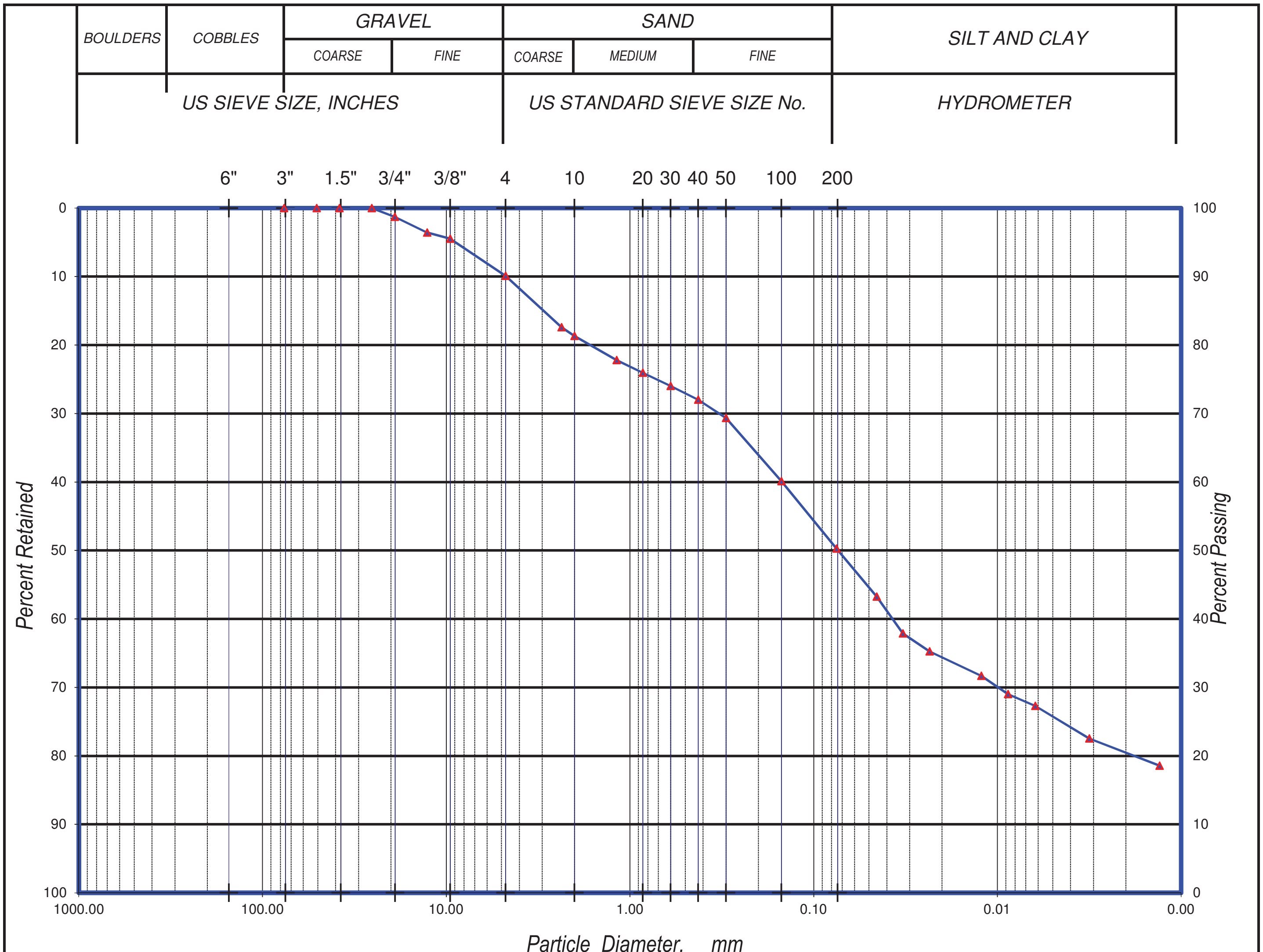
Test Report

ASTM D-6913 / D-7928, (replacing D-422)

Method A: (+/-1%)

Client:	Project No:	Lab Sample No:
ROBSON HOMES LLC	PA22.1048.00	**4933A**

Project Name:	Report Date:
3315 SIERRA ROAD SAN JOSE	November 28, 2022



Symbol	Sample ID	Description	% Gravel	% Sand	% Silt - Clay
▲	DH-2 @ 2-2.5	brown sandy lean clay (CL)	9.9	39.9	50.2

Size Passing, mm D_{60} = 0.15 D_{30} = 0.01 D_{10} = N/A 5 micron (%) = 27

Coefficient of Curvature, C_c: N/A Coefficient of Uniformity, C_u: N/A Fineness Modulus = 1.52

Note: * Percentages are +/- 0.1% based on computer rounding as allowed by ASTM D-6026-01 Section 5.2.3.

