CORNERSTONE
EARTH GROUP

TYPE OF SERVICES Design Level Geotechnical Investigation

PROJECT NAME Modera San Rafael

LOCATION 523-535 4th Street and 912 Irwin Street
San Rafael, California

CLIENT Mill Creek Residential Trust

PROJECT NUMBER 1395-4-4

DATE August 23, 2024 (updated February 7, 2025)

CORNERSTONE
EARTH GROUP

Type of Services Design Level Geotechnical Investigation

Project Name Modera San Rafael

Location 523-535 4th Street & 912 Irwin Street
San Rafael, California

Client Mill Creek Residential Trust

Client Address 3697 Mt. Diablo Boulevard, Suite 350
Lafayette, CA

Project Number 1395-4-4

Date August 23, 2024 (updated February 7, 2024)

Prepared by **Bryan Cervantes Guzman, P.E.**
Project Engineer

John R. Dye, P.E., G.E.
Senior Principal Engineer
Geotechnical Project Manager

REGISTERED PROFESSIONAL ENGINEER
JOHN R. DYE
GE2582
GEOTECHNICAL
STATE OF CALIFORNIA

TABLE OF CONTENTS

SECTION 1: INTRODUCTION 1
1.1 Project Description ---------- 1
1.2 Scope of Services ---------- 2
1.3 Exploration Program ---------- 2
1.4 Previous field Exploration ---------- 2
1.5 Laboratory Testing Program ---------- 3
1.6 Environmental Services ---------- 3

SECTION 2: REGIONAL SETTING 3
2.2 Regional Seismicity ---------- 3
Table 1: Approximate Fault Distances 3

SECTION 3: SITE CONDITIONS 4
3.1 Site Background ---------- 4
3.2 Surface Description ---------- 4
3.2 Subsurface Conditions ---------- 5
3.3 Groundwater ---------- 5

SECTION 4: GEOLOGIC HAZARDS 6
4.1 Fault Rupture ---------- 6
4.2 Estimated Ground Shaking ---------- 6
4.3 Liquefaction Potential ---------- 6
  4.3.1 Background 6
  4.3.2 Analysis 7
  4.3.3 Summary 7
  4.3.4 Ground Deformation Potential 8
4.4 Lateral Spreading ---------- 8
4.5 Seismic Settlement/Unsaturated Sand Shaking ---------- 8
4.6 Flooding ---------- 8
4.7 Tsunami/seiche ---------- 9

SECTION 5: CONCLUSIONS 9
5.1 Summary ---------- 9
  5.1.1 Potential for Significant Static Settlement of Compressible Clays 10
  5.1.2 Potential for Liquefaction-Induced Settlements and Ground Deformation 10
  5.1.3 Presence of Undocumented Fills and Redevelopment Considerations 10
  5.1.4 Shallow Groundwater, Hydro-Static Uplift and Water Proofing 11
  5.1.5 Presence of Expansive Soils 11
  5.1.6 Temporary Shoring and Underpinning 12
5.2 Plans and Specifications Review ---------- 12
5.3 Construction Observation and Testing ---------- 12

SECTION 6: EARTHWORK 12
6.1 Site Demolition ---------- 12

6.1.1 Demolition of Existing Slabs, Foundations and Pavements 13
6.1.2 Abandonment of Existing Utilities 13
6.2 Site Clearing and Preparation---14
6.2.1 Site Stripping 14
6.2.2 Tree and Shrub Removal.. 14
6.3 Mitigation of Undocumented Fills ---14
6.4 Temporary Cut and Fill Slopes ---14
6.5 Below-Grade Excavations ---15
6.5.1 Underpinning 15
6.5.2 Construction Dewatering 15
6.6 Subgrade Preparation--16
6.7 Wet Soil Stabilization Guidelines--16
6.7.1 Scarification and Drying.. 17
6.7.2 Removal and Replacement 17
6.7.3 Chemical Treatment 17
6.7.4 Mat Foundation Excavation Stabilization... 17
6.8 Material for Fill ---17
6.8.1 Re-Use of On-site Soils 17
6.8.2 Re-Use of On-Site Site Improvements 18
6.8.3 Potential Import Sources 18
6.9 Compaction Requirements --18
Table 2: Compaction Requirements.. 19
6.9.1 Construction Moisture Conditioning.. 19
6.10 Trench Backfill --20
6.10.1 Flexible Utility Connections.. 20
6.11 Site Drainage--21
SECTION 7: 2022 CBC SEISMIC DESIGN CRITERIA 21
7.1 Seismic Design Criteria--21
Table 3: CBC Site Categorization and Site Coefficients 22
SECTION 8: FOUNDATIONS 22
8.1 Summary of Recommendations--22
8.2 Reinforced Concrete Mat Foundation--22
8.2.1 Mat Foundation Settlement... 23
8.2.2 Lateral Loading 23
8.2.3 Mat Modulus of Soil Subgrade Reaction 23
8.2.4 Mat Foundation Construction Considerations.. 24
8.2.5 Hydrostatic Uplift and Waterproofing 24
SECTION 9: CONCRETE SLABS AND VEHICULAR PAVEMENTS 24
9.1 Pedestrian Concrete Flatwork ---24
9.2 Vehicular Pavements---24
SECTION 10: RETAINING WALLS.. 25
10.1 Static Lateral Earth Pressures --25
Table 4: Recommended Lateral Earth Pressures 25
10.2 Seismic Lateral Earth Pressures --25

10.3 Wall Drainage --25
10.4 Backfill --25
10.5 Foundations --26

SECTION 11: LIMITATIONS 26

SECTION 12: REFERENCES 27

FIGURE 1: VICINITY MAP
FIGURE 2: SITE PLAN
FIGURE 3: REGIONAL FAULT MAP
FIGURE 4A TO 4C: LIQUEFACTION ANALYSIS SUMMARY – CPT-1 TO CPT-3
FIGURE 5: VARIABLE MODULUS OF SUBGRADE REACTION FOR MAT FOUNDATION

APPENDIX A: FIELD INVESTIGATION
APPENDIX B: LABORATORY TEST PROGRAM
APPENDIX C: ENVIRONMENTAL EXPLORATION LOGS

Type of Services | **Design Level Geotechnical Investigation**
Project Name | **Modera San Rafael**
Location | **523-535 4th Street & 912 Irwin Street**
San Rafael, California

SECTION 1: INTRODUCTION

This design level geotechnical report was prepared for the sole use of Mill Creek Residential Trust for the Modera San Rafael in San Rafael, California. The location of the site is shown on the Vicinity Map, Figure 1. As you know, we performed a preliminary geotechnical investigation for the site and presented our findings in our report dated July 22, 2024. For our use, we were provided with the following documents:

- A preliminary conceptual plan titled "Fourth & Irwin, San Rafael, Draft Plan Revisions," prepared by Trachtenberg Architects dated April 24, 2024.
- A topographic survey titled, "Boundary and Topographic Survey of 523 & 543 Fourth Street" prepared by Muir Consulting, Inc dated August 3, 2021.
- Structural loading diagrams provided by VCA Structural, December 13, 2024 and January 16, 2025.

1.1 PROJECT DESCRIPTION

The project site encompasses three parcels in downtown San Rafael, California (APN's 014-123-21, 014-123-28 and 014-123-27). The approximately 0.92-acre site (three conjoining parcels) is currently occupied by three 2-story commercial office buildings. Based on our review of the conceptual plans provided by Trachtenberg Architects, the planned development will consist of an 8-story residential building with three levels of podium garage parking encompassing a majority of the site. The five residential levels will consist of 213 units of 3-bedroom, 2-bedroom, 1-bedroom, and studio apartments. A common area consisting of a podium garden and patio will be located on the fourth floor. The building will be supported at-grade. Three levels of podium parking will be of concrete construction and the five levels of residential units will likely be of wood-frame construction. Appurtenant utilities, landscaping, and other improvements necessary for site development are also planned.

Structural loads provided by the structural engineer indicate dead plus live (unfactored) foundation pressures will range from approximately 1,100 to 1,400 pounds per square foot for most of the interior portions of the foundation to an average of 2,000 psf around the perimeter of the foundation. Grading plans are not available at this time; however, we understand grading will consist of minimal fills and cuts on the order 2 to 5 feet to accommodate the mat foundation and localized elevator pit areas.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated May 14, 2024, and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of three borings drilled on July 17, 2024, with truck-mounted, hollow-stem auger drilling equipment. The borings were drilled to depths ranging from 30 to 60 feet. The borings were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

The approximate locations of our exploratory borings are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

1.4 PREVIOUS FIELD EXPLORATION

We previously performed field explorations as part of preliminary investigation in December 2023, which consisted of three Cone Penetration Test (CPT) soundings using truck-mounted CPT exploration equipment. The CPTs, CPT-1 to CPT-3, were advanced to depths ranging from 53 to 67 feet below existing site grades. Each CPT encountered CPT refusal at their respective depths.

As part of the preliminary geotechnical investigation, we collected soil samples from our concurrent Phase 2 environmental investigation to observe general soil conditions within the upper approximately 1¼ to 5 feet of the soil profile. Six soil vapor boreholes were hydraulically pushed using a track mounted push probe drilling rig to depths of approximately 1¼ to 5 feet below site grades. Additional field exploration was performed in February 2024 that consisted of eight push probe soil borings to a depth of 10 feet each. Copies of the environmental exploration logs are presented in Appendix B.

The CPTs and push probe boreholes were backfilled in accordance with local requirements. The approximate locations of our CPTs and concurrent soil vapor boreholes are shown on the Site Plan, Figure 2. Details regarding our field program are also included in Appendix A.

1.5 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for foundation design and seismic ground deformation estimates. Testing included moisture contents, dry densities, washed sieve analyses, Plasticity Index tests, consolidation, and triaxial compression tests. Details regarding our laboratory program are included in Appendix C.

1.6 ENVIRONMENTAL SERVICES

Cornerstone Earth Group also provided environmental services for this project, including Phase 1 and 2 site assessments; environmental findings and conclusions are provided under separate covers.

SECTION 2: REGIONAL SETTING

2.2 REGIONAL SEISMICITY

While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated (in 2015) earlier estimates from their 2014 Uniform California Earthquake Rupture Forecast (Version 3; UCERF3) publication. The estimated probability of one or more magnitude 6.7 earthquakes (the size of the destructive 1994 Northridge earthquake) expected to occur somewhere in the San Francisco Bay Area has been revised (increased) to 72 percent for the period 2014 to 2043 (Aagaard et al., 2016). The faults in the region with the highest estimated probability of generating damaging earthquakes between 2014 and 2043 are the Hayward (33%), Calaveras (26%), and San Andreas Faults (22%). In this 30-year period, the probability of an earthquake of magnitude 6.7 or larger occurring is 22 percent along the San Andreas Fault and 33 percent for the Hayward Fault.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The table below presents the State-considered active faults within 25 kilometers of the site.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
Hayward (Total Length)	8.7	14.0
San Andreas	9.6	15.4
San Gregorio	10.3	16.6
Rodger’s Creek	14.6	23.5

A regional fault map is presented as Figure 3, illustrating the relative distances of the site to significant fault zones.

SECTION 3: SITE CONDITIONS

3.1 SITE BACKGROUND

We reviewed historical aerial imagery provided online by Historical Aerials (http://www.historicaerials.com), Environmental Data Resources, and Google Earth Pro (2023). A summary of pertinent surface changes at and in the near vicinity of the site is as follows:

- Prior to 1946: Aerial images were not available prior to 1946; however, prior to 1940 the parcel was shown to be vacant on Sanborn maps from 1927 and 1907. A USGS topographic map from 1895 indicates an unnamed creek channel may have meandered across a portion of the site.
- 1946: The aerial image appears to show the site divided into four smaller parcels with smaller buildings and structures in place. The area in the vicinity appears developed and street layouts match existing layouts. Highway 101 appears to have been built but is narrower than the current widths.
- 1950s-1960s: The parcel at the corner of 4th and Irwin is developed as a gas station. The 912 Irwin Street parcel is occupied by at least one building.
- 1982: The previous buildings and structures were removed, and the site was redeveloped into the two current commercial office buildings and paved parking lot. The building at 912 Irwin Street was also constructed. Highway 101 appears to have been widened to its current width.
- 2020: No major surfaces changes appear to have been made to the site since 1982. The site appears to be in the same condition as during this geotechnical investigation.

From the concurrent Phase 1 and Phase 2 environmental investigation, we understand a fuel station was once present on the corner of 4th Street and Irwin Street prior to redeveloping the site to existing conditions. Typical infrastructure associated with fuel stations include underground storage tanks, canopy footings or piers, piping, and typical service station building, utilities, flatwork, and pavements.

3.2 SURFACE DESCRIPTION

The project site is situated within the highly developed downtown district of the City of San Rafael. The site is situated east of Hwy 101 and is bounded by 4th Street to the north, Irwin Street to the west, and commercial development to the south and east. The site is currently occupied by three 2-story office buildings with a conjoined asphalt parking lot. The parking lot extends beneath portions of the two northern buildings resulting in a “soft story” condition for portions of the two existing buildings. The southern office building has an adjacent concrete parking lot. Existing flatwork envelopes portions of the buildings, landscaping strips and islands were observed, and other site features such as utility equipment and site walls were observed. The site is relatively level but graded to drain to storm drain inlets.

Based on the topographic survey provided by Muir Consulting, Inc., the site grades vary between Elevation 9 and 10 feet (NAVD88 Datum). From our recent exploration, the asphalt pavement ranged from 3 to 4 inches thick and aggregate base sections were observed to be approximately 5 to 6 inches thick. Site pavements appear to be in fair condition with some areas of observed distress consisting of alligator cracking and block cracking. Pavement rehabilitation consisting of slurry seals and asphalt overlays appears to have been previously performed on the pavement.

3.2 SUBSURFACE CONDITIONS

In general, the site subsurface soil profile is anticipated to consist of undocumented fills underlain by Holocene-aged alluvial soils underlain by Franciscan bedrock. Below the surface pavements and based on our recent exploratory borings as well as previously performed environmental exploration, the upper 1¼ to 5 feet of the soil profile primarily consisted of undocumented fills. The undocumented fills are highly variable in consistency and vary between medium stiff to stiff clayey soils such as fat clay, sandy fat clay, fat clay with sand, lean clay with sand and lean clay, as well as loose to medium dense coarse-grained fills consisting of crushed rock/concrete, clayey sand, and silty sand. From the soil samples collected and observed, we estimate that the onsite clayey fill soils with be moderately to highly expansive. Additional undocumented fills may be present from previous development and redevelopment of the site, especially at the former gas station parcel, where underground storage tanks were removed and backfilled. Additional explorations should be performed after the buildings are demolished to confirm the depth of undocumented and consistency of fill present onsite.

Below the fills, our exploratory borings encountered Holocene-aged alluvial soils to depths of 30 to 60 feet, the maximum depth explored during our exploratory borings. The alluvial soils directly beneath the fills consisted of medium stiff to stiff lean to fat clay with sand to depths of 7½ to 9 feet. From our review of the geology in the vicinity, we understand that portions of Downtown San Rafael were once tidal marshes along San Rafael Creek. From the lab test data and our observations and experience in the area, the upper 2 to 3 feet of the alluvial clayey material consists of moderately compressible clayey material known as Bay Mud. Below the Bay Mud layer, older, stiffer alluvial soils consisting of stiff to hard clayey soils and medium dense to dense granular soils were encountered to depths of 60 feet.

Below the bottom of our exploratory borings, based on CPT shear strength estimates, the stiffness of the soil profile appears to reach hard to very hard consistency near the terminal depths of each CPT. Because each CPT encountered tip pressure refusal, we anticipate that Franciscan bedrock was encountered at each CPT terminal depth.

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from approximately 9 to 49 percent moisture. In our opinion, we estimated this corresponds to about 2 to 29 percent above the estimated laboratory optimum moisture content.

3.3 GROUNDWATER

Groundwater was encountered in our Borings EB-1 to EB-3, at depths of 6 to 10 feet depth. Groundwater was interpreted from pore pressure measurements taken at CPT-1 and CPT-2, with inferred groundwater at elevated depths near or above the ground surface. However, we note the pore pressure dissipation tests were performed in water bearing zones at depths ranging 20 to 63 feet that are likely under confined conditions and not representative of the static groundwater level of the upper water bearing soils. CPT-3 pore pressure measurement indicated a groundwater depth of approximately 4½ feet and was conducted at a depth of 15 feet below surface grades. Groundwater depths measured by hand by dropping a measuring tape into the CPT holes indicated groundwater depths ranging from 7 to 9 feet depth but are not considered stabilized water levels. All measurements were taken at the time of drilling and may not represent the stabilized levels that can be higher than the initial levels encountered.

Based on our review of monitoring well data from the California GeoTracker website for the 520 4th Street cleanup program site, multiple monitoring wells were installed along 4th Street or in adjacent parcels. Groundwater was recorded in these monitoring wells between 2015 to 2021. From our review, groundwater depths at these wells varied between approximately 1 to 4½ feet.

In general, fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.
Based on the above information, we recommend a design groundwater depth of 3 feet below current grades.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. The site is not located within a State-designated Alquist-Priolo Earthquake Fault Zone. As shown in Figure 3, no known surface expression of fault traces is thought to cross the site; therefore, fault rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A site modified peak ground acceleration (PGA_m) was determined in accordance with Section 21.5 of ASCE 7-16. Therefore, we recommend a site-specific MCE_G peak ground acceleration, PGA_m, of 0.56g for this project.

4.3 LIQUEFACTION POTENTIAL

Currently, the California Geologic Survey has not issued a quadrangle map designating seismicity hazards for San Rafael. From our review of the Association of Bay Area Government's liquefaction susceptibility map, the site is designated as an area with high to very high susceptibility to liquefaction. Our field programs addressed this issue by testing potentially liquefiable layers to depths of at least 50 feet and evaluating CPT data.

