CORNERSTONE
EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**Block 21**
Location	**East 3rd Avenue and South Delaware Street** **San Mateo, California**
Client	**Windy Hill Property Ventures**
Client Address	**530 Emerson Street, Suite 150** **Palo Alto, CA**
Project Number	**803-11-1**
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Prepared by **Stephen C. Ohlsen, P.E.**
Project Engineer
Geotechnical Project Manager

REGISTERED PROFESSIONAL ENGINEER
STEPHEN C. OHLSEN
C 90614
CIVIL
★ STATE OF CALIFORNIA ★

Erin L. Steiner, P.E., G.E.
Principal Engineer
Quality Assurance Reviewer

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SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Windy Hill Property Ventures for the Block 21 in San Mateo, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- An architectural plan set titled, "A Planning Application For: Block 21, E 3rd Avenue & S Delaware Street, San Mateo, CA 94401," prepared by ArcTec Inc, dated Prelim Planning Resubmittal, April 21, 2021.

1.1 PROJECT DESCRIPTION

We understand that a new five-story building with two levels of below-grade parking is currently planned for the site. The new building and basement will take up the majority/entirety of the site/city block. The first three levels will consist of open office space with the top two floors (fourth and fifth floor) will primarily consist of studio and one-bedroom apartments with the fourth floor containing a shared office space and two balconies. The two below-grade parking levels will provide 390 new parking stalls for both office and residential use as well as storage and electrical/mechanical rooms. The below-grade parking levels will likely consist of concrete-frame construction while the five above grade levels will likely consist of wood- or steel-frame construction. The planned development will have a footprint of approximately 59,227 feet.

Cuts are anticipated to be on the order of 24 to 28 feet from existing grades to accommodate the two levels of below-grade parking. Structural loads have not been provided; however, we anticipate them to be representative of similar type structures.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated June 29, 2021 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 6 borings drilled on July 21 to 23, 2021 with truck-mounted, hollow-stem auger drilling equipment. The borings were drilled to depths ranging from approximately 40 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 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, and a plasticity Index test. Details regarding our laboratory program are included in Appendix B.

1.5 ENVIRONMENTAL SERVICES

Environmental services were not requested for this project. If environmental concerns are determined to be present during future evaluations, the project environmental consultant should review our geotechnical recommendations for compatibility with the environmental concerns.

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

2.1.1 Regional Geologic Setting

The relatively flat-lying plain along the western edge of the San Francisco Bay is bounded by the Santa Cruz Mountains on the west and the San Francisco Bay to the east. The Coast Ranges is a geomorphic province of California that stretches from the Oregon border nearly to Point Conception. In the San Francisco Bay area, most of the Coast Ranges have developed on a basement of tectonically mixed Cretaceous- and Jurassic-age (70- to 200-million years old) rocks of the Franciscan Complex. Younger sedimentary and volcanic units locally cap these basement rocks. Still younger surficial deposits that reflect geologic conditions of the last million years or so cover most of the Coast Ranges.

Movement on the many splays of the San Andreas Fault system has produced the dominant northwest-oriented structural and topographic trend seen throughout the Coast Ranges today. This trend reflects the boundary between two of the Earth's major tectonic plates: the North American plate to the east and the Pacific plate to the west. The San Andreas Fault system and its major branching faults is about 40 miles wide in the Bay area and extends from the San

Gregorio Fault near the coastline to the Coast Ranges-Central Valley blind thrust at the western edge of the Great Central Valley as shown on the Regional Fault Map, Figure 3. The San Andreas Fault is the dominant structure in the system, nearly spanning the length of California, and capable of producing the highest magnitude earthquakes. Many other subparallel or branch faults within the San Andreas system are equally active and nearly as capable of generating large earthquakes. Right-lateral movement dominates on these faults but an increasingly large amount of thrust faulting resulting from compression across the system is now being identified also.

2.1.2 Local Geology

Roughly half the San Mateo 7.5-Minute Quadrangle and adjacent areas are covered by Quaternary alluvial sediment shed from the northwest-trending Santa Cruz Mountains that occupy the area west of the site (Pampeyan, 1994) as seen on Figure 4, Vicinity Geologic Map. The site is in an area adjacent to the San Francisco Bay where Holocene age (11,000 years or less before present) alluvial fan deposits account for the majority of Quaternary sediment deposited in the area, and is shown as underlain by medium-grained alluvium (Qam) of Holocene age over older alluvium (Qoa) of Pleistocene age.