This testing is based upon accepted industry practice as well as the test method listed. These results apply only to the samples supplied and tested for the above referenced job.

L : Labexcel \ Projects \ Client \ Client Name \ 4933 \ 4933A-ma	Print Date:	Entered By:	Reviewed By:	LSN:
DCN: MA-rp (rev. 6/27/12)	11/30/22	PP	MK	**4933A**

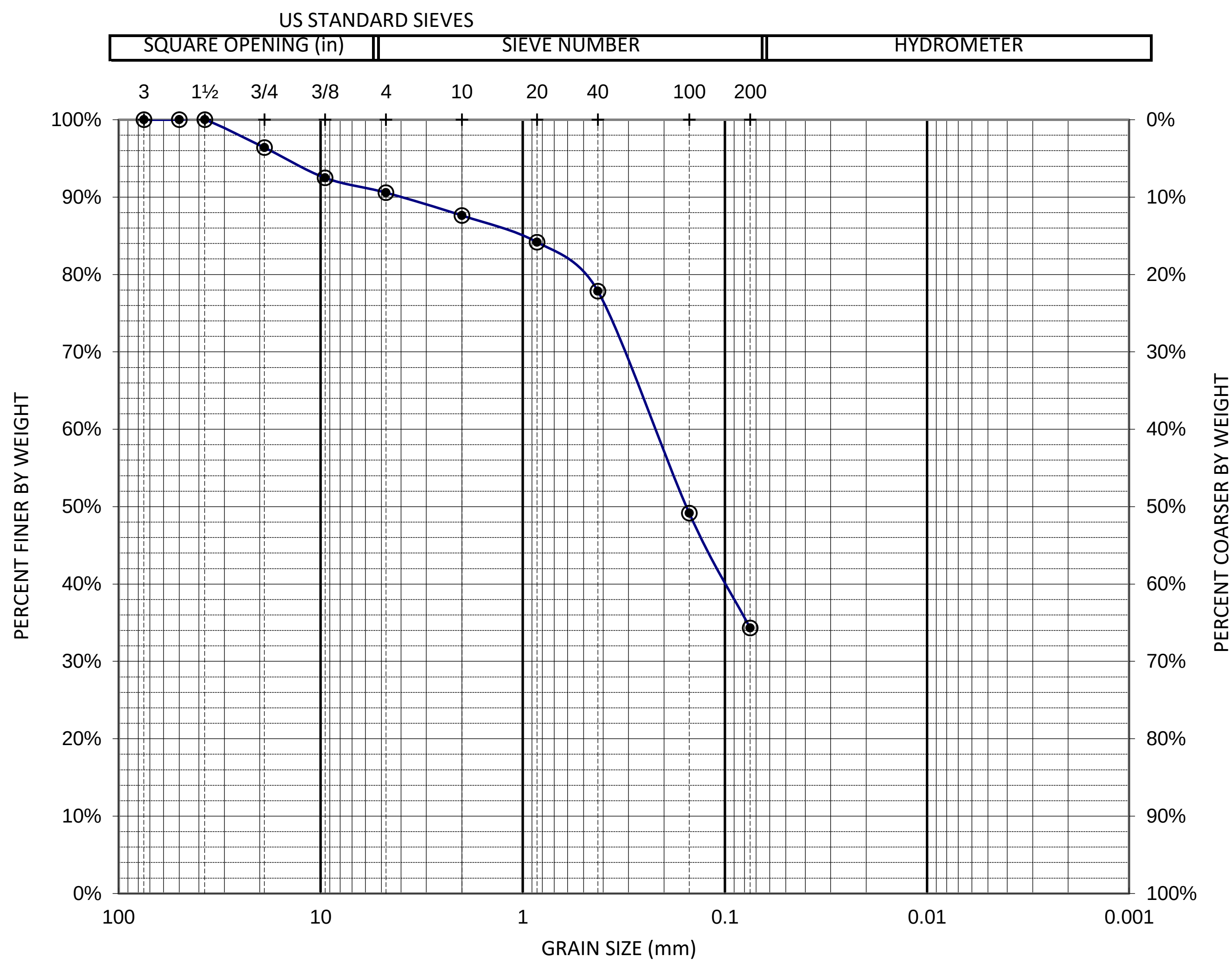
FIGURE B-4

GRAIN SIZE TEST RESULTS

PROJECT NAME	3315 Sierra Road					**PROJECT No.**	PA22.1048
DRILL HOLE No.	DH-3	**DEPTH (ft)**	14.5-15	**SAMPLE**	0	**DATE OF TEST**	12/6/2022