4.3.1 Background

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data are available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 4 percent of the liquefied layer thickness can occur. Soils most susceptible to liquefaction are loose, non-cohesive soils that are saturated and are bedded with poor drainage, such as sand and silt layers bedded with a cohesive cap.

4.3.2 Analysis

As discussed in the "Subsurface" section above, several sand layers were encountered below the design ground water depth of 3 feet. Following the liquefaction analysis framework in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008), incorporating updates in *CPT and SPT Based Liquefaction Triggering Procedures* (Boulanger and Idriss, 2014), and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008) for quantitative analysis, these layers were analyzed for liquefaction triggering and potential post-liquefaction settlement. These methods compare the ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil's estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.3 are considered to be potentially liquefiable and capable of post-liquefaction re-consolidation (i.e. settlement).

The CSR for each layer quantifies the stresses anticipated to be generated due to a design-level seismic event, is based on the peak horizontal acceleration generated at the ground surface discussed in the "Estimated Ground Shaking" section above, and is corrected for overburden and stress reduction factors as discussed in the procedure developed by Seed and Idriss (1971) and updated in the 2008 Idriss and Boulanger monograph.

The soil's CRR is estimated from the in-situ measurements from CPTs. The tip pressures are corrected for effective overburden stresses, taking into consideration both the ground water level at the time of exploration and the design ground water level, and stress reduction versus depth factors. The CPT method utilizes the soil behavior type index (I_C) to estimate the plasticity of the layers.

The results of our preliminary CPT analyses (CPT-1 to CPT-3) are presented in Appendix B of this report.

4.3.3 Summary

Our analyses indicate that several layers could potentially experience liquefaction triggering that could result in post-liquefaction total settlement at the ground surface ranging from ½ to 1 inch based on the Yoshimine (2006) method. As discussed in SP 117A, differential movement for level ground sites over deep soil sites will be up to about two-thirds of the total settlement between independent foundation elements. In our opinion, differential seismic settlements are anticipated to be on the order of ¾ inch or less across the future mat foundation.

4.3.4 Ground Deformation Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground rupture or sand boils. For ground deformation to occur, the pore water pressure within the liquefiable soil layer will need to be great enough to break through the overlying non-liquefiable layer, which could cause significant ground deformation and settlement. The work of Youd and Garris (1995) indicates that the current approximately 4- to 17-feet thick layer of non-liquefiable cap is sufficient to prevent ground rupture; therefore, the above total settlement estimates are reasonable.

4.4 LATERAL SPREADING

Lateral spreading is horizontal/lateral ground movement of relatively flat-lying soil deposits towards a free face such as an excavation, channel, or open body of water. Typically, lateral spreading is associated with liquefaction of one or more subsurface layers near the bottom of the exposed slope. As failure tends to propagate as block failures, it is difficult to analyze and estimate where the first tension crack will form.

The San Rafael Creek is located approximately 640 feet from the southern edge of the site to the top of the bank. The bottom of the creek is not currently known. For our preliminary analysis, we have assumed a 7-foot free-face of San Rafael Creek that is susceptible to lateral spreading. We calculated the Lateral Displacement Index (LDI) for potentially liquefiable layers based on methods presented in the 2008 monograph, Soil Liquefaction During Earthquakes (Idriss and Boulanger, 2008). LDI is a summation of the maximum shear strains versus depth, which is a measurement of the potential maximum displacement at that exploration location. Summations of the LDI values to a depth equal to twice the open face height were included. Estimated displacements for areas near CPT-1 through CPT-3 based on the LDI calculations are on the order of 1 inch or less. In our opinion, the potential for lateral spreading to impact the project is relatively low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. We evaluated the potential for seismic compaction of sands encountered above the preliminary design groundwater table based on the work by Robertson and Shao (2010). Based on our analyses, the potential for significant seismic settlement affecting the proposed improvements is low.

4.6 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone AE, an area with a base flood elevation of 10 feet. We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

4.7 TSUNAMI/SEICHE

The terms tsunami or seiche are described as ocean waves or similar waves usually created by undersea fault movement or by a coastal or submerged landslide. Tsunamis may be generated at great distance from shore (far field events) or nearby (near field events). Waves are formed, as the displaced water moves to regain equilibrium, and radiates across the open water, similar to ripples from a rock being thrown into a pond. When the waveform reaches the coastline, it quickly raises the water level, with water velocities as high as 15 to 20 knots. The water mass, as well as vessels, vehicles, or other objects in its path create tremendous forces as they impact coastal structures.

Tsunamis have affected the coastline along the Pacific Northwest during historic times. The Fort Point tide gauge in San Francisco recorded approximately 21 tsunamis between 1854 and 1964. The 1964 Alaska earthquake generated a recorded wave height of 7.4 feet and drowned eleven people in Crescent City, California. For the case of a far-field event, the Bay area would have hours of warning; for a near field event, there may be only a few minutes of warning, if any.

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the mapping of tsunami inundation potential for the San Francisco Bay Area by CGS (conservation.ca.gov/cgs/tsunami/maps), areas most likely to be inundated are marshlands, tidal flats, and former bay margin lands that are now artificially filled, but are still at or below sea level, and are generally within 1½ miles of the shoreline. The site is approximately 1½ miles inland from the San Rafael Bay shoreline and is approximately 9 to 10 feet above mean sea level. According to the available tsunami hazard map provided by the CGS, the site is located within a tsunami hazard zone (CGS, Tsunami Hazard Area Map, County of Marin, 2022). The potential for inundation due to tsunami at the site is considered high.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

From a geotechnical viewpoint, the project is feasible provided the concerns listed below are addressed in the project design. Descriptions of each geotechnical concern with brief outlines of our recommendations follow the listed concerns.

- Potential for significant static settlements of compressible clays
- Potential for liquefaction-induced settlements
- Presence of undocumented fill and redevelopment considerations
- Shallow groundwater, hydro-static uplift, and Waterproofing
- Presence of expansive soils
- Temporary shoring and underpinning

5.1.1 Potential for Significant Static Settlement of Compressible Clays

As discussed, the site is underlain by up to 20 feet of moderately compressible clay that will settle under the weight of new fill and from heavy building loads. At this time, we understand that new fill would not be required for the new building; therefore, total settlement is anticipated to occur solely due to the weight of the new building. Because of the compressible nature of the clays near the surface, lower allowable bearing pressures are required in these soils that would make typical spread and strip footings unfeasible. From the provided structural loading diagrams, we evaluated the use of a rigid mat foundation to distribute building loads. We evaluated consolidation settlement due to static building loads assuming preliminary average aerial mat foundation pressures of approximately 1,300 pounds per square foot (psf). The settlement analysis was updated based on revised loading ranging from an average of 1,00 to 1,400 psf near the central portion of the mat to approximately 2,000 psf near the perimeter of the mat.

Based on the soil profile encountered in our explorations and the provided foundation contact pressures, we anticipate that approximately 2 to 4½ inches of settlement could occur across a mat foundation due to consolidation of the underlying clay layers. A significant portion of the consolidation settlement occurs in the underlying compressible clay layers between depths of approximately 5 to 15 feet below existing surface grades. Based on our discussions with the design team, we understand that a rigid mat foundation can be designed to tolerate the anticipated total and differential settlement without the need for ground improvement. Settlement at building entrances will be mitigated using suitable hinged slabs or walkways that can be leveled as needed. In addition, flexible utility connections will be required that are capable of accommodating up to 4 inches of differential settlement between the adjacent sidewalk and the edge of the mat. Recommendations are presented in the “Foundations” section of this report.

5.1.2 Potential for Liquefaction-Induced Settlements and Ground Deformation

As discussed, our liquefaction analysis indicates that there is a potential for liquefaction of localized sand layers during a significant seismic event on the order of ½ to 1 inch, resulting in differential settlement up to ¾ inch. Foundations should also be designed to tolerate the anticipated total and differential settlements in addition to the static settlements referenced above.

5.1.3 Presence of Undocumented Fills and Redevelopment Considerations

As discussed, the site is blanketed by up to 5 feet of undocumented fill. The fill is immediately underlain by soft to medium stiff, moderately compressible clay. Locally, deeper fills on the order of 8 to 10 feet deep are present where the prior gas station fuel tanks were removed. Localized zones of deeper fill may be present where grading and development occurred for the three existing office buildings.

For the planned mat foundation, some of the existing undocumented fill will be excavated and removed from the site during foundation preparation. Fills encountered outside the footprint of a mat foundation will need to be scarified, moisture conditioned and re-compacted beneath any new at-grade improvements or prior to placing any new fill. We recommend the mat foundation cut area be stabilized and re-compacted prior to foundation construction. Recommendations for mitigating undocumented fill are presented in the “Earthwork” section.

As discussed, the site is currently occupied by three two-story commercial office buildings, asphalt parking lot, and appurtenant flatwork, site fixtures, and landscaping. Older buildings typically were constructed with widely varying foundation systems. On sites near creeks and the Bay, it is common to find deeper foundation systems that may consist of deepened footings, drilled piers, belled piers, or deep foundations. If as-built drawings are available for the existing buildings, please forward them to our office for review.

We assume that all the existing improvements will be demolished for the construction of the new building. Potential issues that are often associated with redeveloping sites include demolition of existing improvements, disturbance to surficial soils due to foundation removal, abandonment of existing utilities, and discovery of localized deeper undocumented fill. The former fuel station is expected to have had deeper underground storage tanks that were removed when the current development was built and filled in with undocumented fill.

5.1.4 Shallow Groundwater, Hydro-Static Uplift and Water Proofing

As previously discussed, groundwater has been measured on-site at depths of approximately 4 ½ to 7 feet and at nearby monitoring wells along 4th Street at depths ranging from approximately 1 to 4½ feet below the existing ground surface. As discussed above, we recommend a design groundwater depth of 3 feet below existing ground surface. Our experience with similar sites in the vicinity indicates that shallow ground water could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable foundation or excavation subgrade, difficulty achieving compaction, and difficult underground utility installation. Due to the high moisture content of the fill and native soils, we recommend chemically treating the upper 18 inches of exposed fill or native soils with lime to reduce moisture content, improve soil strength, and to create a stiff building pad to support construction equipment. Recommendations addressing this concern are included in the “Earthwork” section.

Temporary dewatering and shoring of utility trenches may be required in some isolated areas of the site. Where portions of the mat foundation and related deepened structures extend below the design groundwater level, including bottoms of mat foundations or elevator pits, they should be water-proofed and designed to resist potential hydrostatic uplift pressures.

5.1.5 Presence of Expansive Soils

The site surficial soils are moderately to highly expansive. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. To reduce the potential for damage to the planned improvements, slabs-on-grade outside of the mat footprint should have sufficient reinforcement and be supported on a layer of non-expansive fill; shallow footings should extend below the zone of seasonal moisture fluctuation. In addition, it is important to limit moisture changes in the surficial soils by using positive drainage away from buildings as well as limiting landscaping watering. Recommendations addressing this concern are presented in the following sections of this report.

5.1.6 Temporary Shoring and Underpinning

If site grading or foundation excavations deeper than 2 to 3 feet are performed adjacent to existing property boundaries, the excavations could impact adjacent properties and potentially undermine shallow foundations or slabs. Temporary shoring or underpinning may be necessary to support adjacent structures or slabs to prevent detrimental movement. We estimate ¼ to ½ inch of settlement could occur within about 5 to 10 feet beyond the perimeter of the new mat foundation. The contractor should plan to provide underpinning or shoring support, as needed. We should review temporary shoring or underpinning plans to provide input and additional recommendations.

5.2 PLANS AND SPECIFICATIONS REVIEW

We recommend that we be retained to review the geotechnical aspects of the project structural, civil, and landscape plans and specifications, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction.

5.3 CONSTRUCTION OBSERVATION AND TESTING

As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that a Cornerstone representative be present to provide geotechnical observation and testing during earthwork and foundation construction. This will allow us to form an opinion and prepare a letter at the end of construction regarding contractor compliance with project plans and specifications, and with the recommendations in our report. We will also be allowed to evaluate any conditions differing from those encountered during our investigation and provide supplemental recommendations as necessary. For these reasons, the recommendations in this report are contingent of Cornerstone providing observation and testing during construction. Contractors should provide at least 48-hour notice when scheduling our field personnel.

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION

All existing improvements not to be reused for the current development, including all foundations, flatwork, pavements, utilities, and other improvements should be demolished and removed from the site. Recommendations in this section apply to the removal of these improvements, which are currently present on the site, prior to the start of mass grading or the construction of new improvements for the project.

Cornerstone should be notified prior to the start of demolition and should be present on at least a part-time basis during all backfill and mass grading as a result of demolition. Occasionally, other types of buried structures (wells, cisterns, debris pits, etc.) can be found on sites with prior development. If encountered, Cornerstone should be contacted to address these types of structures on a case-by-case basis.

6.1.1 Demolition of Existing Slabs, Foundations and Pavements

All slabs, foundations, and pavements should be completely removed from within planned building areas. A discussion of recycling existing improvements is provided later in this report.

Special care should be taken during the demolition and removal of existing floor slabs, foundations, utilities and pavements to minimize disturbance of the subgrade. Excessive disturbance of the subgrade, which includes either native or previously placed engineered fill, resulting from demolition activities can have serious detrimental effects on planned foundation and paving elements.

Existing foundations are typically mat-slabs, shallow footings, or piers/piles. If slab or shallow footings are encountered, they should be completely removed. If drilled piers are encountered, they should be cut off at an elevation at least 60-inches below proposed footings or the final subgrade elevation, whichever is deeper. The remainder of the drilled pier could remain in place. Foundation elements to remain in place should be surveyed and superimposed on the proposed development plans to determine the potential for conflicts or detrimental impacts to the planned construction. Following review, additional mitigation or planned foundation elements may need to be modified.

6.1.2 Abandonment of Existing Utilities

All utilities should be completely removed from within planned building areas. For any utility line to be considered acceptable to remain within building areas, the utility line must be completely backfilled with grout or sand-cement slurry (sand slurry is not acceptable), the ends outside the building area capped with concrete, and the trench fills either removed and replaced as engineered fill with the trench side slopes flattened to at least 1:1, or the trench fills are determined not to be a risk to the structure. The assessment of the level of risk posed by the particular utility line will determine whether the utility may be abandoned in place or needs to be completely removed. The contractor should assume that all utilities will be removed from within building areas unless written confirmation is provided from both the owner and the geotechnical engineer.

Utilities extending beyond the building area may be abandoned in place provided the ends are plugged with concrete, they do not conflict with planned improvements, and that the trench fills do not pose significant risk to the planned surface improvements.

The risk for owners associated with abandoning utilities in place include the potential for future differential settlement of existing trench fills, and/or partial collapse and potential ground loss into utility lines that are not completely filled with grout.

6.2 SITE CLEARING AND PREPARATION

6.2.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements to be removed within the proposed development area. Demolition of existing improvements is discussed in the prior paragraphs. A detailed discussion of removal of existing fills is provided later in this report. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight.

6.2.2 Tree and Shrub Removal

Trees and shrubs designated for removal should have the root balls and any roots greater than ½-inch diameter removed completely. Mature trees are estimated to have root balls extending to depths of 2 to 4 feet, depending on the tree size. Significant root zones are anticipated to extend to the diameter of the tree canopy. Grade depressions resulting from root ball removal should be cleaned of loose material and backfilled in accordance with the recommendations in the "Compaction" section of this report.

6.3 MITIGATION OF UNDOCUMENTED FILLS

The site is blanketed by up to 5 feet of undocumented fill. Locally deeper fill in the former underground storage tank removal area may extend to depths of about 8 to 10 feet. The extent and depth of the former UST fill should be further evaluated during site demolition. The mat foundation excavation will likely be on the order of 2 to 3 feet below current site grades; therefore, a majority of the fill will be removed as part of the foundation excavation. The remaining exposed fill should be stabilized and re-compacted as discussed below. In the former UST area, the existing undocumented fill should be over-excavated and re-compacted prior to proceeding with foundation subgrade stabilization.

Based on review of the samples collected from our borings, it appears that the fill in the upper 5 feet may be reused. Re-use of the former UST backfill we need to be determined once building demolition is complete. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should be screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the "Compaction" section below.

6.4 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. On a preliminary basis, the upper 10 feet at the site may be classified as OSHA Soil Type C materials.

Excavations performed during site demolition and fill removal should be sloped at 2:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Actual excavation inclinations should be reviewed in the field during construction, as needed. Excavations below building subgrade and excavations in pavement and flatwork areas should be sloped in accordance with OSHA soil classification requirements.

6.5 BELOW-GRADE EXCAVATIONS

Below-grade excavations may be constructed with temporary slopes in accordance with the “Temporary Cut and Fill Slopes” section above if space allows. A pre-condition survey including photographs and installation of monitoring points for existing site improvements should be included in the contractor’s scope. The project structural engineer and/or grading contractor should be consulted regarding support of adjacent structures.

6.5.1 Underpinning

For the planned mat foundation, where shallow foundations for adjacent buildings are above an imaginary 1:1 line projected up from the bottom of the proposed mat foundation, or where potential settlement due to the proximity of the mat foundation loading will induce ¼ to ½ inch of settlement within 5 to 10 feet of the mat, existing adjacent foundations may need to be underpinned. If underpinning is required, helical anchors, slant piles or offset piers may be acceptable methods to underpin adjacent structures. Underpinning should extend at least 2 to 3 feet into the medium dense to dense sands or 20 feet, whichever is deeper. The underpinning designer should review the subsurface data to estimate ultimate support capacity for the chosen support and should apply an appropriate factor of safety to the ultimate capacity, as required. To reduce movement and provide adequate foundation support during installation of the underpinning piers, adjacent piers should not be drilled or excavated concurrently. We recommend underpinning support should be preloaded prior to dry packing or anchor bolt installation. We should observe the installation of the underpinning anchors/piles/piers to check that adequate embedment has been achieved.