The Qam unit is described as “unconsolidated to moderately consolidated, moderately sorted fine sand, silt and clayey silt.” The Qam unit is generally less than 20 feet thick, was deposited at the edge of coarse-grained alluviual fans (Qac) and locally interfingers with coarse and fine-grained alluvium (Qaf). It forms much of the flatland alluvial plain along the western edge of the Bay in the San Mateo quadrangle. The Qoa unit is designated as “(Late Pleistocene) older alluvial fan deposits” and is described as “unconsolidated to moderately consolidated gravel, sand and silt.”

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)	Distance (kilometers)
San Andreas	3.4	5.5
Monte Vista-Shannon	9.7	15.6
San Gregorio	10.4	16.8
Hayward (Total Length)	15.0	24.1

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). A summary of pertinent surface changes at and in the near vicinity of the site is as follows:

- 1956: The city block is divided into multiple parcels with the parcels along South Delaware Street appearing to be residentials homes and yards. The parcels along South Claremont Street appear to be commercial properties with commercial buildings and lots. The street layout appears to match today's street layout.
- 1968: The existing ARCO gas station appears to have been built on the corner of East 3rd Ave and South Delaware Street. The residential property on the corner of East 4th Avenue and South Delaware Street appears to have been removed and a commercial building built.
- 1988: No pertinent surface changes are observed.
- 2018: No pertinent surface changes are observed.

3.2 SURFACE DESCRIPTION

The project site encompasses a city block that is bounded by East 3rd Avenue, South Delaware Street, East 4th Avenue, and South Claremont Street. The project site is composed of multiple individual parcels consisting of residential homes, commercial buildings, restaurants, repair shops, and a gas station. The buildings are generally one- to two-stories with no indications of below grade basements. The commercial lots are generally covered with asphalt concrete with some portions covered in Portland cement concrete. Residential parcels generally have gravel in driveway areas with multiple large trees observed. The site is generally level with current city streets and sidewalks, with the paved areas graded to drain to storm drain facilities.

Surface pavement at Boring EB-6 generally consisted of 1-inch of asphalt concrete over 3 inches of aggregate base. Boring EB-3 consisted of 6 inches of Portland cement concrete pavement directly over subgrade. Borings EB-1 and EB-2 were advanced in gravel driveways and consist of 3 inches of gravel over subgrade. Boring EB-4 was being used as a laydown yard for nearby construction and consisted of a layer of ¾ inch gravel over 3 inches of asphalt concrete. Based on visual observations, the existing pavements are in poor shape with moderate alligator cracking.

3.3 SUBSURFACE CONDITIONS

Below the surface pavements and surface grades, our explorations generally encountered interbedded alluvial soils to the maximum depths explored during this investigation. Approximately 1 foot of undocumented fill was encountered below the gravel driveway in Boring EB-4. The undocumented fill consisted of loose clayey sand with gravel. In general, the borings encountered a clayey surface layer consisting of stiff to hard lean clay with sand to sandy lean clay to depths of 2 to 12 feet from existing site grades. Below the clayey layer, a sandy/gravelly layer consisting of medium dense to dense clayey sand with gravel, well graded sand with clay and gravel, and well graded gravel with clay and sand was observed to depths of 28½ to 37 feet. An interbedded two-foot layer of medium stiff sandy lean clay was observed in Boring EB-6 at a depth of 24½ feet as well as a three-foot layer of very stiff sandy lean clay in Boring EB-3 at a depth of 17 feet below existing site grades. Below the deeper sandy/gravelly layer, our borings generally encountered medium stiff to hard lean clay with sand to sandy lean clay to the maximum explored depth of 60 feet. An interbedded layer of well graded sand with clay and gravel was observed in Boring EB-5 between depths of 52 feet to 56½ feet.

3.3.1 Plasticity/Expansion Potential

We performed one Plasticity Index (PI) test on a representative sample. The test result was used to evaluate expansion potential of surficial soils, and the plasticity of the fines in potentially liquefiable layers. The result of the surficial PI test indicated a PI of 30, indicating high expansion potential to wetting and drying cycles. The result of the PI test in the potentially liquefiable layers indicated a PI of 24.

3.3.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 24 to 28 feet of the soil profile range from 15 to 26 percent moisture. In our opinion, we estimated this corresponds to about 1 to 12 percent above the estimated laboratory optimum moisture content at the time of our exploration.