SOURCE/QUARRY: ---

DESCRIPTION OF SOIL: Clayey Sand



	GRAVEL		SAND			SILT & CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES	GRAVEL		SAND			SILT & CLAY
	9.4%		56.3%			34.3%

REMARKS:

GEO-LOGIC ASSOCIATES | **FIGURE** **B-5**

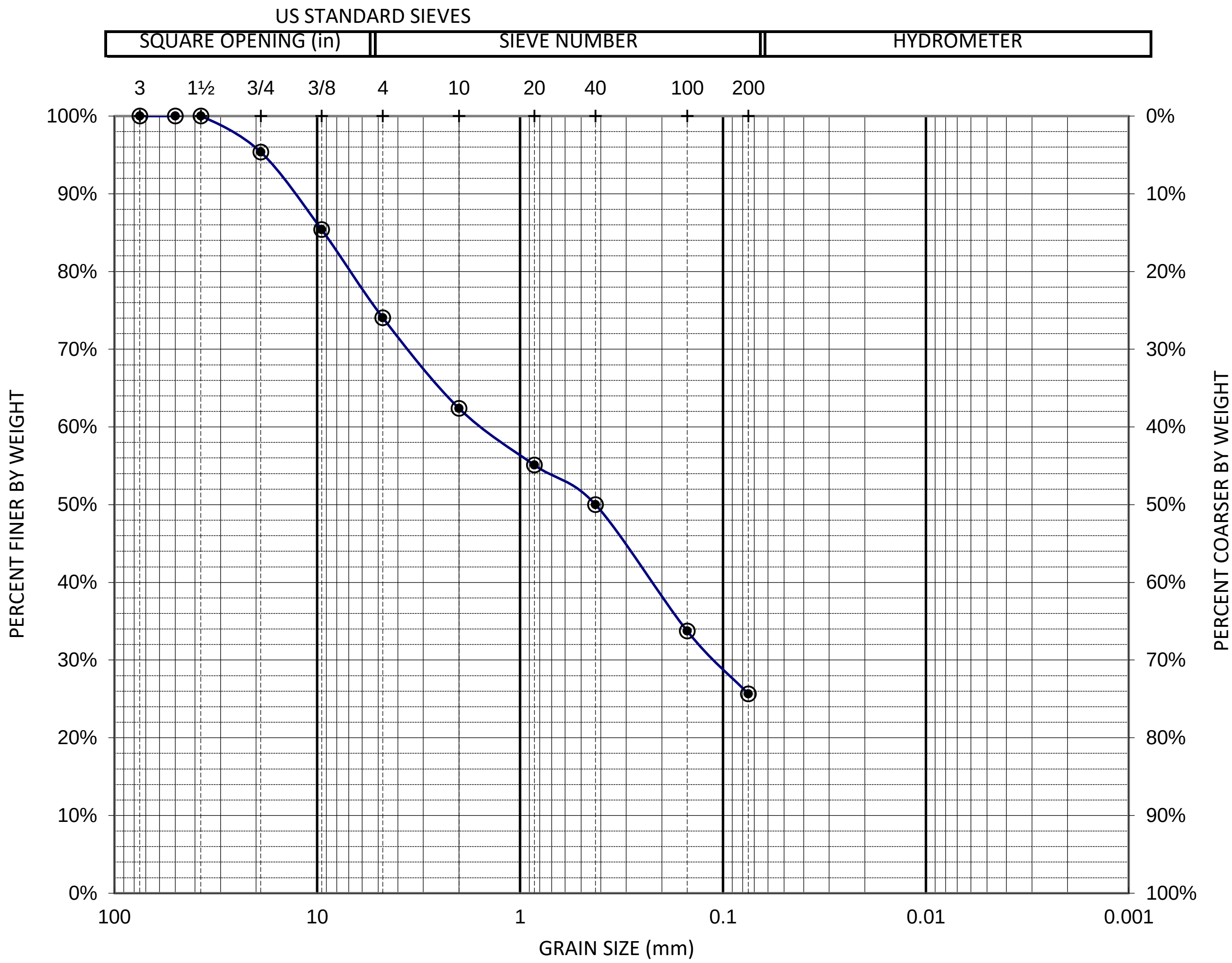
RevOct20070

GRAIN SIZE TEST RESULTS

PROJECT NAME	3315 Sierra Road					**PROJECT No.**	PA22.1048
DRILL HOLE No.	DH-4	**DEPTH (ft)**	9.5-10	**SAMPLE**	0	**DATE OF TEST**	11/17/2022