Underpinning support should be designed by the underpinning contractor, and we should review the geotechnical aspects of the underpinning design.

6.5.2 Construction Dewatering

Design groundwater levels are expected to be near or a couple feet above the planned excavation bottom for elevator pits; therefore, temporary dewatering may be necessary during construction. Design, selection of the equipment and dewatering method, and construction of temporary dewatering should be the responsibility of the contractor. Modifications to the dewatering system are often required in layered alluvial soils and should be anticipated by the contractor. The dewatering plan, including planned dewatering well filter pack materials, should be forwarded to our office for review prior to implementation.

The dewatering design should maintain groundwater at least 2 feet below localized excavations such as deepened mat areas, elevator shafts, and utilities. If the dewatering system was to shut down for an extended period of time, destabilization and/or heave of the excavation bottom requiring over-excavation and stabilization, flooding and softening, and/or shoring failures could occur; therefore, we recommend that a backup power source be considered.

Depending on the groundwater quality and previous environmental impacts to the site and surrounding area, settlement and storage tanks, particulate filtration, and environmental testing may be required prior to discharge, either into storm or sanitary, or trucked to an off-site facility.

6.6 SUBGRADE PREPARATION

After site clearing and demolition is complete, and prior to backfilling any excavations resulting from fill removal or demolition, the excavation subgrade and subgrade within areas to receive additional site fills, slabs-on-grade and/or pavements should be scarified to a depth of 12 inches, moisture conditioned and compacted in accordance with the “Compaction” section below.

The proposed mat foundation will extend into near-saturated, medium stiff clays or loose silty sands and near or into groundwater. These soils will be difficult to compact when moisture contents exceed 5 percent above their optimum moisture content. Therefore, we recommend chemically treating exposed fill or native soils at the exposed mat foundation subgrade with lime and/or cement to reduce moisture content, improve soil strength, and to create a stiff building pad to support construction equipment. The depth of chemical treatment should be at least 18 inches to provide an effective, stiff bearing surface for planned mat foundation. For preliminary planning, we suggest at least 4 to 5 percent high-calcium quicklime (by weight) be considered.

6.7 WET SOIL STABILIZATION GUIDELINES

Native soil and fill materials, especially soils with high fines contents such as clays and silty soils, can become unstable due to high moisture content, whether from high in-situ moisture contents or from winter rains. As the moisture content increases over the laboratory optimum, it becomes more likely the materials will be subject to softening and yielding (pumping) from construction loading or become unworkable during placement and compaction.

As discussed in the “Subsurface” section in this report, the in-situ moisture contents are about 2 to 29 percent over the estimated laboratory optimum in the upper 10 feet of the soil profile. For areas outside of the planned mat stabilization area, the contractor may also need to dry the at-grade soils prior to reusing them as fill. In addition, repetitive rubber-tire loading could de-stabilize shallow soils.

There are several methods to address potential unstable soil conditions and facilitate fill placement and trench backfill. Some of the methods are briefly discussed below. Implementation of the appropriate stabilization measures should be evaluated on a case-by-case basis according to the project construction goals and the site conditions.

6.7.1 Scarification and Drying

For shallow grading with 1 to 2 feet of existing grades, the subgrade may be scarified to a depth of 6 to 12 inches and allowed to dry to near optimum conditions, if sufficient dry weather is anticipated to allow sufficient drying. More than one round of scarification may be needed to break up the soil clods.

6.7.2 Removal and Replacement

As an alternative to scarification, the contractor may choose to over-excavate the unstable soils and replace them with dry import materials. A Cornerstone representative should be present to provide recommendations regarding the appropriate depth of over-excavation, whether a geosynthetic (stabilization fabric or geogrid) is recommended, and what materials are recommended for backfill.

6.7.3 Chemical Treatment

Where the unstable area exceeds about 5,000 to 10,000 square feet and/or site winterization is desired, chemical treatment with quicklime (CaO), kiln-dust, or cement may be more cost-effective than removal and replacement. Recommended chemical treatment depths will typically range from 12 to 18 inches depending on the magnitude of the instability.

6.7.4 Mat Foundation Excavation Stabilization

As the planned mat foundation excavation will extend into soil that is too wet to compact and is near or into the design groundwater level, chemical treatment can be considered at the mat foundation subgrade level. Chemical stabilization will also aid constructability and support of ground improvement equipment if this option is required. Due to the variable type and consistency of the fill ranging from fat clay to silty/clayey sand, a material suitable for these soils should be considered. For planning purposes, a minimum of 5 to 6 percent (by weight) chemical treatment should be considered that includes either a 50/50 blend of quicklime and cement, only high-calcium quicklime, or as recommended by the stabilization contractor at the time of construction. The contractor should plan for a minimum 18 inch treatment depth. We recommend an optional cost to over-excavate and chemically treat soils to a depth of 24 to 30 inches also be considered.

6.8 MATERIAL FOR FILL

6.8.1 Re-Use of On-site Soils

On-site soils with an organic content less than 3 percent by weight may be reused as general fill. General fill should not have lumps, clods or cobble pieces larger than 6 inches in diameter; 85 percent of the fill should be smaller than 2½ inches in diameter. Minor amounts of oversize material (smaller than 12 inches in diameter) may be allowed provided the oversized pieces are not allowed to nest together and the compaction method will allow for loosely placed lifts not exceeding 12 inches.

6.8.2 Re-Use of On-Site Site Improvements

We anticipate that significant quantities of asphalt concrete (AC) grindings and aggregate base (AB) and some Portland Cement Concrete (PCC) could potentially be generated during site demolition; however, we assume site constraints will prevent on-site grinding and crushing during demolition.

If the site area allows for on-site pulverization of PCC and provided the PCC is pulverized to meet the “Material for Fill” requirements of this report, it may be used as select fill within the building footprint, excluding the capillary break layer; as typically pulverized PCC comes close to or meets Class 2 AB specifications, the recycled PCC may likely be used beneath the building.

6.8.3 Potential Import Sources

Non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less, and not contain recycled asphalt concrete where it will be used within the habitable building areas. To prevent significant caving during trenching or foundation construction, imported material should have sufficient fines. Samples of potential import sources should be delivered to our office at least 10 days prior to the desired import start date. Information regarding the import source should be provided, such as any site geotechnical reports. If the material will be derived from an excavation rather than a stockpile, potholes will likely be required to collect samples from throughout the depth of the planned cut that will be imported. At a minimum, laboratory testing will include PI tests. Material data sheets for select fill materials (Class 2 aggregate base, ¾-inch crushed rock, quarry fines, etc.) listing current laboratory testing data (not older than 6 months from the import date) may be provided for our review without providing a sample. If current data is not available, specification testing will need to be completed prior to approval.

Environmental and soil corrosion characterization should also be considered by the project team prior to acceptance. Suitable environmental laboratory data to the planned import quantity should be provided to the project environmental consultant; additional laboratory testing may be required based on the project environmental consultant’s review. The potential import source should also not be more corrosive than the on-site soils, based on pH, saturated resistivity, and soluble sulfate and chloride testing.

6.9 COMPACTION REQUIREMENTS

All fills, and subgrade areas where fill, slabs-on-grade, and pavements are planned, should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in the table below. In general, clayey soils should be compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory equipment; open-graded materials such as crushed rock should be placed in lifts no thicker than 18 inches and consolidated in place with vibratory equipment. Each lift of fill and all subgrade should be firm and unyielding under construction equipment loading in addition to meeting the compaction requirements to be approved. The contractor (with input from a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions. General recommendations for soil stabilization are provided in the "Wet Soil Stabilization Guidelines" section of this report. Where the soil's PI is 20 or greater, the expansive soil criteria should be used.

Table 2: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Expansive Soils	87 – 92	>3
	Low Expansion Soils	90	>1
UST Fill Over-Excavation (below a depth of 5 feet)	On-Site Expansive Soils	92	>3
	Low Expansion Soils	95	>1
Trench Backfill	On-Site Expansive Soils	87 – 92	>3
	Low Expansion Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Low Expansion Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum
Flatwork Subgrade	On-Site Expansive Soils	87 - 92	>3
	Low Expansion Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade (Public)	On-Site Expansive Soils	87 - 92	>3
	Low Expansion Soils	95	>1
Pavement Aggregate Base (Public)	Class 2 Aggregate Base[3]	95	Optimum

1 – Relative compaction based on maximum density determined by ASTM D1557 (latest version)
2 – Moisture content based on optimum moisture content determined by ASTM D1557 (latest version)
3 – Class 2 aggregate base shall conform to Caltrans Standard Specifications, latest edition, except that the relative compaction should be determined by ASTM D1557 (latest version)

6.9.1 Construction Moisture Conditioning

Expansive soils can undergo significant volume change when dried then wetted. The contractor should keep all exposed expansive soil subgrade (and also trench excavation side walls) moist until protected by overlying improvements (or trenches are backfilled). If expansive soils are allowed to dry out significantly, re-moisture conditioning may require several days of re-wetting (flooding is not recommended), or deep scarification, moisture conditioning, and re-compaction.

6.10 TRENCH BACKFILL

Utility lines constructed within public right-of-way should be trenched, bedded and shaded, and backfilled in accordance with the local or governing jurisdictional requirements. Utility lines in private improvement areas should be constructed in accordance with the following requirements unless superseded by other governing requirements.

All utility lines should be bedded and shaded to at least 6 inches over the top of the lines with crushed rock (⅜-inch-diameter or greater) or well-graded sand and gravel materials conforming to the pipe manufacturer's requirements. Open-graded shading materials should be consolidated in place with vibratory equipment and well-graded materials should be compacted to at least 90 percent relative compaction with vibratory equipment prior to placing subsequent backfill materials.

General backfill over shading materials may consist of on-site native materials provided they meet the requirements in the "Material for Fill" section, and are moisture conditioned and compacted in accordance with the requirements in the "Compaction" section.

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence, or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

On expansive soils sites it is desirable to reduce the potential for water migration into building and pavement areas through granular shading materials. We recommend that a plug of low-permeability clay soil, sand-cement slurry, or lean concrete be placed within trenches just outside where the trenches pass into building and pavement areas.

6.10.1 Flexible Utility Connections

For a rigid mat foundation that is not supported on ground improvement, we anticipate about 2 to 4 inches of long-term consolidation settlement will occur following construction. Flexible utility connections are recommended for critical utilities such as the water and gas lines and electrical trenches that will be connected to the proposed building. In addition, gravity flow utilities such as storm and sewer should be designed to accommodate the settlement to prevent grade reversal from foundation to public right-of-way areas. Depending on the settlement mitigation measures chosen for the project, we can provide additional recommendations, as needed.

6.11 SITE DRAINAGE

Ponding should not be allowed adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 2 percent towards suitable discharge facilities; landscape areas should slope at least 3 percent towards suitable discharge facilities. Roof runoff should be directed away from building areas in closed conduits, to approved infiltration facilities, or on to hardscaped surfaces that drain to suitable facilities. Retention, detention or infiltration facilities located immediately adjacent to the building should be designed with a liner to prevent water migration into soils immediately adjacent to the foundation.

SECTION 7: 2022 CBC SEISMIC DESIGN CRITERIA

7.1 SEISMIC DESIGN CRITERIA

We understand that the project structural design will be based on the 2022 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The “Seismic Coefficients” used to design buildings are established based on a series of tables and figures addressing different site factors, including the time-weighted average shear wave velocity of the top approximately 100 feet (30 meters) of the soil profile (V_{S30})/soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system.

Our boring explorations generally encountered younger and older alluvial deposits to a depths of approximately 30 to 60 feet. Our CPTs encountered refusal at depths ranging from 53 to 67 feet likely indicated the presence of bedrock near the CPT refusal depths. Shear wave velocity (V_S) measurements were performed while advancing CPT-1, resulting in a time-averaged shear wave velocity for the top 30 meters (V_{S30}) of 278 meters per second (910 feet per second). Therefore, we have classified the site as Soil Classification D. Because we used site specific data from our explorations and laboratory testing, the site class should be considered as “determined” for the purposes of estimating the seismic design parameters from the code outlined below. The mapped spectral acceleration parameters S_s and S_1 were calculated using the web-based program ATC Hazards by Locations, located at https://hazards.atcouncil.org/, based on the site coordinates presented below and the site classification. From our discussion with the project structural engineer, the exception can be taken per ASCE 7-16 Section 11.4.8. Recommended values for design are presented in Table 4. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 3: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.971885°
Site Longitude	-122.520472°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.5g
1-second Period Mapped Spectral Acceleration[1], S_1	0.6g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv[1]	1.7
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.5g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}[1]	1.02g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.0g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}[1]	0.68g
Site Amplification Factor at PGA – F_{PGA}	1.1
Site Modified Peak Ground Acceleration – PGA_M	0.56g

1 – Per project structural engineer, values determined based on 11.4.8 of ASCE 7-16 after the exception is taken

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed building may be supported on a shallow mat foundation provided the anticipated static and seismic settlements are tolerable, and the recommendations in the “Earthwork” section and the sections below are followed. As discussed in the “Conclusions” section, based on discussions with the design team, a rigid mat foundation can be designed to tolerate anticipated static and seismic settlement. Foundation recommendations are presented in the following sections.

8.2 REINFORCED CONCRETE MAT FOUNDATION

The structure may be supported on a mat foundation bearing on engineered fill prepared in accordance with the “Earthwork” section of this report and designed in accordance with the recommendations below. Reinforced concrete mat foundations should be designed in accordance with the 2022 California Building Code. The following criteria is based on a mat without ground improvement.

For our analysis, based on structural loading provided by VCA Structural, we applied a mat contact pressures ranging from approximately 1,100 to 1,400 psf for dead plus live loads across the central portion of the mat and localized edge mat pressures averaging approximately 2,000 psf. The areal pressure is applied at the bottom of the anticipated mat foundation plus an additional 1½ foot depth since the building pad will be lime-treated. The maximum allowable localized bearing pressure should be limited to 2,500 psf at wall or column load locations. When evaluating wind and seismic conditions, allowable bearing pressures may be increased by one-third. These pressures are net values; the weight of the mat may be neglected for the portion of the mat extending below grade. Top and bottom mats of reinforcing steel should be included as required to help span irregularities and differential settlement. If the assumed weight (average areal bearing pressure) is higher than assumed maximum, or there are other aspects of design not accounted for in this report, please notify us so that we may revise our recommendations.

8.2.1 Mat Foundation Settlement

We calculated estimated static settlement of the building using the Rocscience Settle3 program. We evaluated the static settlement at different time stages following construction. We estimated settlements for the 1-, 5-, 30-, and 50-year lifespan of the building. The 50-year static settlements for the building range from approximately 2 to 4½ inches with estimated static differential settlements of approximately 2 inches from the center to the edge of the mat. Approximately 50 to 75 percent of the consolidation settlement is estimated to occur within the first 1 to 2 years following building completion. In addition, the mat should also be designed to accommodate the estimated ¾ inch differential seismic settlement across the mat foundation.

8.2.2 Lateral Loading

Lateral loads may be resisted by friction between the bottom of mat foundation and the supporting subgrade, and also by passive pressures generated against deepened mat edges. An ultimate frictional resistance of 0.45 applied to the mat dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. The upper 12 inches of soil should be neglected when determining passive pressure capacity when adjacent to landscaping.

8.2.3 Mat Modulus of Soil Subgrade Reaction

The modulus of soil subgrade reaction is a model element that represents the response to a specific loading condition, including the magnitude, rate, and shape of loading, given the subsurface conditions at that location. Design experts recommend using a variable modulus of soil subgrade reaction to provide a more accurate soil response and prediction of shears and moments in the mats. This required two iterations between our soil model and the structural SAFE analysis for the mat. As discussed above, the structural engineer provided areal mat pressures ranging from approximately 1,100 to 1,300 psf within the central portion of the structure, increasing to approximately 2,000 psf near the edges of the mat. Based on these updated pressures, we calculated a variable modulus of subgrade reaction value for the mat foundation.

For the SAFE runs, we recommend final modulus of soil subgrade reaction values ranging from approximately 4 to 12 kips per cubic foot (kcf), as shown on the attached Figure 4.

8.2.4 Mat Foundation Construction Considerations

Prior to mat construction or placement of vapor retarder or waterproofing, the subgrade should be proof-rolled and visually observed by a Cornerstone representative to confirm stable subgrade conditions. The building pad should generally be kept free of water and disturbed materials prior to pouring the foundation. The building pad should also be watered occasionally to avoid desiccation and cracking prior to placing waterproofing.

8.2.5 Hydrostatic Uplift and Waterproofing

Mat foundations that extend below the recommended design groundwater level of 3 feet should be designed to resist potential hydrostatic uplift pressures. Elevator pit walls extending below design groundwater should be designed to resist hydrostatic pressure for the full wall height.

In addition, the portions of the structure extending below design groundwater should be waterproofed to limit moisture infiltration, including mat foundation, all construction joints, and any elevator pit retaining walls. We recommend that a waterproofing specialist design the waterproofing system.

SECTION 9: CONCRETE SLABS AND VEHICULAR PAVEMENTS

9.1 PEDESTRIAN CONCRETE FLATWORK

Exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading should be at least 4 inches thick and supported on at least 6 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the “Earthwork” recommendations of this report. Flatwork that will be subject to heavier or frequent vehicular loading should increase the concrete section 6-inches-thick. Concrete flatwork in public rights-of way should be designed in accordance with City of San Rafael requirements.

If the mat foundation will be constructed without ground improvement, new flatwork at building entrances should be designed as a hinged slab that is connected to the edge of the mat using dowels and adequate construction and control joints to tolerate the anticipated differential settlement. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork in non-egress areas should be isolated from adjacent foundations to allow for potential future movement.

9.2 VEHICULAR PAVEMENTS

Where future pavements are planned such as the planned driveway entrance into the building or rehabilitating the roadways adjacent to the project site, those pavements should be designed according to City of San Rafael standards and specifications.