3.4 GROUNDWATER

Groundwater was encountered in all of our explorations EB-1 to EB-6 at depths ranging from 19½ to 22 feet below current grades. In addition, groundwater was encountered in our explorations across the street at 3rd and Railroad Avenue at depths ranging from approximately 16 to 19 feet below current grades and a monitoring well at 4th and Railroad indicated groundwater at about 17 feet below the existing grade in March 2011. 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.

Published data (CGS, San Mateo 7.5-minute Quadrangle, 2018) indicated that seasonal and/or historical high groundwater levels in the vicinity of the site are on the order of 12 to 13 feet below the ground surface, as seen on Figure 5, Depth to Historic High Groundwater.

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 and our experience in the area, we recommend a design groundwater depth of 12 feet below current grades.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT SURFACE 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 surface 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 peak ground acceleration (PGA_M) was estimated following the ground motion hazard analysis procedure presented in Chapter 16 and 18 and Appendix J of the 2019 California Building Code (CBC) and Chapter 21, Section 21.2 of ASCE 7-16 and Supplement No. 1. For our analysis we used a PGA_M of 0.90g which was determined in accordance with Section 21.5 of ASCE 7-16.

4.3 LIQUEFACTION POTENTIAL

The site is not located within a State-designated Liquefaction Hazard Zone (CGS, San Mateo 7.5-minute Quadrangle, 2018). However, we screened the site for liquefaction during our site exploration by retrieving samples from the site, performing visual classification on sampled materials, and performing various tests to further classify the soil properties.

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 is 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 12 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 laboratory testing on samples retrieved from our borings. SPT “N” values obtained from hollow-stem auger borings were used in borings EB-1, 2, and 4 analyses as a comparison. Typically, SPT “N” values obtained from hollow-stem auger borings are not used in our analyses, as the “N” values obtained are less reliable in sands below ground water.

We evaluated the potential for liquefaction based on the soil conditions encountered in Borings EB-1, 2, and 4. For this analysis, the soil’s CRR is estimated from the in-situ density and strength obtained from field SPT blow counts (“N” value). The “N” values are corrected for effective overburden stresses, taking into consideration both the groundwater level at the time of exploration and the design groundwater level, and stress reduction versus depth factors. For overburden stress correction, C_N, we used the published equation from Idriss and Boulanger (2008). The “N” values are also corrected for fines content, hammer efficiency, boring diameter, rod length, and sampler type (with or without liners). For fines content, we performed washed sieve tests for each sandy layer and used the fines content from the tests in published equations from Idriss and Boulanger (2008). For hammer efficiency, the drill rig used a down-hole slide hammer with an efficiency of approximately 48 to 51 percent. For boring diameter correction, C_B, we used a value of 1.15 based on the relationships published by Skempton (1986) for a 7.87 in (200mm) borehole diameter. For the rod length correction, C_R, we used published relationships for variable rod lengths, varying from 0.95 to 1.0 for the rod lengths used to sample the sand layers below 25 feet in borings EB-1, 2 and 4. The sampler that was used was not designed to have liners, therefore, no correction factor was applied for the sampler type.

4.3.3 Summary

As discussed in the “Subsurface” section above, we primarily encountered stiff to hard clays and medium dense to dense sands below the anticipated excavation depths for the below-grade parking structure. Medium dense clayey sands could potentially be liquefiable; however, we performed additional washed sieve analysis and a plasticity index test to further evaluate the material properties of several potentially liquefiable layers. Based on the results of our additional laboratory testing, as well as our experience in the site vicinity, our liquefaction analysis indicates that several layers could potentially experience liquefaction triggering that could result in post-liquefaction total settlement at the bottom of basement ranging up to ¾-inch based on the Yoshimine (2006) method. In our opinion, differential settlements are anticipated to be on the order of ½-inch over a horizontal distance of 50 feet.

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.

Our analysis indicates a low potential for liquefaction at the site. Additionally, there are no open faces within a distance considered susceptible to lateral spreading; therefore, in our opinion, the potential for lateral spreading to affect the site is low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. As the soils encountered at the site were predominantly stiff to hard clays and medium dense to dense sands, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low.

4.6 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 1-mile inland from the San Francisco Bay shoreline and is approximately 24 to 26 feet above mean sea level (Google Earth, WGS84). According to published maps (CGS, County of San Mateo Tsunami Hazard Area Map, 2021), the site is not within a tsunami inundation zone. Therefore, the potential for inundation due to tsunami or seiche is considered low.

4.7 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone X, an area of minimal flood hazard. We recommend the project civil engineer be retained to confirm this information.

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 concern with brief outlines of our recommendations follow the listed concerns.