SOURCE/QUARRY: ---

DESCRIPTION OF SOIL: Clayey Sand with Gravel



	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES	GRAVEL		SAND			SILT & CLAY
	25.9%		48.4%			25.7%

REMARKS:

GEO-LOGIC ASSOCIATES | **FIGURE B-6**

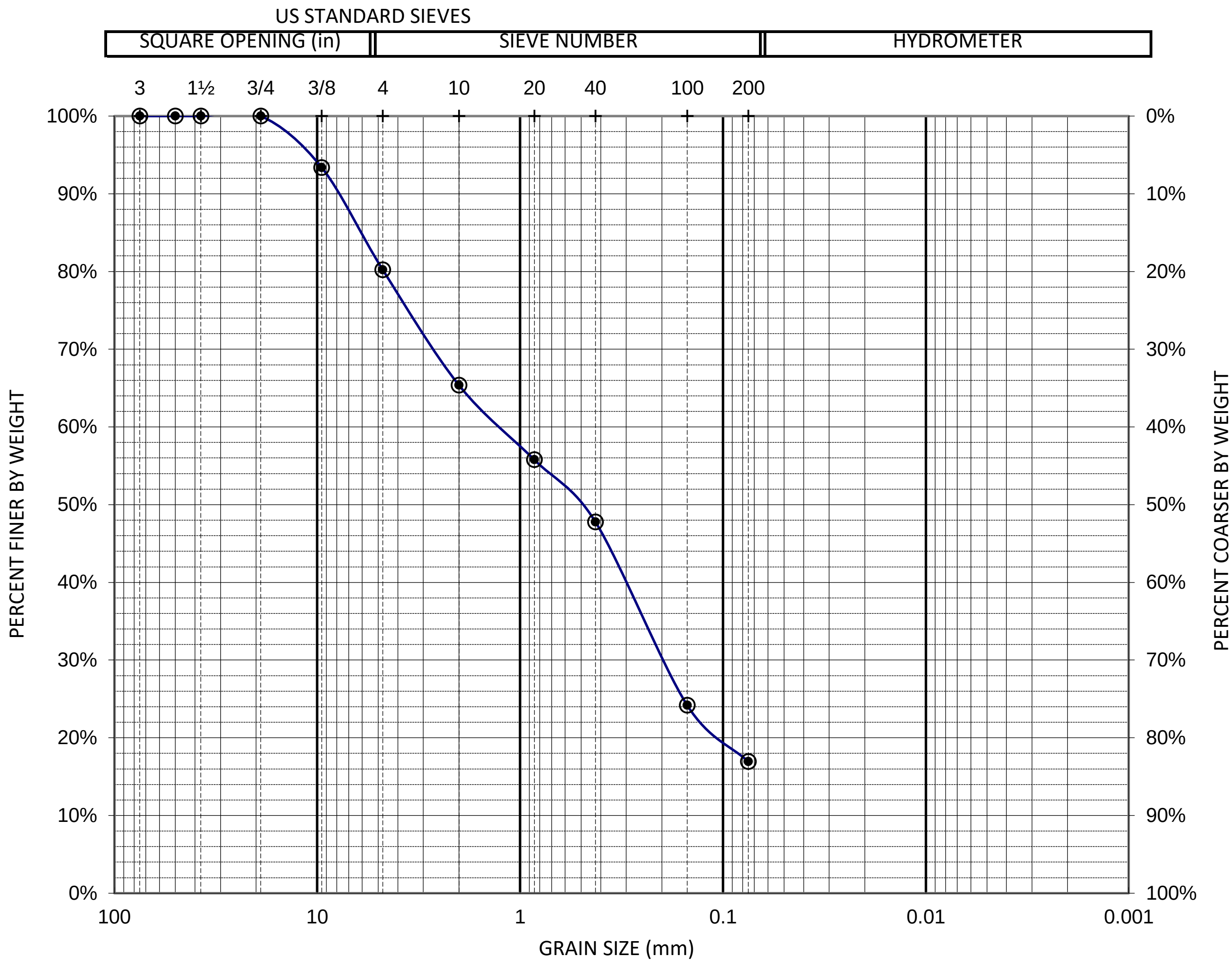
RevOct20070

GRAIN SIZE TEST RESULTS

PROJECT NAME	3315 Sierra Road					**PROJECT No.**	PA22.1048
DRILL HOLE No.	DH-4	**DEPTH (ft)**	20	**SAMPLE**	0	**DATE OF TEST**	11/17/2022