SECTION 10: RETAINING WALLS

10.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall, such as elevator pit walls, should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall. Provided a drainage system is constructed behind the wall to prevent the build-up of hydrostatic pressures as discussed in the section below, we recommend that the walls with level backfill be designed for the following pressures, which includes hydrostatic pressure for an undrained wall:

Table 4: Recommended Lateral Earth Pressures

Wall Condition	Lateral Earth Pressure*	Additional Surcharge Loads
Restrained – Braced Wall	85 pcf + 8H** psf	½ of vertical loads at top of wall

* Lateral earth pressures are based on an equivalent fluid pressure for level backfill conditions as well as added hydrostatic pressure

** H is the distance in feet between the bottom of footing and top of retained soil

As discussed above, the design groundwater level is recommended at a depth of 3 feet below current site grades. The retaining walls should be designed to resist restrained lateral earth pressures combined with hydrostatic pressures. Damp proofing or waterproofing of the walls may be considered where moisture penetration and/or efflorescence are not desired.

10.2 SEISMIC LATERAL EARTH PRESSURES

The 2022 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. Currently, we are not aware of any retaining walls for the project greater than 6 feet in height. In our opinion, design of these walls for seismic lateral earth pressures in addition to static earth pressures is not warranted.

10.3 WALL DRAINAGE

As discussed above, since the design groundwater depth is at 3 feet, retaining walls for elevator pits should be designed for the undrained conditioned and be designed to resist added hydrostatic pressure.

10.4 BACKFILL

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 95 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

10.5 FOUNDATIONS

The retaining walls for elevator pits are likely to be supported on the planned mat foundation. The mat foundation supporting basement retaining walls should be designed in accordance with the recommendations presented in the "Foundations" and "Earthwork" sections of this report.

SECTION 11: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Mill Creek Residential Trust specifically to support the design of the Modera San Rafael project in San Rafael, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

Recommendations in this report are based upon the soil and groundwater conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

Mill Creek Residential Trust may have provided Cornerstone with plans, reports and other documents prepared by others. Mill Creek Residential Trust understands that Cornerstone reviewed and relied on the information presented in these documents and cannot be responsible for their accuracy.

Cornerstone prepared this report with the understanding that it is the responsibility of the owner or their representatives to see that the recommendations contained in this report are presented to other members of the design team and incorporated into the project plans and specifications, and that appropriate actions are taken to implement the geotechnical recommendations during construction.

Conclusions and recommendations presented in this report are valid as of the present time for the development as currently planned. Changes in the condition of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Therefore, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes beyond Cornerstone's control. This report should be reviewed by Cornerstone after a period of three (3) years has elapsed from the date of this report. In addition, if the current project design is changed, then Cornerstone must review the proposed changes and provide supplemental recommendations, as needed.

An electronic transmission of this report may also have been issued. While Cornerstone has taken precautions to produce a complete and secure electronic transmission, please check the electronic transmission against the hard copy version for conformity.

Recommendations provided in this report are based on the assumption that Cornerstone will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design, and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, Cornerstone cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

SECTION 12: REFERENCES

Aagaard, B.T., Blair, J.L., Boatwright, J., Garcia, S.H., Harris, R.A., Michael, A.J., Schwartz, D.P., and DiLeo, J.S., 2016, Earthquake outlook for the San Francisco Bay region 2014–2043 (ver. 1.1, August 2016): U.S. Geological Survey Fact Sheet 2016–3020, 6 p., http://dx.doi.org/10.3133/fs20163020.

ATC Hazards by Location, Hazards by Location, 2024, https://hazards.atcouncil.org/

ASCE 7-16. American Society of Civil Engineers. (2016). *Minimum Design Loads and Associated Criteria for Buildings and Other Structures.*

Boulanger, R.W. and Idriss, I.M., 2014, CPT and SPT Based Liquefaction Triggering Procedures, Department of Civil & Environmental Engineering, College of Engineering, University of California at Davis, Report No. UCD/GCM-14/01, April 2014

California Building Code, 2022, Structural Engineering Design Provisions, Vol. 2.

California Division of Mines and Geology (2008), "Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, September.

CGS. (2021). *Earthquake Zones of Required Investigation.* CGS Homepage. https://maps.conservation.ca.gov/cgs/EQZApp/app/

Federal Emergency Management Administration (FEMA), effective March 16, 2016, FIRM City of San Rafael, California, Community Panel #06041C0457E.

Idriss, I.M., and Boulanger, R.W., 2008, Soil Liquefaction During Earthquakes, Earthquake Engineering Research Institute, Oakland, CA, 237 p.

Ritter, J.R., and Dupre, W.R., 1972, Map Showing Areas of Potential Inundation by Tsunamis in the San Francisco Bay Region, California: San Francisco Bay Region Environment and Resources Planning Study, USGS Basic Data Contribution 52, Misc. Field Studies Map MF-480.

Robertson, P.K., Shao, Lisheng, 2010, Estimation of Seismic Compression in Dry Soils Using the CPT, 5th International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Paper No. 4.05a, May 24-29, 2010.

Seed, Raymond B., Cetin, K.O., Moss, R.E.S., Kammerer, Ann Marie, Wu, J., Pestana, J.M., Riemer, M.F., Sancio, R.B., Bray, Jonathan D., Kayen, Robert E., and Faris, A., 2003, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework., University of California, Earthquake Engineering Research Center Report 2003-06.

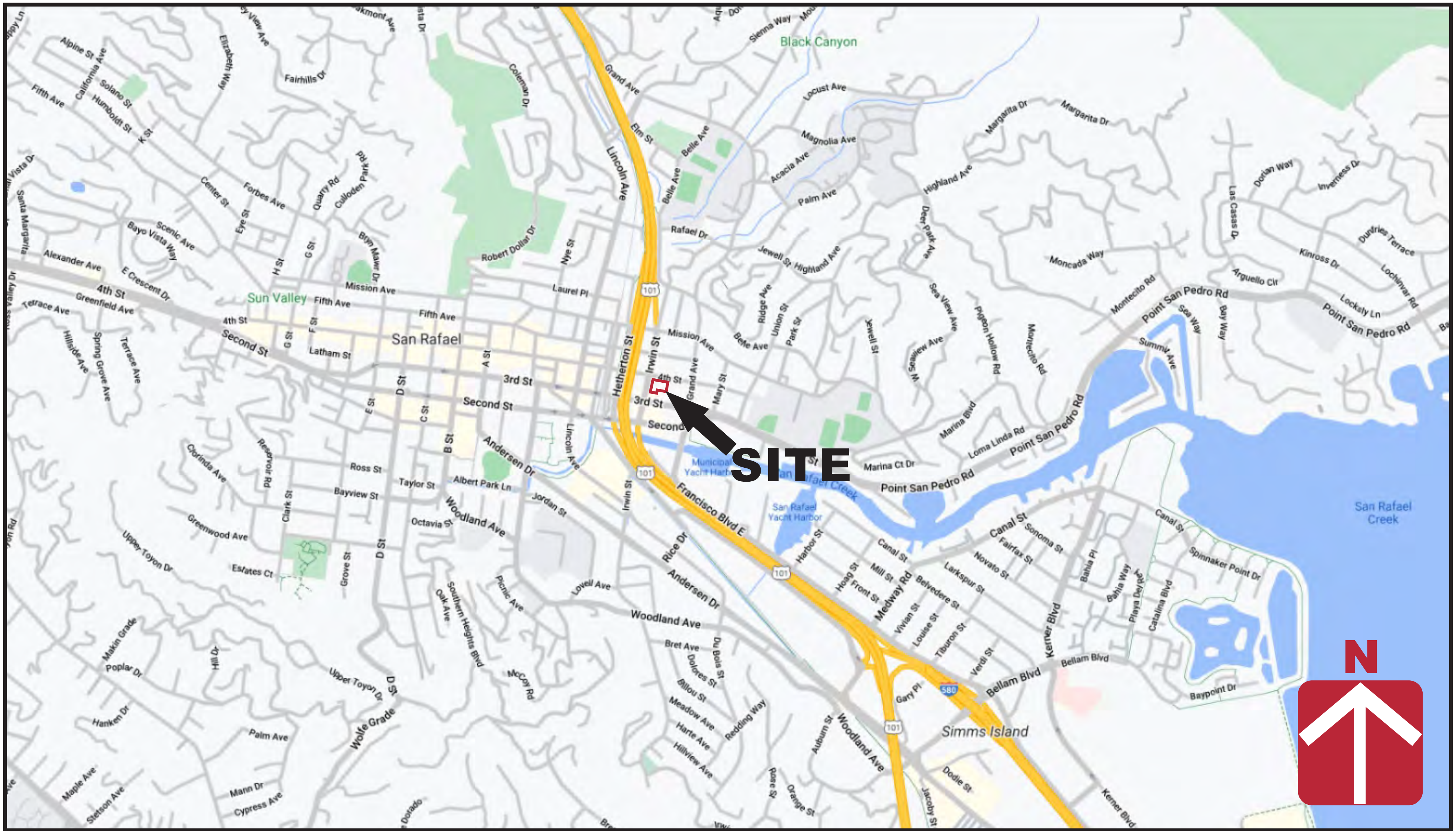
Witter, R.C., Knudsen, K.L., Sowers, J.M., Wentworth, C.M., Koehler, R.D., Randolph, C.E., Brooks, S, K. and Gans, K.D., 2006, Maps of Quaternary deposits and liquefaction susceptibility in the central San Francisco Bay region, California: U.S. Geological Survey, Open-File Report OF-2006-1037, scale 1:200000.

Working Group on California Earthquake Probabilities, 2015, The Third Uniform California Earthquake Rupture Forecast, Version 3 (UCERF), U.S. Geological Survey Open File Report 2013-1165 (CGS Special Report 228)*. KMZ files available at:* www.scec.org/ucerf/images/ucerf3_timedep_30yr_probs.kmz

Youd, T.L. and C.T. Garris, 1995, Liquefaction-Induced Ground-Surface Disruption: Journal of Geotechnical Engineering, Vol. 121, No. 11, pp. 805 - 809.

Youd, T.L. and Idriss, I.M., et al, 1997, Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils: National Center for Earthquake Engineering Research, Technical Report NCEER - 97-0022, January 5, 6, 1996.

Youd et al., 2001, “Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils,” ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vo. 127, No. 10, October, 2001.

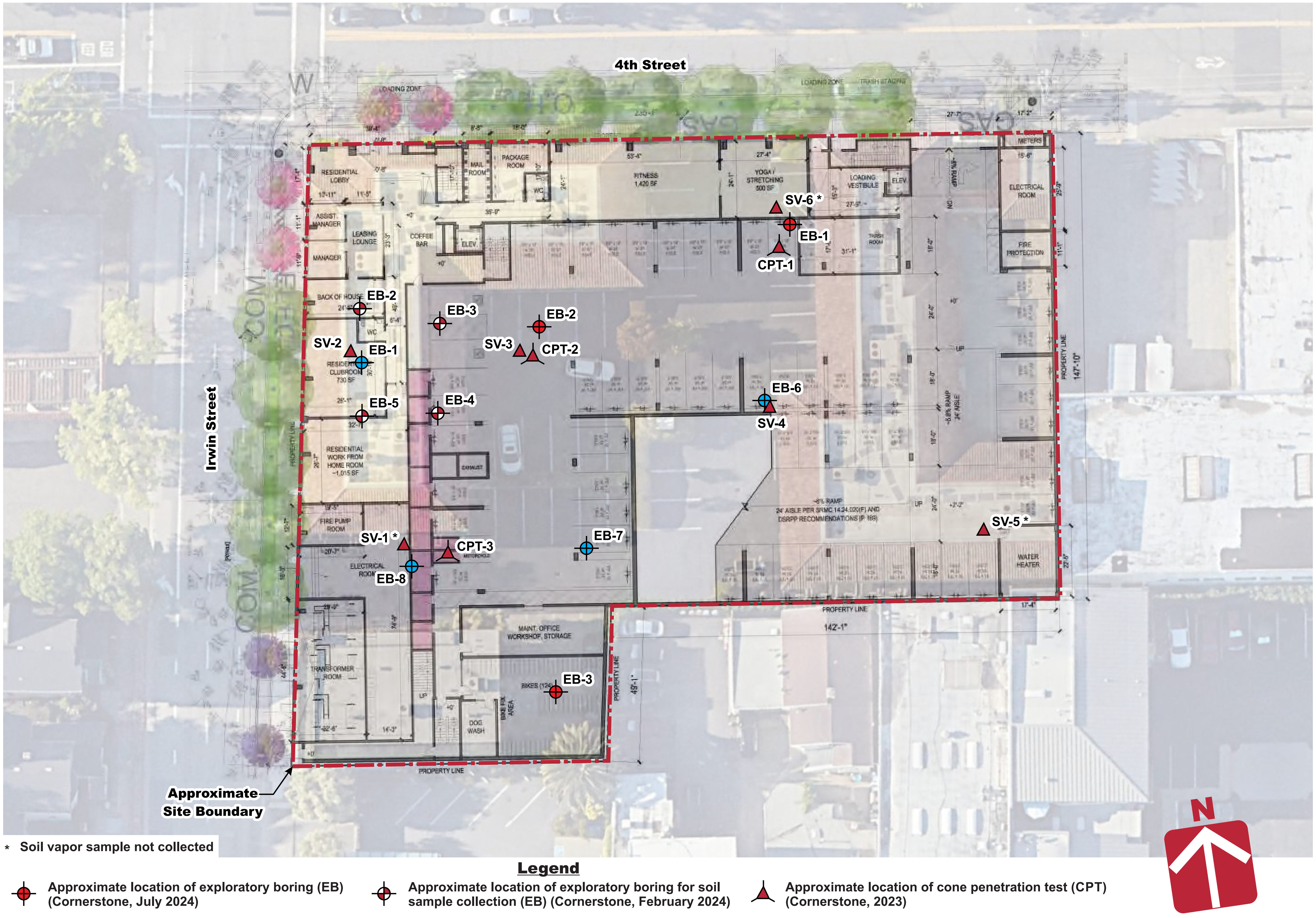


Vicinity Map

CORNERSTONE EARTH GROUP

523 & 535 4th Street and
912, 914, and 930 Irwin Street
San Rafael, CA

Project Number	1395-4-4
Figure Number	Figure 1
Date	July 2024
Drawn By	RRN



* Soil vapor sample not collected

Legend

⊕	Approximate location of exploratory boring (EB) (Cornerstone, July 2024)
⊕	Approximate location of exploratory boring for soil sample collection (EB) (Cornerstone, February 2024)
⊕	Approximate location of exploratory boring for groundwater and soil sample collection (EB) (Cornerstone, February 2024)
▲	Approximate location of cone penetration test (CPT) (Cornerstone, 2023)
▲	Approximate location of soil vapor sample (SV) (Cornerstone, 2023)

0 30 60
APPROXIMATE SCALE (FEET)

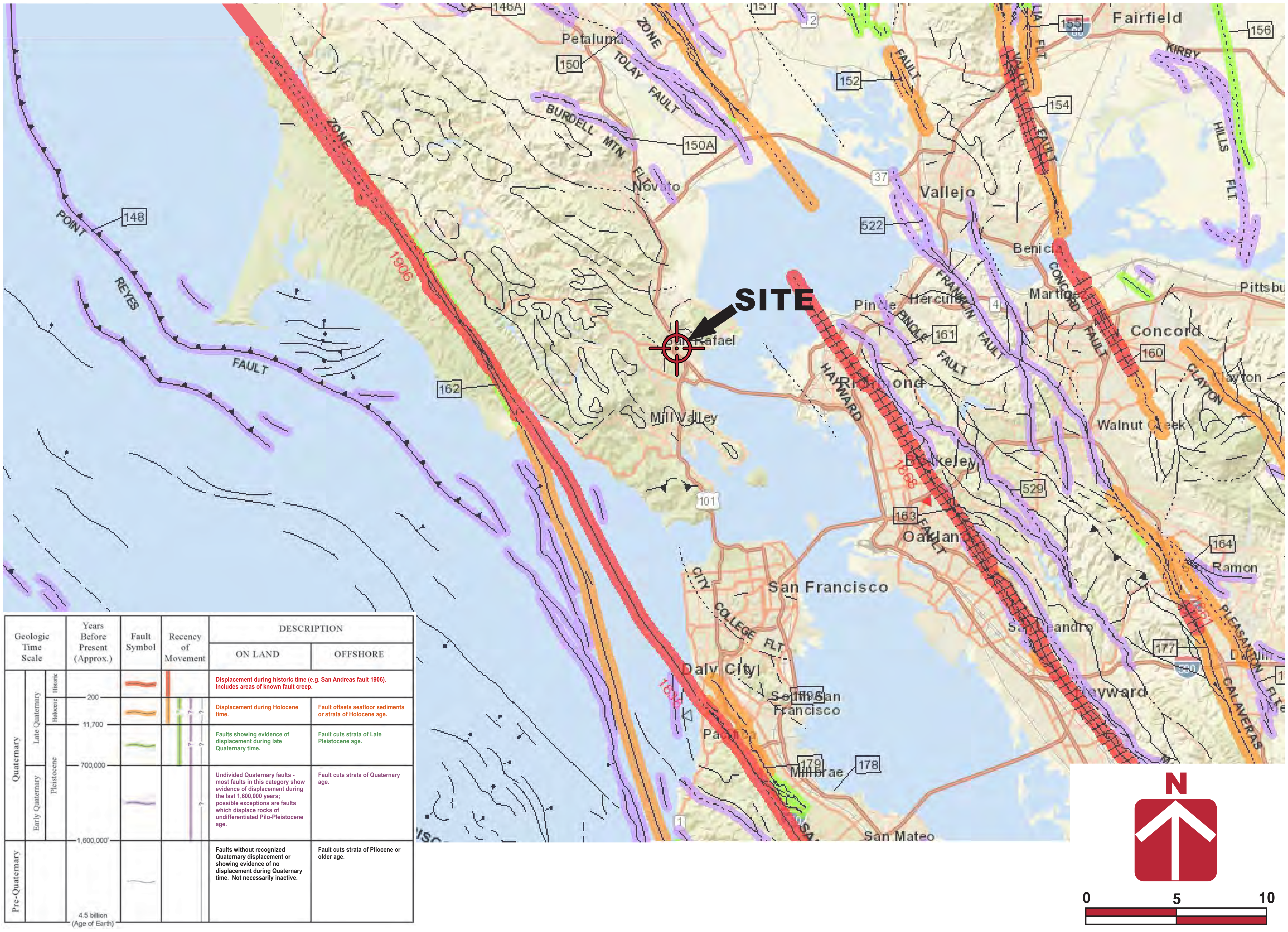
Base by Google Earth, dated 05/29/2023.
Overlay by Trachtenberg Architects, Plan at Ground Level - A2, dated 08/16/2024.