- Potential for liquefaction-induced settlements
- Potential for static settlement
- Depth to groundwater
- Proximity to existing improvements and structures
- Construction dewatering induced settlements
- Differential movement at on-grade to on-structure transitions
- Potential presence of moderately to highly expansive soils
- Redevelopment considerations

5.1.1 Potential for Liquefaction-Induced Settlements

As discussed, our liquefaction analysis indicates that there is a potential for liquefaction of localized sand layers below the bottom of proposed basement during a significant seismic event. Although the potential for liquefied sands to vent to the ground surface through cracks in the surficial soils is low, our analysis indicates that liquefaction-induced settlement on the order of ¾-inches could occur, resulting in differential settlement up to ½-inch over a horizontal distance of 50 feet. Foundations should be designed to tolerate the anticipated total and differential settlements. Detailed foundation recommendations are presented in the "Foundations" section.

5.1.2 Potential for Static Settlement

In our opinion, based on the site and subsurface conditions, the proposed structure may be supported on a mat foundation. As structural loading was not provided, we estimated loads based on similar type of structures. Static settlements for the mat foundation are anticipated to be 1-inch with differential on the order of ½-inch between from the center of the mat to the edge of mat. Foundations should be designed to tolerate the anticipated total and differential settlements. Detailed foundation recommendations are presented in the "Foundations" section.

5.1.3 Depth to Groundwater

As discussed above, we recommend a high groundwater level of 12 feet below existing grades be used for design of the below-grade parking garage. We understand that the below-grade parking garage will be two levels, with cuts on the order of up to 24 to 28 feet. Based on the current and design groundwater depths, the garage slab-on-grade, foundations, and garage walls would need to be designed for hydrostatic uplift pressures and increased lateral wall earth pressures for the depth below the design groundwater. We recommend waterproofing the below-grade walls and slab, and designing the parking structure slab foundation and garage walls, including construction joints, to resist hydrostatic pressure. Detailed recommendations addressing this concern, including dewatering and shoring, are presented in the "Earthwork" section of this report.

5.1.4 Proximity to Existing Improvements and Structures

Support of the adjacent improvements, such as streets, sidewalks, and utilities without distress should be the contractor's responsibility. We recommend that the contractor implement a monitoring program to determine the effects of the construction on nearby improvements, including the monitoring of cracking and vertical movement of adjacent structures, and nearby streets, sidewalks, utilities, and other improvements. In critical areas, we recommend that inclinometers or other instrumentation be installed as part of the shoring system to closely monitor lateral movement. Detailed shoring recommendations are also provided in this report.

5.1.5 Construction Dewatering Induced Settlements

We understand that two levels of below-grade parking are currently planned for the site resulting in basement excavation cuts extending below the seasonal and current groundwater depths. Dewatering wells will be needed to lower the groundwater table to at least 5 feet below bottom of the mass excavation. We evaluated the potential settlement of the surrounding ground for a two-level below-grade parking garage. Our analysis assumed a dewatering depth up to 35 feet at wellpoints, resulting in about ¾-inch of settlement near well points, decreasing at greater distances from well points. If this settlement is considered tolerable to the adjacent structure as well as City improvements, dewatering should be feasible. If settlement due to dewatering is not desired, the shoring can be designed as undrained cutoff walls, with secant soil-cement columns or similar.

5.1.6 Differential Movement from On-grade to On-Structure Transitions

As mentioned above, the plans indicate that the at-grade building will be entirely supported by the below-grade parking structure; however, we anticipate that some improvements will transition from on-grade support to overlying the structure. Where the depth of soil cover overlying the basement roof is thin or where basement walls extend to within inches of finished grade, these transition areas typically experience increased differential movement due to a variety of causes, including difficulty in achieving compaction of retaining wall backfill closest to the wall. We recommend construction and expansion joints be dowelled at this transition. Consideration should be given to structurally spanning at entrances into the garage and doweling in the hardscape. If surface improvements are included that are highly sensitive to differential movement, additional measures may be necessary. We understand hinge slabs may be considered for garage entrances. If surface improvements are included that are highly sensitive to differential movement, additional measures may be necessary. We also recommend that retaining wall backfill be compacted to 95 percent where surface improvements are planned (see “Retaining Wall” section).

5.1.7 Potential Presence of Moderately to Highly Expansive Soils

As discussed, highly expansive surficial soils were encountered in the surficial soils that blanket the site. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. Due to the expansive soils, we recommend that at-grade flatwork should be supported by at least 6 inches of inches of non-expansive fill overlying subgrade prepared in accordance with the “Earthwork” recommendations of this report. 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. Grading and foundation recommendations addressing this concern are presented in the “Earthwork” section of this report.