SOURCE/QUARRY: ---

DESCRIPTION OF SOIL: Clayey Sand with Gravel



	GRAVEL		SAND			SILT & CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES	GRAVEL		SAND			SILT & CLAY
	19.8%		63.3%			16.9%

REMARKS:

GEO-LOGIC ASSOCIATES | **FIGURE B-7**

RevOct20070

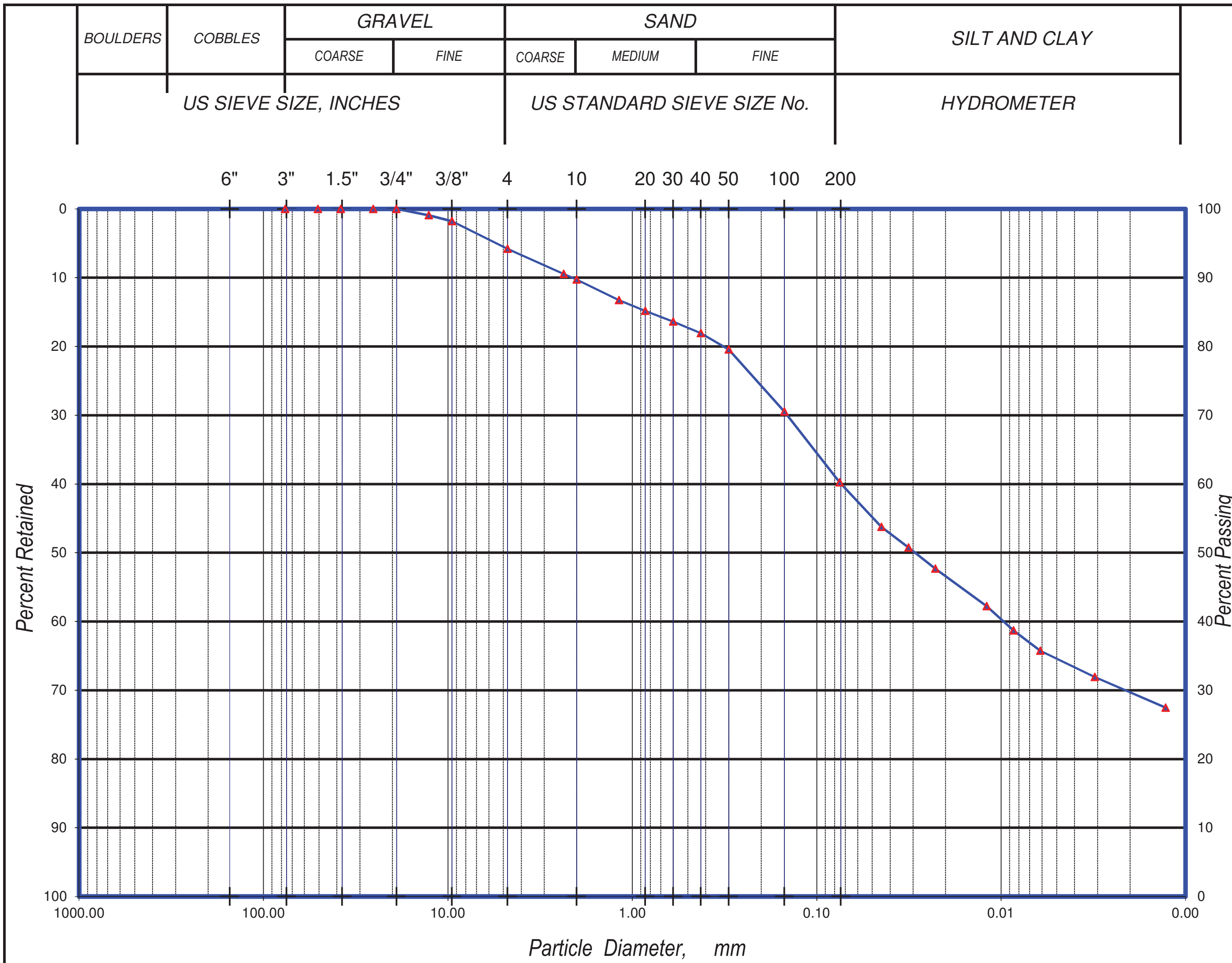
PARTICLE SIZE ANALYSIS

Test Report

ASTM D-6913 / D-7928, (replacing D-422)
Method A: (+/-1%)