CORNERSTONE EARTH GROUP

Site Plan

523 & 535 4th Street and
912, 914, and 930 Irwin Street
San Rafael, CA

Project Number 1395-4-4
Figure Number Figure 2
Date August 2024
Drawn By RRN



Base by California Geological Survey - 2010 Fault Activity Map of California (Jennings and Bryant, 2010)

Project Number
1395-4-4

Figure Number
Figure 3

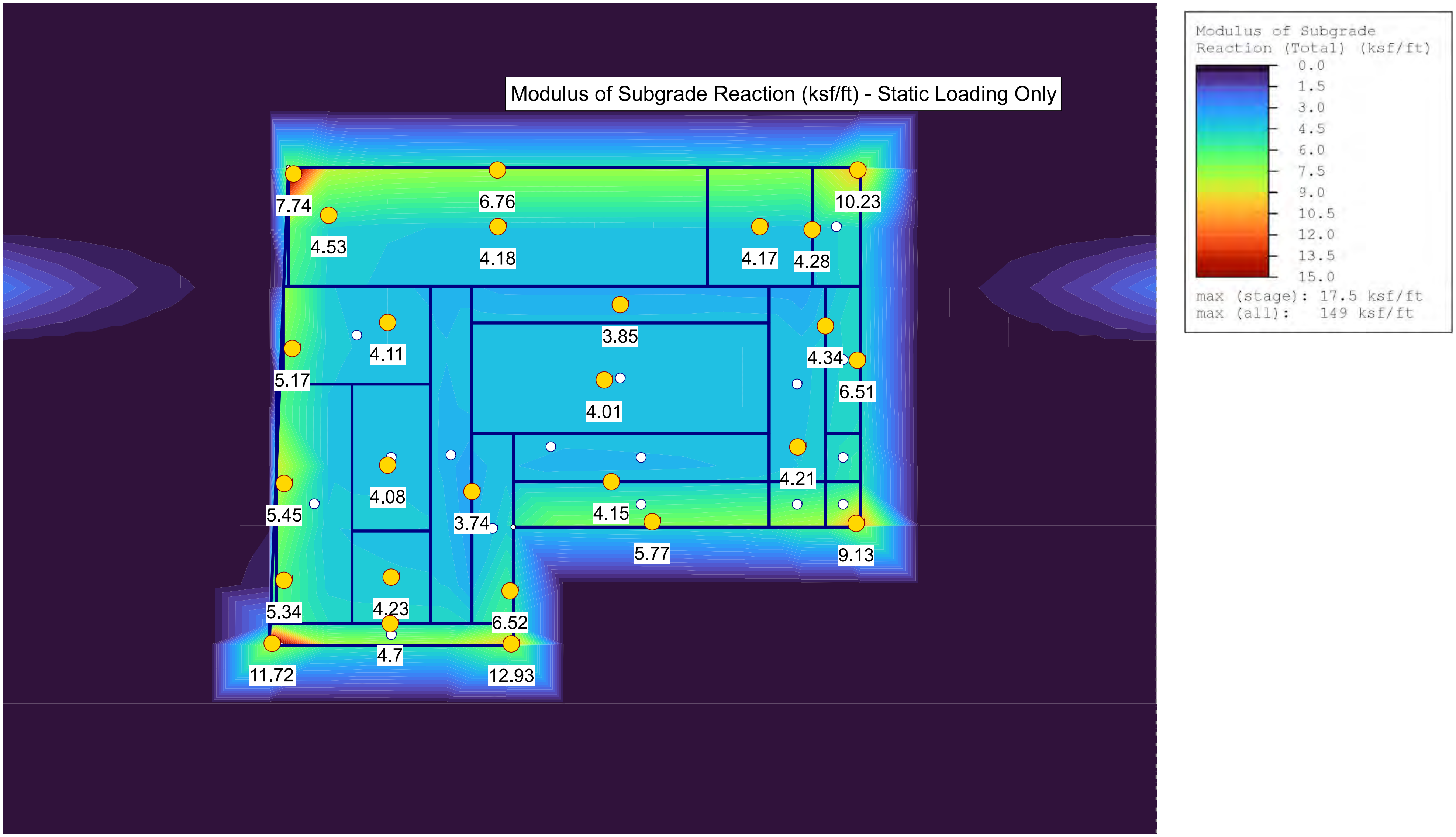
Date
July 2024

Drawn By
RRN

Regional Fault Map

523 & 535 4th Street and
912, 914, and 930 Irwin Street
San Rafael, CA

CORNERSTONE
EARTH GROUP



CORNERSTONE EARTH GROUP

Variable Modulus of Subgrade Reaction for Mat Foundation

523 & 535 4th Street and
912, 914, and 930 Irwin Street
San Rafael, CA

Project Number	1395-4-4
Figure Number	Figure 4
Date	February 2025
Drawn By	RRN

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem, auger drilling equipment and 30-ton truck-mounted Cone Penetration Test equipment. Three 8-inch-diameter exploratory borings were drilled on July 17, 2024, to depths of 30 to 60 feet. Three CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on December 18, 2023, to depths ranging from 53 to 67 feet. Additional environmental push probe borings were also performed during December 2023 and February 2024 using a track mounted push-probe rig. The approximate locations of exploratory borings and CPTs are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

Boring and CPT locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. Boring and CPT elevations were based on interpolation of survey plan contours. The locations and elevations of the borings and CPTs should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Relatively undisturbed samples were also obtained with 2.875-inch I.D. Shelby Tube sampler which were hydraulically pushed. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

The CPT involved advancing an instrumented cone-tipped probe into the ground while simultaneously recording the resistance at the cone tip (q_c) and along the friction sleeve (f_s) at approximately 5-centimeter intervals. Based on the tip resistance and tip to sleeve ratio (R_f), the CPT classified the soil behavior type and estimated engineering properties of the soil, such as equivalent Standard Penetration Test (SPT) blow count, internal friction angle within sand layers, and undrained shear strength in silts and clays. A pressure transducer behind the tip of the CPT cone measured pore water pressure (u_2). Graphical logs of the CPT data is included as part of this appendix.

Field tests included an evaluation of the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

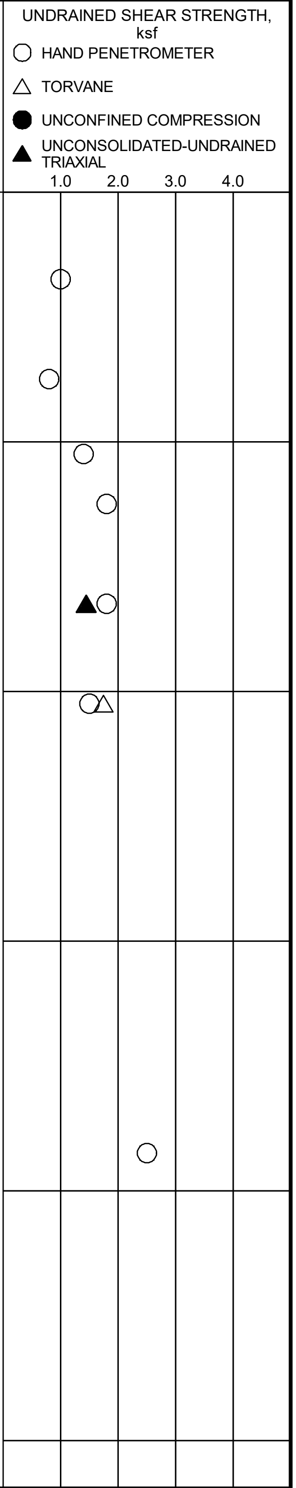
Attached boring and CPT logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring and CPT locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

BORING NUMBER EB-1

PROJECT NAME 4th and Irwin Street
PROJECT NUMBER 1395-4-4
PROJECT LOCATION San Rafael, CA
DATE STARTED 7/17/24 **DATE COMPLETED** 7/17/24
GROUND ELEVATION 13 FT +/- **BORING DEPTH** 30 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.971967° **LONGITUDE** -122.520125°
DRILLING METHOD Mobile B-53B, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RA
▽ **AT TIME OF DRILLING** 7 ft.
NOTES
▼ **AT END OF DRILLING** 7 ft.

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
13.0	0		3 inches asphalt concrete over 5 inches aggregate base						
12.8									
12.3			**Clayey Sand (SC) [Fill]** medium dense, moist, gray, fine to medium sand						
				12	MC-1B	109	9		14
11.0			**Sandy Lean Clay (CL) [Fill]** medium stiff, moist, gray, fine to medium sand, some shell fragments, moderate plasticity						
				11	MC-2B	104	21		
8.8			**Silty Sand (SM) [Fill]** loose, moist, gray and brown mottled, fine sand						
8.0	5		**Lean Clay with Sand (CL)** stiff, moist, dark gray to gray with brown mottles, fine to medium sand, moderate plasticity						
				13	3A MC 3B	98 103	24 23		
				13	MC-4B	108	20		
	10				ST-5	107	20		
1.0			**Clayey Sand with Gravel (SC)** medium dense, moist, reddish brown with gray mottles, fine to coarse sand, fine to coarse subangular gravel						
				30	MC-6B	110	17		
	15								
-4.0			**Sandy Lean Clay (CL)** very stiff, moist, gray and reddish brown mottled, fine to coarse sand, moderate plasticity						
				25	MC-7B	108	21		
	20								
-9.0			**Clayey Sand with Gravel (SC)** dense, moist, reddish brown with gray mottles, fine to coarse sand, fine to coarse subangular gravel						
				49	MC-8B	117	15		21
-12.0	25								



Continued Next Page

BORING NUMBER EB-1

PAGE 2 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME 4th and Irwin Street
PROJECT NUMBER 1395-4-4
PROJECT LOCATION San Rafael, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
-12.0	25	▨								
-14.0		▨	**Sandy Lean Clay (CL)** very stiff, moist, reddish brown with gray mottles, fine to medium sand, moderate plasticity							
		▨		51	⧗ MC-9B	107	20			○
-17.0	30		Bottom of Boring at 30.0 feet.							
	35									
	40									
	45									
	50									

CORNERSTONE EARTH GROUP

BORING NUMBER EB-2

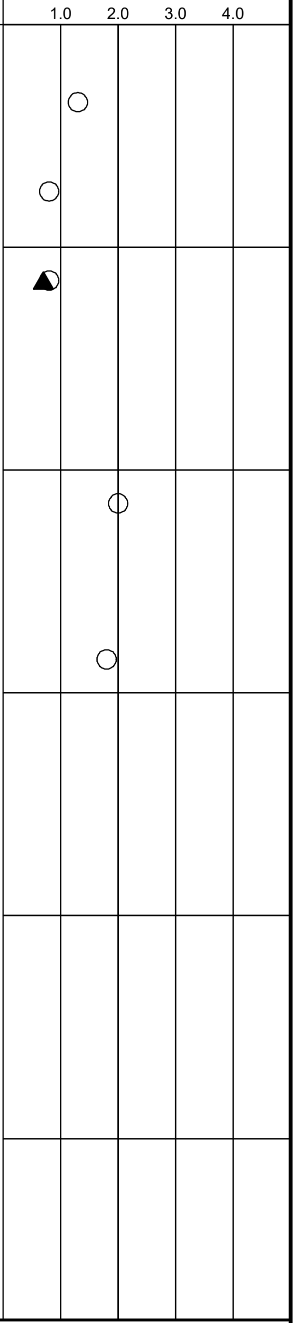
PAGE 1 OF 2

PROJECT NAME 4th and Irwin Street
PROJECT NUMBER 1395-4-4
PROJECT LOCATION San Rafael, CA
DATE STARTED 7/17/24 **DATE COMPLETED** 7/17/24
GROUND ELEVATION 13 FT +/- **BORING DEPTH** 60 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.971923° **LONGITUDE** -122.520460°
DRILLING METHOD Mobile B-53B, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY RA
▽ **AT TIME OF DRILLING** 10 ft.
NOTES
▼ **AT END OF DRILLING** 7 ft.

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
13.0 12.8	0	3 inches asphalt concrete over 6 inches aggregate base						
12.3		**Lean Clay with Sand (CL) [Fill]** stiff, moist, gray with brown mottles, fine to medium sand, moderate to high plasticity	25	MC-1B	90	35		
10.5		**Silty Sand (SM) [Fill]** loose, moist, gray and brown mottled, fine to medium sand	13	MC-2B	68	67		
9.0	5	**Fat Clay (CH) [Bay Mud/Bay Mud Crust]** medium stiff, moist, dark gray, some fine sand, high plasticity	7	MC-3B	99	26		
7.5		**Sandy Lean Clay (CL)** medium stiff, moist, gray, fine to medium sand, moderate plasticity						
5.5		**Clayey Sand (SC)** medium dense, moist, gray with brown mottles, fine to medium sand		ST-4	97	19		30
2.5	10	**Sandy Lean Clay (CL)** stiff, moist, gray with brown mottles, fine to medium sand, low plasticity	9	NR-5		20		
			32	MC-6B	107	21		
-2.0	15	**Silty Sand (SM)** medium dense, moist, gray with reddish brown mottles, fine to medium sand, some fine to coarse subangular to subrounded gravel	30	MC-7B	110	19		
			21	SPT-8		14		20
	20	decreasing gravel content	26	MC-9B	105	20		33
-9.5		**Clayey Sand (SC)** medium dense, moist, gray with reddish brown mottles, fine to medium sand	38	MC-10B	107	19		48
-13.0	25	**Lean Clay with Sand (CL)** hard, moist, reddish brown with gray mottles, fine sand, moderate plasticity						
-15.0								

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL



Continued Next Page

BORING NUMBER EB-2

PROJECT NAME 4th and Irwin Street

PROJECT NUMBER 1395-4-4

PROJECT LOCATION San Rafael, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
-15.0		▨	**Lean Clay with Sand (CL)** hard, moist, reddish brown with gray mottles, fine sand, moderate plasticity	35	MC-11B	103	24		
	30	▨							
		▨	becomes very stiff	35	MC-12B	114	17		
-21.5	35	▨	**Clayey Sand with Gravel (SC)** medium dense, moist, reddish brown with gray mottles, fine to coarse sand, fine to coarse subangular gravel						
-24.0		▨	**Lean Clay with Sand (CL)** hard, moist, gray with brown mottled, fine to medium sand, moderate plasticity						
		▨		23	MC				
	40	▨							
		▨	becomes very stiff	35	MC-14B	103	21		
	45	▨							
		▨	becomes stiff	19	SPT-15		23		
	50	▨							
-39.0		▨	**Lean Clay (CL)** very stiff, moist, gray, some fine sand, moderate plasticity						
		▨		20	MC				
	55	▨							
-44.0		▨	**Clayey Sand with Gravel (SC)** medium dense, moist, reddish brown with gray mottles, fine to coarse sand, fine to coarse subangular gravel	38	MC-17B	120	14		
-47.0	60		Bottom of Boring at 60.0 feet.						

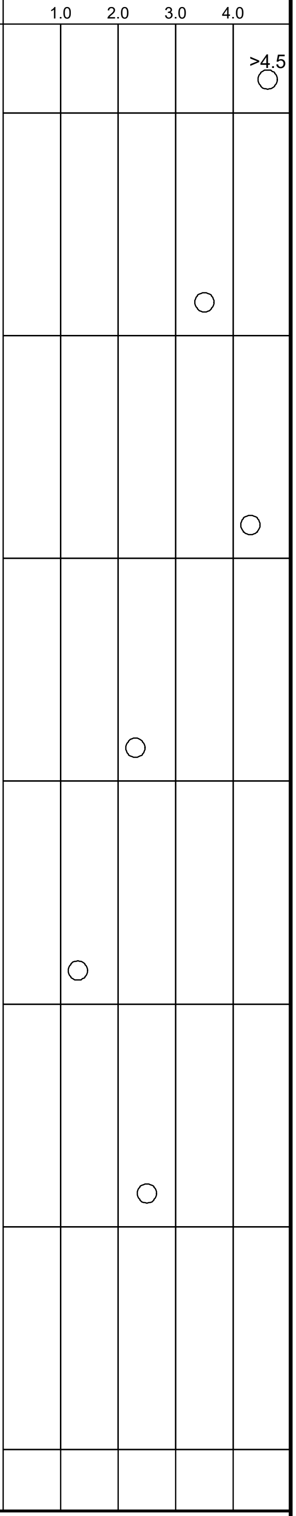
UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL



BORING NUMBER EB-3

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME 4th and Irwin Street

PROJECT NUMBER 1395-4-4

PROJECT LOCATION San Rafael, CA

DATE STARTED 7/17/24 **DATE COMPLETED** 7/17/24

GROUND ELEVATION 12 FT +/- **BORING DEPTH** 30 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE 37.971658° **LONGITUDE** -122.520469°

DRILLING METHOD Mobile B-53B, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY RA

∇ **AT TIME OF DRILLING** 6 ft.

NOTES

▼ **AT END OF DRILLING** 6 ft.

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

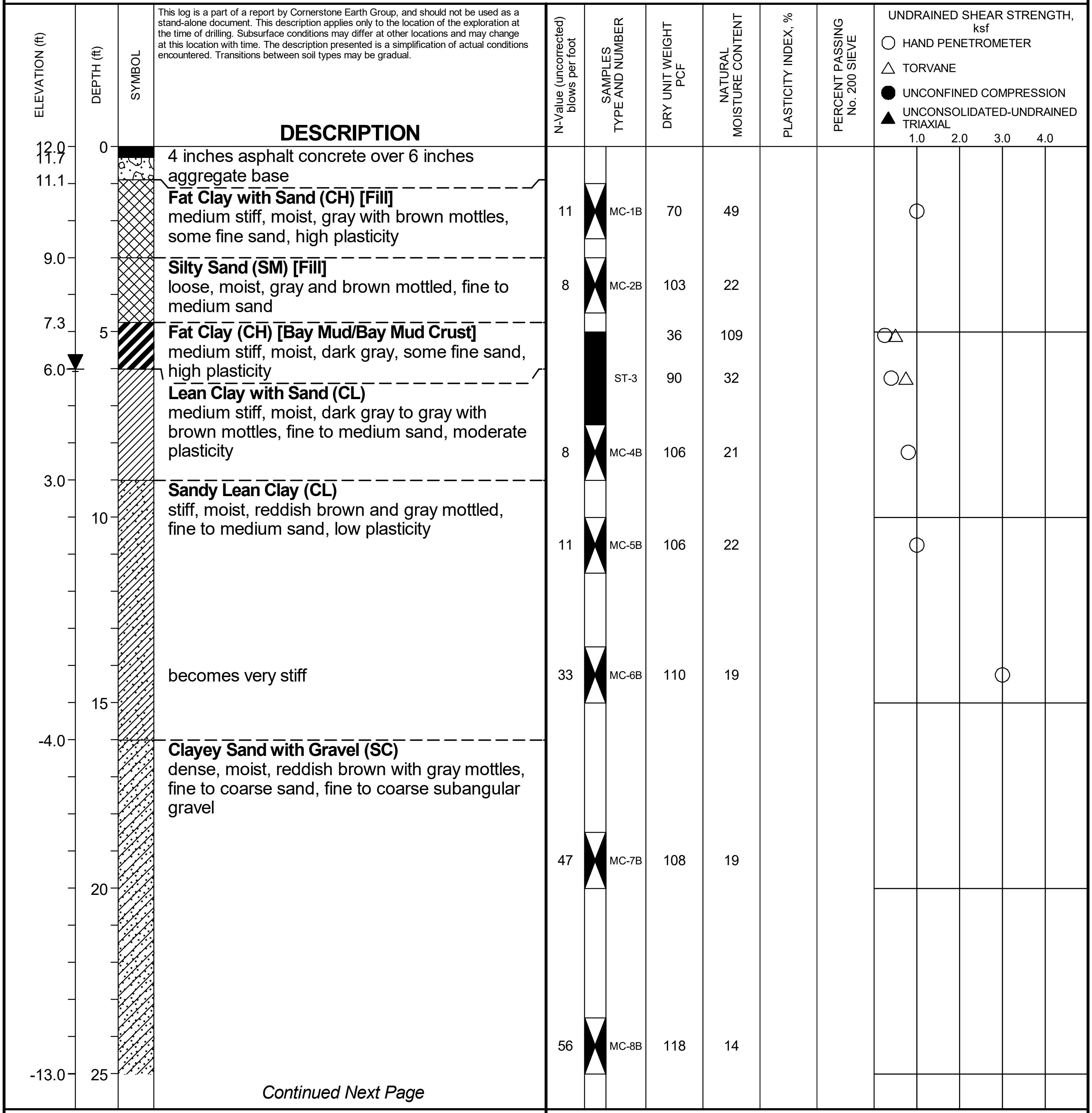
UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL



ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
12.0 11.7	0	4 inches asphalt concrete over 6 inches aggregate base						
11.1		**Fat Clay with Sand (CH) [Fill]** medium stiff, moist, gray with brown mottles, some fine sand, high plasticity	11	MC-1B	70	49		
9.0		**Silty Sand (SM) [Fill]** loose, moist, gray and brown mottled, fine to medium sand	8	MC-2B	103	22		
7.3	5	**Fat Clay (CH) [Bay Mud/Bay Mud Crust]** medium stiff, moist, dark gray, some fine sand, high plasticity			36	109		
6.0 ▼		**Lean Clay with Sand (CL)** medium stiff, moist, dark gray to gray with brown mottles, fine to medium sand, moderate plasticity		ST-3	90	32		
			8	MC-4B	106	21		
3.0	10	**Sandy Lean Clay (CL)** stiff, moist, reddish brown and gray mottled, fine to medium sand, low plasticity	11	MC-5B	106	22		
		becomes very stiff	33	MC-6B	110	19		
-4.0	15	**Clayey Sand with Gravel (SC)** dense, moist, reddish brown with gray mottles, fine to coarse sand, fine to coarse subangular gravel						
	20		47	MC-7B	108	19		
			56	MC-8B	118	14		
-13.0	25							

Continued Next Page

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-3

PAGE 2 OF 2

PROJECT NAME 4th and Irwin Street

PROJECT NUMBER 1395-4-4

PROJECT LOCATION San Rafael, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

UNDRAINED SHEAR STRENGTH, ksf
- ○ HAND PENETROMETER
- △ TORVANE
- ● UNCONFINED COMPRESSION
- ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf 1.0 2.0 3.0 4.0
-13.0	25									
-14.0		▨	**Lean Clay with Sand (CL)** hard, moist, reddish brown and gray mottled, fine sand, moderate plasticity							
				45	MC-9B	102	23			○
-18.0	30		Bottom of Boring at 30.0 feet.							
	35									
	40									
	45									
	50									

Cornerstone Earth Group

Middle Earth
GEO TESTING INC.

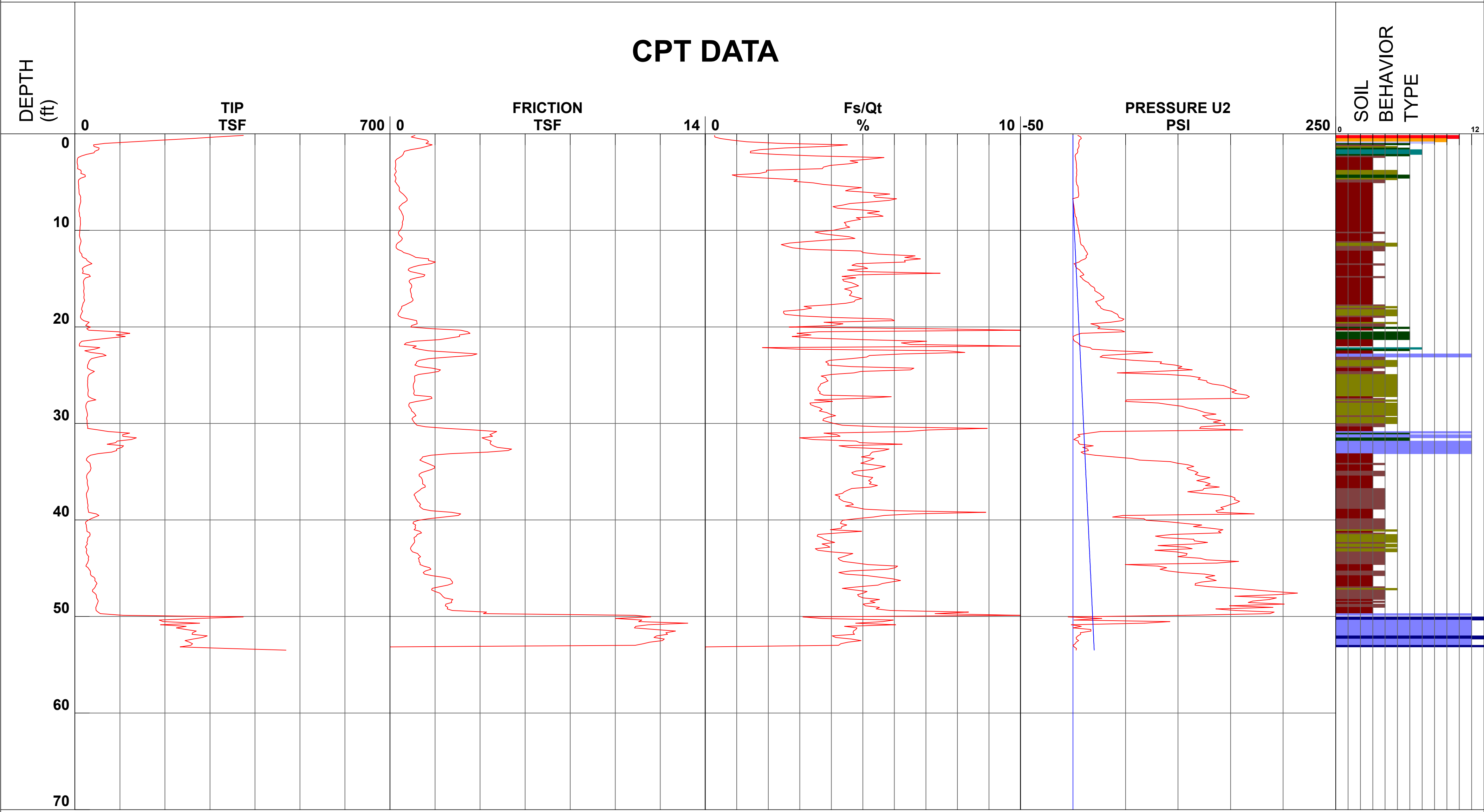
Project 4th and Irwin Preliminary GI
Job Number 1395-4-2
Hole Number CPT-01
EST GW Depth During Test 7.00 ft

Operator JM-FA
Cone Number DDG1596
Date and Time 12/18/2023 2:09:23 PM

Filename SDF(590).cpt
GPS
Maximum Depth 53.48 ft

Net Area Ratio .8

CPT DATA



⬜ 1 - sensitive fine grained
⬛ 2 - organic material
🟥 3 - clay

🟫 4 - silty clay to clay
🟨 5 - clayey silt to silty clay
🟩 6 - sandy silt to clayey silt

🟩 7 - silty sand to sandy silt
🟪 8 - sand to silty sand
🟧 9 - sand

🟥 10 - gravelly sand to sand
🟪 11 - very stiff fine grained (*)
🟦 12 - sand to clayey sand (*)

Cone Size 15cm²

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

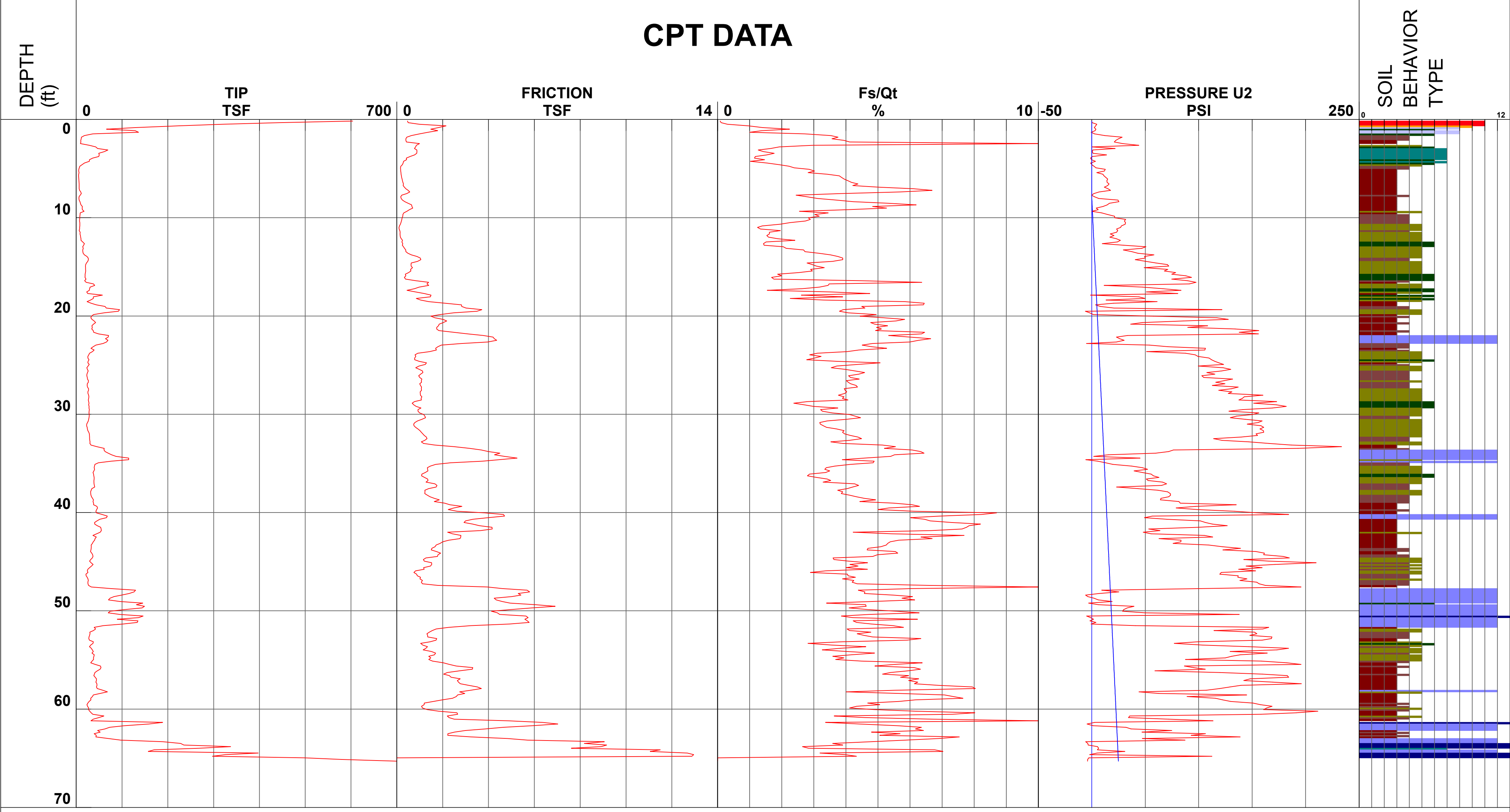
Middle Earth
Geo Testing Inc.

Cornerstone Earth Group

Project	4th and Irwin Preliminary GI	Operator	JM-FA	Filename	SDF(588).cpt
Job Number	1395-4-2	Cone Number	DDG1596	GPS	
Hole Number	CPT-02	Date and Time	12/18/2023 9:58:41 AM	Maximum Depth	65.29 ft
EST GW Depth During Test		8.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm²

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

Middle Earth
Geo Testing Inc.

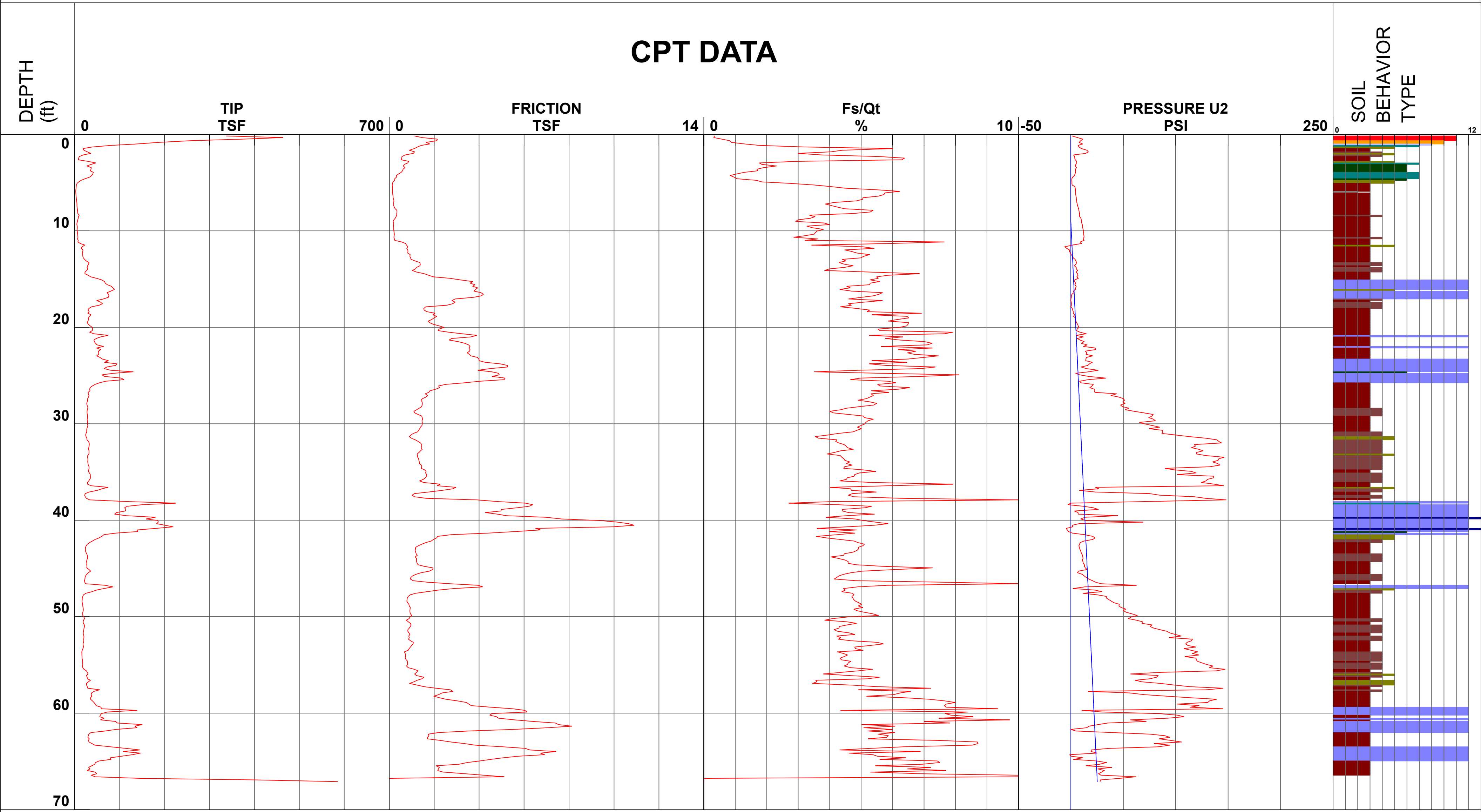
Cornerstone Earth Group

Project 4th and Irwin Preliminary GI
Job Number 1395-4-2
Hole Number CPT-03
EST GW Depth During Test 9.00 ft