Client:	Project No:	Lab Sample No:
ROBSON HOMES LLC	PA22.1048.00	**4933B**

Project Name:	Report Date:
3315 SIERRA ROAD SAN JOSE	November 28, 2022



Symbol	Sample ID	Description	% Gravel	% Sand	% Silt - Clay
▲	DH-6 @ 2-2.5	brown sandy lean clay (CL)	5.8	34.0	60.2

Size Passing, mm D_{60} = 0.07 D_{30} = 0.00 D_{10} = N/A 5 micron (%) = 35

Coefficient of Curvature, C_c: N/A Coefficient of Uniformity, C_u: N/A Fineness Modulus = 0.97

Note: * Percentages are +/- 0.1% based on computer rounding as allowed by ASTM D-6026-01 Section 5.2.3.

This testing is based upon accepted industry practice as well as the test method listed. These results apply only to the samples supplied and tested for the above referenced job.

L : Labexcel \ Projects \ Client \ Client Name \ 4933 \ 4933B-ma

Print Date:	Entered By:	Reviewed By:	LSN:
11/30/22	PP	MK	**4933B**

DCN: MA-rp (rev. 6/27/12)

FIGURE B-8

'R' VALUE CA 301

Project 3315 Sierra Road

Date: 11/19/22

By: LD

Job #: PA22.1048

Sample : On Site Soil

Soil Type: Brown, Silty Clay w. trace F. Gravel

TEST SPECIMEN		A	B	C	D
Compactor Air Pressure	psi	**100**	**65**	**55**	
Initial Moisture Content	%	**12.8**	**12.8**	**12.8**	
Water Added	ml	**60**	**92**	**120**	
Moisture at Compaction	%	18.4	21.4	24.1	
Sample & Mold Weight	gms	**3162**	**3169**	**3140**	
Mold Weight	gms	**2084**	**2103**	**2096**	
Net Sample Weight	gms	1078	1066	1044	
Sample Height	in.	**2.484**	**2.544**	**2.543**	
Dry Density	pcf	111.0	104.5	100.3	
Pressure	lbs	**8250**	**3965**	**2300**	
Exudation Pressure	psi	657	316	183	
Expansion Dial	x 0.0001	**45**	**14**	**0**	
Expansion Pressure	psf	195	61	0	
Ph at 1000lbs	psi	**45**	**66**	**71**	
Ph at 2000lbs	psi	**118**	**141**	**154**	
Displacement	turns	**3.4**	**4.22**	**4.77**	
R' Value		21	7	2	
Corrected 'R' Value		**21**	**7**	**2**	

FINAL 'R' VALUE		
By Exudation Pressure (@ 300 psi):		**6**
By Epansion Pressure	:	**N/A**
TI =	5	

FIGURE B-9

Geo-Logic ASSOCIATES

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

5 December, 2022

Job No. 2211024
Cust. No. 10854

Mr. Beeson Liang
Geo-Logic Associates
6300 San Ignacio Ave., Suite A
San Jose, CA 95119

Subject: Project No.: PA22.1048
Project Name: 3315 Sierra Rd.
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Liang:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on November 17, 2022. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, Sample No.002 is classified as "corrosive". Sample No.001 is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations are none detected and 72 mg/kg and are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentrations are 45 and 100 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils are 8.10 and 7.49, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials are 270 and 290-mV and are indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

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

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

Very truly yours,
CERCO ANALYTICAL, INC.

Sherri Moore

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

JDH/jdl
Enclosure

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

Client: Geo-Logic Associates
Client's Project No.: PA22.1048
Client's Project Name: 3315 Sierra Rd.
Date Sampled: 11-Nov-22
Date Received: 17-Nov-22
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 5-Dec-2022

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2211024-001	DH-1, 1.5'+4.5'	270	8.10	-	3,100	-	N.D.	45
2211024-002	DH-6, 1.5'+4.5'	290	7.49	-	1,000	-	72	100

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	1-Dec-2022	30-Nov-2022	-	1-Dec-2022	-	30-Nov-2022	30-Nov-2022

* Results Reported on "As Received" Basis

N.D. - None Detected

Sherri Moore

Sherri Moore
Chemist

Quality Control Summary - All laboratory quality control parameters were found to be within established limits