Operator JM-FA
Cone Number DDG1596
Date and Time 12/18/2023 12:05:03 PM

Filename SDF(589).cpt
GPS
Maximum Depth 67.09 ft

Net Area Ratio .8



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

Cone Size 15cm²

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

APPENDIX B: LABORATORY TEST PROGRAM

The laboratory testing program was performed to evaluate the physical and mechanical properties of the soils retrieved from the site to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on 35 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 32 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on five samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

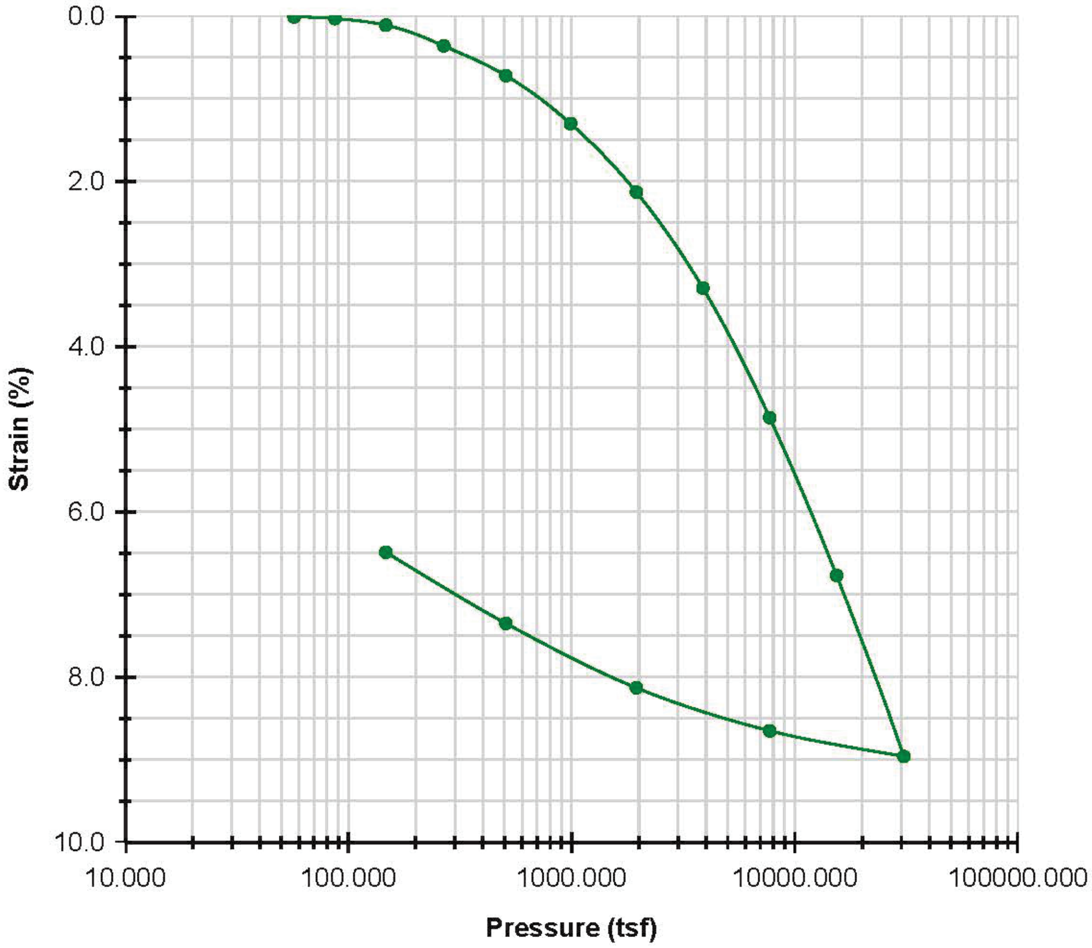
Consolidation: Two consolidation tests (ASTM D2435) were performed on relatively undisturbed samples of the subsurface clayey soils to assist in evaluating the compressibility property of this soil. Results of the consolidation tests are presented graphically in this appendix.

Triaxial Shear Strength: Two unconsolidated, undrained triaxial strength tests (ASTM D2850) were performed on relatively undisturbed samples of the subsurface clayey soils to assist in evaluating the strength of these soils. Results of the triaxial strength tests are presented graphically on the boring logs and in this appendix.

Consolidation Test ASTM D2435

Boring: EB-1 **Sample:** 5 **Depth:** 11.2’

Description: Lean Clay with Sand (CL)



	BEFORE	AFTER
Moisture (%)	20.4	18.1
Dry Density (pcf)	106.5	113.8
Saturation (%)	93.3	100.0
Void Ratio	0.59	0.49

●— (A) Stress Strain Curve

CORNERSTONE
EARTH GROUP

Strain-Log Curve - EB-1 @ 11.2’

523 & 535 4th Street and
912, 914, and 930 Irwin Street
San Rafael, CA

Project Number: 1395-4-4

Figure Number: Figure B1

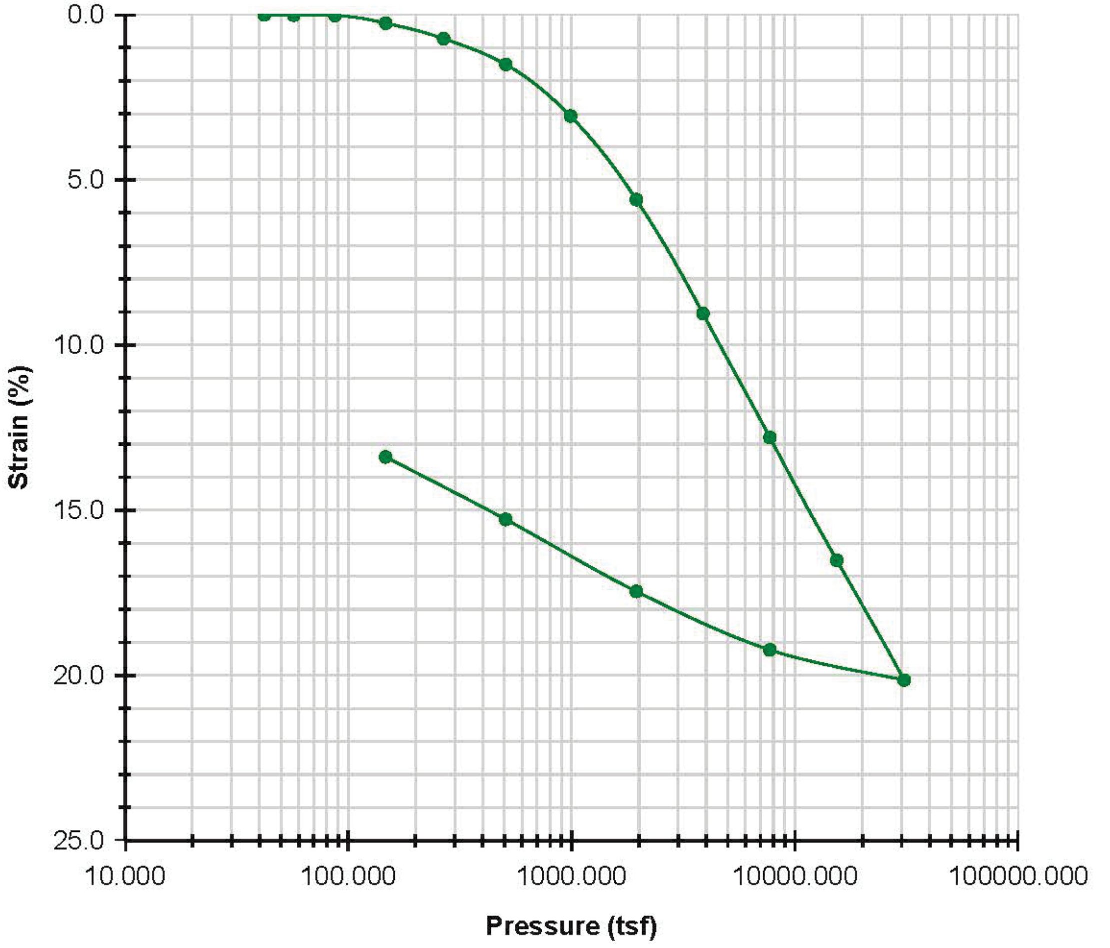
Date: August 2024

Drawn By: FLL

Consolidation Test ASTM D2435

Boring: EB-3 **Sample:** 3 **Depth:** 7.0’

Description: Lean Clay with Sand (CL)



	BEFORE	AFTER
Moisture (%)	31.6	24.3
Dry Density (pcf)	90.0	102.3
Saturation (%)	97.1	100.0
Void Ratio	0.89	0.66

CORNERSTONE EARTH GROUP

Strain-Log Curve - EB-3 @ 7.0’

523 & 535 4th Street and
912, 914, and 930 Irwin Street
San Rafael, CA

Project Number 1395-4-4

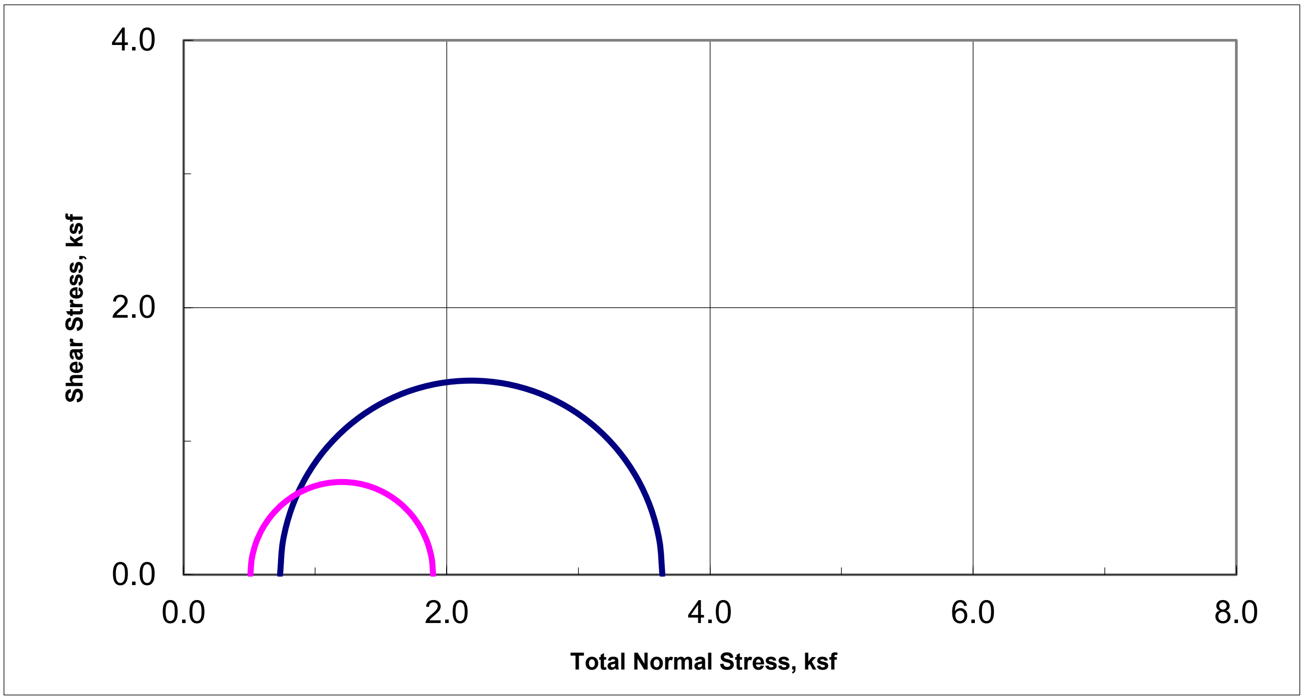
Figure Number Figure B2

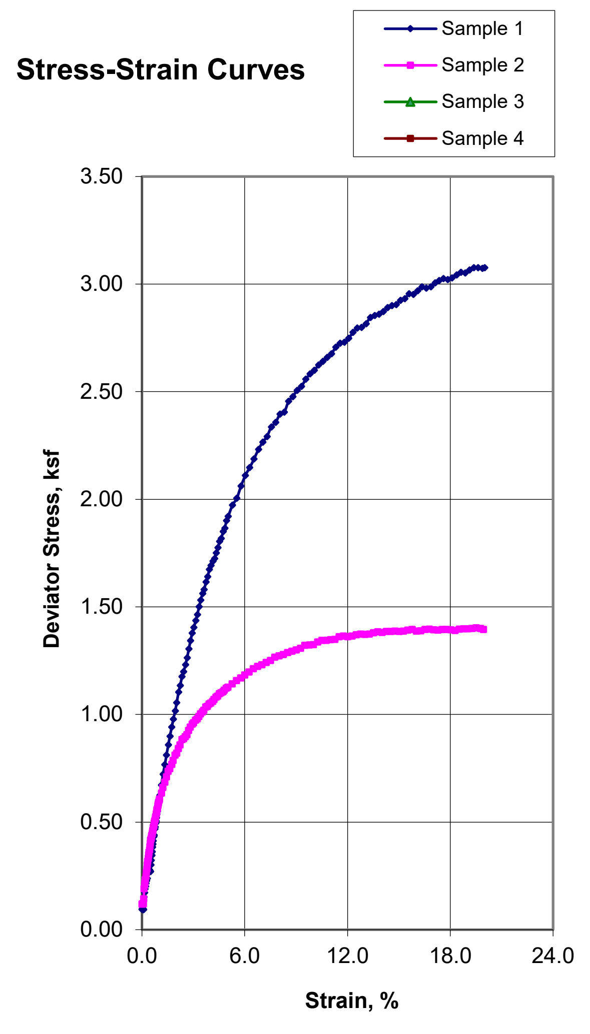
Date August 2024

Drawn By FLL

Unconsolidated-Undrained Triaxial Test

ASTM D2850





Sample Data				
	1	2	3	4
Moisture %	20.4	26.1		
Dry Den,pcf	108.1	98.8		
Void Ratio	0.560	0.706		
Saturation %	98.2	99.8		
Height in	4.97	4.98		
Diameter in	2.41	2.42		
Cell psi	5.1	3.5		
Strain %	15.00	15.00		
Deviator, ksf	2.905	1.388		
Rate %/min	1.00	0.99		
in/min	0.050	0.049		
Job No.:	**640-1560**			
Client:	**Cornerstone Earth Group**			
Project:	**1395-4-4**			
Boring:	EB-1	EB-2		
Sample:	4B	3B		
Depth ft:	8.5	6.0		

Visual Soil Description	
Sample #	
1	Gray Sandy CLAY some Gravel
2	Dark Gray Sandy CLAY some Gravel
3	
4	

Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

APPENDIX C: ENVIRONMENTAL EXPLORATION LOGS

BORING NUMBER SV-1

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME 4th Street and Irwin Street Residential
PROJECT NUMBER 1395-4-2
PROJECT LOCATION San Rafael, CA
DATE STARTED 12/19/23 **DATE COMPLETED** 12/19/23
GROUND ELEVATION ____ **BORING DEPTH** 2 ft.
DRILLING CONTRACTOR Penecore
LATITUDE ____ **LONGITUDE** ____
DRILLING METHOD Geoprobe 7822DT
GROUND WATER LEVELS:
LOGGED BY SQN
▽ **AT TIME OF DRILLING** Not Encountered
NOTES ____
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0.0		asphalt concrete over aggregate base							
			Well Graded Gravel with Sand (GW) [Fill] moist, gray, possible crushed concrete							
			Lean Clay (CL) moist, bluish gray with brown mottles							
			Bottom of Boring at 2.0 feet.							
	2.5									
	5.0									

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 1/10/24 06:50 - P:\DRAFTING\GINT FILES\1395-4-1 4TH AND IRWIN GE SV.GPJ

BORING NUMBER SV-2

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME 4th Street and Irwin Street Residential

PROJECT NUMBER 1395-4-2

PROJECT LOCATION San Rafael, CA

DATE STARTED 12/19/23 **DATE COMPLETED** 12/19/23

GROUND ELEVATION **BORING DEPTH** 1.25 ft.

DRILLING CONTRACTOR Penecore

LATITUDE **LONGITUDE**

DRILLING METHOD Portable Power Auger

GROUND WATER LEVELS:

LOGGED BY SQN

▽ **AT TIME OF DRILLING** Not Encountered

NOTES

▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf 1.0 2.0 3.0 4.0
	0.0		asphalt concrete over aggregate base							
			Well Graded Gravel with Sand (GW) [Fill] moist, gray, possible crushed concrete							
			Lean Clay (CL) moist, dark gray							
			Bottom of Boring at 1.3 feet.							
	2.5									
	5.0									

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 1/10/24 06:50 - P:\DRAFTING\GINT FILES\1395-4-1 4TH AND IRWIN GE SV.GPJ

BORING NUMBER SV-3

CORNERSTONE EARTH GROUP

PROJECT NAME 4th Street and Irwin Street Residential
PROJECT NUMBER 1395-4-2
PROJECT LOCATION San Rafael, CA
DATE STARTED 12/19/23 **DATE COMPLETED** 12/19/23
GROUND ELEVATION **BORING DEPTH** 2.5 ft.
DRILLING CONTRACTOR Penecore
LATITUDE **LONGITUDE**
DRILLING METHOD Geoprobe 7822DT
GROUND WATER LEVELS:
LOGGED BY SQN
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
	0.0	■	asphalt concrete over aggregate base							
		▨	**Clayey Sand (SC) [Fill]** moist, dark gray, some fine subrounded gravel							
		▨	**Fat Clay (CH) [Fill]** moist, bluish gray							
	2.5		Bottom of Boring at 2.5 feet.							
	5.0									

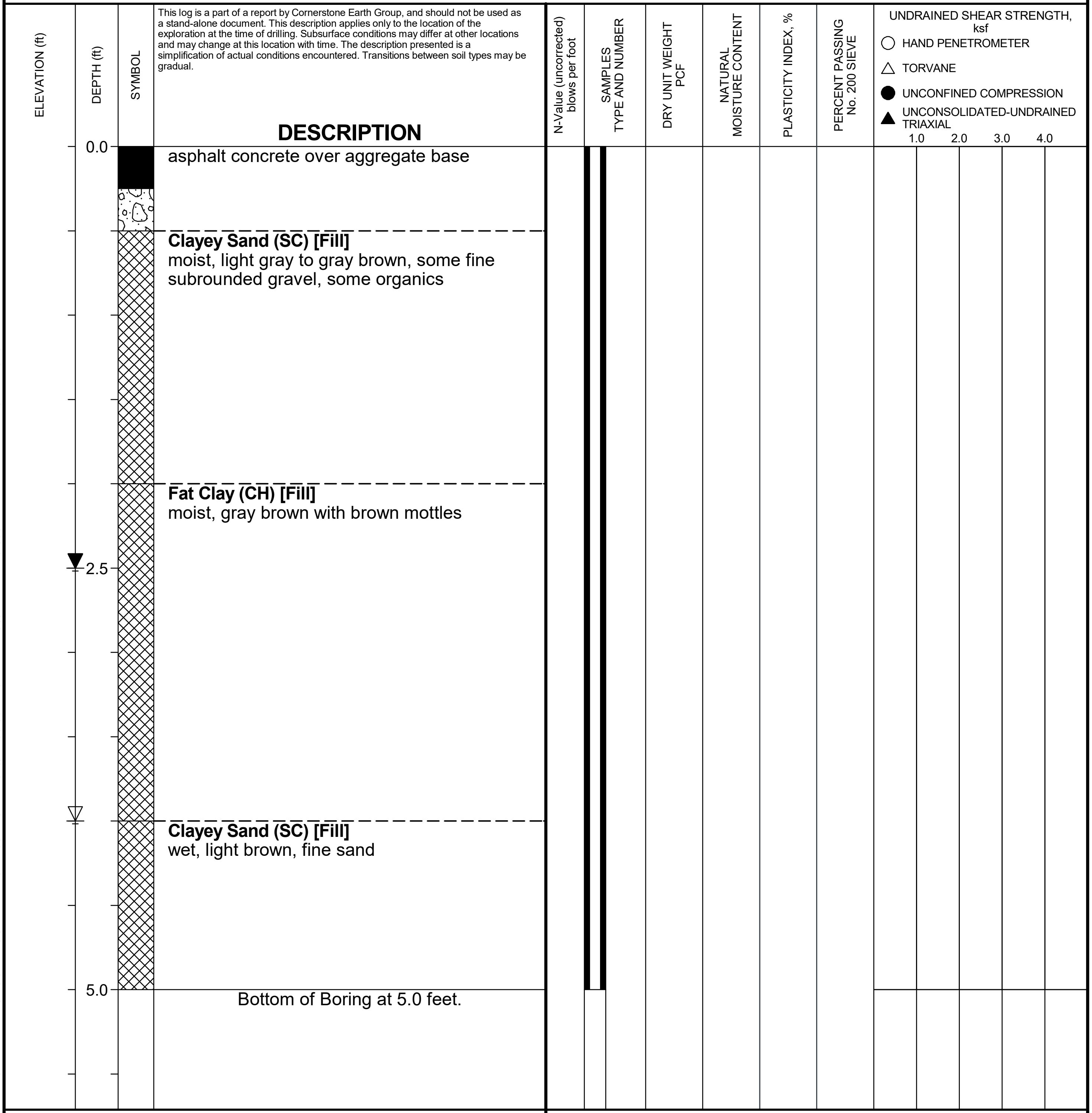
BORING NUMBER SV-4

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME 4th Street and Irwin Street Residential
PROJECT NUMBER 1395-4-2
PROJECT LOCATION San Rafael, CA
DATE STARTED 12/19/23 **DATE COMPLETED** 12/19/23
GROUND ELEVATION ______ **BORING DEPTH** 5 ft.
DRILLING CONTRACTOR Penecore
LATITUDE ______ **LONGITUDE** ______
DRILLING METHOD Geoprobe 7822DT
GROUND WATER LEVELS:
LOGGED BY SQN
▽ **AT TIME OF DRILLING** 4 ft.
NOTES ______
▼ **AT END OF DRILLING** 2.5 ft.

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.



asphalt concrete over aggregate base

Clayey Sand (SC) [Fill]
moist, light gray to gray brown, some fine subrounded gravel, some organics

Fat Clay (CH) [Fill]
moist, gray brown with brown mottles

Clayey Sand (SC) [Fill]
wet, light brown, fine sand

Bottom of Boring at 5.0 feet.

BORING NUMBER SV-5

PAGE 1 OF 1

PROJECT NAME 4th Street and Irwin Street Residential

PROJECT NUMBER 1395-4-2

PROJECT LOCATION San Rafael, CA

DATE STARTED 12/19/23 **DATE COMPLETED** 12/19/23

GROUND ELEVATION **BORING DEPTH** 1.25 ft.

DRILLING CONTRACTOR Penecore

LATITUDE **LONGITUDE**

DRILLING METHOD Portable Power Auger

GROUND WATER LEVELS:

LOGGED BY SQN

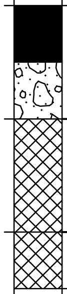
∇ **AT TIME OF DRILLING** Not Encountered

NOTES

▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0.0		asphalt concrete over aggregate base							
			Well Graded Gravel with Sand (GW) [Fill] moist, gray, possible crushed concrete							
			Lean Clay (CL) [Fill] moist, dark gray							
			Bottom of Boring at 1.3 feet.							
	2.5									
	5.0									



UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

1.0 2.0 3.0 4.0

BORING NUMBER SV-6

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME 4th Street and Irwin Street Residential
PROJECT NUMBER 1395-4-2
PROJECT LOCATION San Rafael, CA
DATE STARTED 12/19/23 **DATE COMPLETED** 12/19/23
GROUND ELEVATION **BORING DEPTH** 2 ft.
DRILLING CONTRACTOR Penecore
LATITUDE **LONGITUDE**
DRILLING METHOD Geoprobe 7822DT
GROUND WATER LEVELS:
LOGGED BY SQN
▽ **AT TIME OF DRILLING** 1.2 ft.
NOTES
▼ **AT END OF DRILLING** 1.2 ft.

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
	0.0		asphalt concrete over aggregate base							
▼			**Clayey Sand (SC) [Fill]** moist, dark gray, some fine subrounded gravel							
			Bottom of Boring at 2.0 feet.							
	2.5									
	5.0									

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 1/10/24 06:50 - P:\DRAFTING\GINT FILES\1395-4-1 4TH AND IRWIN GE SV.GPJ

BORING NUMBER EB-1

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME 523 4th Street and 930 Irwin Street Residential
PROJECT NUMBER 1395-4-2
PROJECT LOCATION San Rafael, CA
DATE STARTED 2/14/24 **DATE COMPLETED** 2/14/24
GROUND ELEVATION **BORING DEPTH** 10 ft.
DRILLING CONTRACTOR Penecore
LATITUDE **LONGITUDE**
DRILLING METHOD Geoprobe 420M, Dolly Rig
GROUND WATER LEVELS:
LOGGED BY SQN
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	Sample Type and Interval	Sample Submitted for Laboratory Analysis	Percent Recovery (%)	OVM Reading (ppm)	Odors or Discoloration	Notes
	0.0		asphalt concrete over aggregate base							
			Clayey Sand (SC) [Fill] moist, light gray, fine sand							
								347.5		
	2.5						40	356.8	Petroleum Odor	
			becomes wet							
								52.8		
	5.0		**Fat Clay (CH) [Bay mud]** moist, dark gray							
								237.9		
								25.5		
	7.5						80		Petroleum Odor	
			Fat Clay (CL-CH) moist, light gray to light brown, some organics					22.1		
								23.9		
	10.0		Bottom of Boring at 10.0 feet.							

CORNERSTONE EARTH GROUP

BORING NUMBER EB-2

PAGE 1 OF 1

PROJECT NAME 523 4th Street and 930 Irwin Street Residential

PROJECT NUMBER 1395-4-2

PROJECT LOCATION San Rafael, CA

DATE STARTED 2/14/24 **DATE COMPLETED** 2/14/24

GROUND ELEVATION **BORING DEPTH** 10 ft.

DRILLING CONTRACTOR Penecore

LATITUDE **LONGITUDE**

DRILLING METHOD Geoprobe 420M, Dolly Rig

GROUND WATER LEVELS:

LOGGED BY SQN

∇ **AT TIME OF DRILLING** Not Encountered

NOTES

▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	Sample Type and Interval	Sample Submitted for Laboratory Analysis	Percent Recovery (%)	OVM Reading (ppm)	Odors or Discoloration	Notes
	0.0	asphalt concrete over aggregate base							
		Well Graded Sand with Gravel (SW) [Fill] moist, dark gray							
							3500		
		Clayey Sand (SC) [Fill] moist, dark gray, fine to coarse sand							
	2.5					40	650	Strong Petroleum Odor	
							950		
	5.0	**Fat Clay (CH) [Bay mud]** moist, dark gray to light gray							
							5700		
							6810		
	7.5					98		Strong Petroleum Odor	
		Fat Clay (CL-CH) moist, light gray to light brown, some organics					7022		
							116.3		
	10.0	Bottom of Boring at 10.0 feet.							

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-3

PAGE 1 OF 1

PROJECT NAME 523 4th Street and 930 Irwin Street Residential
PROJECT NUMBER 1395-4-2
PROJECT LOCATION San Rafael, CA
DATE STARTED 2/14/24 **DATE COMPLETED** 2/14/24
GROUND ELEVATION **BORING DEPTH** 10 ft.
DRILLING CONTRACTOR Penecore
LATITUDE **LONGITUDE**
DRILLING METHOD Geoprobe 420M, Dolly Rig
GROUND WATER LEVELS:
LOGGED BY SQN
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	Sample Type and Interval	Sample Submitted for Laboratory Analysis	Percent Recovery (%)	OVM Reading (ppm)	Odors or Discoloration	Notes
	0.0		asphalt concrete over aggregate base							
			Lean Clay (CL) [Fill] moist, light gray, some subrounded gravel					12.7		
			Clayey Sand (SC) [Fill] moist, light gray brown to dark gray, fine to coarse sand							
	2.5						50	5.1	Slight Petroleum Odor	
								6.9		
	5.0		**Fat Clay (CH) [Bay mud]** moist, dark gray							
								42.1		
	7.5						80	6.8	Slight Petroleum Odor	
			Fat Clay (CL-CH) moist, light gray to light brown, some fine sand							
								6.6		
	10.0		Bottom of Boring at 10.0 feet.							

BORING NUMBER EB-4

PAGE 1 OF 1

CORNERSTONE
EARTH GROUP

PROJECT NAME 523 4th Street and 930 Irwin Street Residential
PROJECT NUMBER 1395-4-2
PROJECT LOCATION San Rafael, CA

DATE STARTED 2/14/24 **DATE COMPLETED** 2/14/24
GROUND ELEVATION **BORING DEPTH** 10 ft.
DRILLING CONTRACTOR Penecore
LATITUDE **LONGITUDE**
DRILLING METHOD Geoprobe 420M, Dolly Rig
GROUND WATER LEVELS:
LOGGED BY SQN
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	Sample Type and Interval	Sample Submitted for Laboratory Analysis	Percent Recovery (%)	OVM Reading (ppm)	Odors or Discoloration	Notes
	0.0		asphalt concrete over aggregate base							
			Clayey Sand (SC) [Fill] moist, light brown to gray brown, fine to coarse sand, some subrounded gravel							
								26.2		
	2.5						50	17.0	Slight Petroleum Odor	
								10.4		
	5.0		**Fat Clay (CH) [Bay mud]** moist, dark gray							
								16.5		
	7.5						60	11.5	Slight Petroleum Odor	
			Fat Clay (CL-CH) moist, light gray to light brown, some subrounded gravel, some organics							
								118.1		
	10.0		Bottom of Boring at 10.0 feet.							

BORING NUMBER EB-5

CORNERSTONE EARTH GROUP

PROJECT NAME 523 4th Street and 930 Irwin Street Residential
PROJECT NUMBER 1395-4-2
PROJECT LOCATION San Rafael, CA
DATE STARTED 2/14/24 **DATE COMPLETED** 2/14/24
GROUND ELEVATION **BORING DEPTH** 10 ft.
DRILLING CONTRACTOR Penecore
LATITUDE **LONGITUDE**
DRILLING METHOD Geoprobe 420M, Dolly Rig
GROUND WATER LEVELS:
LOGGED BY SQN
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	Sample Type and Interval	Sample Submitted for Laboratory Analysis	Percent Recovery (%)	OVM Reading (ppm)	Odors or Discoloration	Notes
	0.0		asphalt concrete over aggregate base							
			Clayey Sand (SC) [Fill] moist, dark gray							
								8.0		
	2.5						20		Slight Petroleum Odor	
								12.3		
	5.0		**Fat Clay (CH) [Bay mud]** moist, dark gray to light gray with light brown mottles							
								37.2		
	7.5						50	33.5	Slight Petroleum Odor	
			Fat Clay (CL-CH) moist, light gray to light brown with reddish brown mottles							
								21.6		
	10.0		Bottom of Boring at 10.0 feet.							

BORING NUMBER EB-6

CORNERSTONE EARTH GROUP

PROJECT NAME 523 4th Street and 930 Irwin Street Residential
PROJECT NUMBER 1395-4-2
PROJECT LOCATION San Rafael, CA
DATE STARTED 2/14/24 **DATE COMPLETED** 2/14/24
GROUND ELEVATION ____ **BORING DEPTH** 10 ft.
DRILLING CONTRACTOR Penecore
LATITUDE ____ **LONGITUDE** ____
DRILLING METHOD Geoprobe 7822DT
GROUND WATER LEVELS:
LOGGED BY SQN
▽ **AT TIME OF DRILLING** Not Encountered
NOTES ____
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	Sample Type and Interval	Sample Submitted for Laboratory Analysis	Percent Recovery (%)	OVM Reading (ppm)	Odors or Discoloration	Notes
	0.0		asphalt concrete over aggregate base							
			Clayey Sand (SC) [Fill] moist, dark brown, some subrounded gravel, some organics							
								10.1		
	2.5						65	12.7		
			Fat Clay (CH) [Bay mud] moist, dark gray, some organics					7.1		
	5.0									
								15.3		
	7.5						40	8.1		
			Fat Clay (CL-CH) moist, light gray to light brown with reddish brown mottles							
	10.0		Bottom of Boring at 10.0 feet.							

BORING NUMBER EB-7

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME 523 4th Street and 930 Irwin Street Residential
PROJECT NUMBER 1395-4-2
PROJECT LOCATION San Rafael, CA
DATE STARTED 2/14/24 **DATE COMPLETED** 2/14/24
GROUND ELEVATION **BORING DEPTH** 10 ft.
DRILLING CONTRACTOR Penecore
LATITUDE **LONGITUDE**
DRILLING METHOD Geoprobe 7822DT
GROUND WATER LEVELS:
LOGGED BY SQN
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	Sample Type and Interval	Sample Submitted for Laboratory Analysis	Percent Recovery (%)	OVM Reading (ppm)	Odors or Discoloration	Notes
	0.0		asphalt concrete over aggregate base							
			Well Graded Sand with Gravel (SW) [Fill] moist, gray, subrounded gravel					15.3		
			Lean Clay (CL) [Fill] moist, bluish gray with brown mottles							
	2.5						50	17.7		
			Fat Clay (CH) [Bay mud] moist, bluish gray to dark gray					7.0		
	5.0									
								17.1		
	7.5						50	19.9		
								25.4		
	10.0		Bottom of Boring at 10.0 feet.							

BORING NUMBER EB-8

PAGE 1 OF 1

CORNERSTONE
EARTH GROUP

PROJECT NAME 523 4th Street and 930 Irwin Street Residential
PROJECT NUMBER 1395-4-2
PROJECT LOCATION San Rafael, CA
DATE STARTED 2/14/24 **DATE COMPLETED** 2/14/24
GROUND ELEVATION **BORING DEPTH** 10 ft.
DRILLING CONTRACTOR Penecore
LATITUDE **LONGITUDE**
DRILLING METHOD Geoprobe 7822DT
GROUND WATER LEVELS:
LOGGED BY SQN
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	Sample Type and Interval	Sample Submitted for Laboratory Analysis	Percent Recovery (%)	OVM Reading (ppm)	Odors or Discoloration	Notes
	0.0		asphalt concrete over aggregate base							
			Well Graded Gravel with Sand (GW) [Fill] moist, gray, possible crushed concrete							
								17.7		
			Lean Clay (CL) [Fill] moist, bluish gray with light gray mottles, some fine subrounded gravel							
								22.8		
	2.5						65			
								15.2		
			Fat Clay (CH) [Bay mud] moist, dark gray					20.2		
	5.0									
								18.9		
	7.5						45	10.3		
			Fat Clay (CL-CH) moist, light gray to light brown and bluish gray							
	10.0		Bottom of Boring at 10.0 feet.							