5.1.8 Potential for Cohesionless Sand Layers

As discussed in the “Subsurface” section, several sandy soils were encountered in our explorations with fines contents ranging from 11 to 24 percent passing through the No. 200 sieve in the upper 30 feet. The contractor should consider the following issues during scheduling and evaluation of means and methods:

- Temporary shoring:
    - Potential caving of tie-back excavations – consider casing during drilling
    - Potential sloughing of excavation sidewalls – excavation and trimming of sidewalls may need to be done in limited sections that can be lagged during the same shift where layers of cleaner soils are encountered

- Foundation excavations:
    - Contractor may not be able to cut excavations neat, may need forming
    - Excavation bottoms will likely need to be proof compacted with vibratory equipment prior to placing reinforcing steel to address excavation disturbance

- Below-grade garage subgrade preparation:
    - Construction vehicle and foot traffic will likely disturb the below-grade subgrade during foundation and other construction activities that will occur prior to constructing the mat foundation. The contractor should consider pouring a rat slab to create a working surface.

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 a 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 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.

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 provided written confirmation 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. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Based on our site observations, surficial stripping should extend about 3 to 6 inches below existing grade in vegetated areas.

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 REMOVAL OF EXISTING FILLS

Shallow fills were encountered in one of our borings. Based on the proposed depth of the below-grade parking structure, we anticipate these fills will be removed during excavation.

All fills should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations. Based on review of the samples collected from our borings, it appears that the fill may be reused. 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.

Fills extending into planned pavement and flatwork areas may be left in place provided they are determined to be a low risk for future differential settlement and that the upper 12 to 18 inches of fill below pavement subgrade is re-worked and compacted as discussed in 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

28 feet at the site may be classified as OSHA Site C materials. Recommended soil parameters for temporary shoring are provided in the "Temporary Shoring" section of this report.

Excavations performed during site demolition and fill removal should be sloped at 3:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Excavations extending more than 5 feet below building subgrade and excavations in pavement and flatwork areas should be slope at a 1:1 inclination unless the OSHA soil classification indicates that slope should not exceed 1.5:1.

6.5 BELOW-GRADE EXCAVATIONS

We anticipate temporary shoring will support the planned cuts up to 28 feet. We have provided geotechnical parameters for shoring design in the section below. The choice of shoring method should be left to the contractor's judgment based on experience, economic considerations and adjacent improvements such as utilities, pavements, and foundation loads. Temporary shoring should support adjacent improvements without distress and should be the contractor's responsibility. A pre-condition survey including photographs and installation of monitoring points for existing site improvements should be included in the contractor's scope. We should be provided the opportunity to review the geotechnical parameters of the shoring design prior to implementation; the project structural engineer should be consulted regarding support of adjacent structures.

6.5.1 Temporary Shoring

Based on the site conditions encountered during our investigation, the cuts may be supported by soldier beams and tie-backs, braced excavations, or potentially other methods. Where shoring will extend more than about 10 feet, restrained shoring will most likely be required to limit detrimental lateral deflections and settlement behind the shoring. In addition to soil earth pressures, the shoring system will need to support adjacent loads such as construction vehicles and incidental loading, existing structure foundation loads, and street loading. We recommend that heavy construction loads (cranes, etc.) and material stockpiles be kept at least 15 feet behind the shoring. Where this loading cannot be set back, the shoring will need to be designed to support the loading. The shoring designer should provide for timely and uniform mobilization of soil pressures that will not result in excessive lateral deflections. Minimum suggested geotechnical parameters for shoring design are provided in the table below.

Table 2: Suggested Temporary Shoring Design Parameters

Design Parameter	Design Value
Minimum Lateral Wall Surcharge (upper 5 feet)	120 psf
[1]Restrained Wall – Uniform Earth Pressure	25H*
[2]Passive Pressure – Starting at 2 feet below the bottom of the excavation	400 pcf up to 2,000 psf maximum uniform pressure

[1]H equals the height of the excavation; hinge point occurs at ¼H.

[2]Passive pressures are assumed to act over twice the soldier pile diameter

The restrained earth pressure may also be distributed as described in Figure 24 of the *FHWA Circular No. 4 – Ground Anchors and Anchored Systems* (with the hinge points at ¼H and ¾H) provided the total pressure is established from the uniform pressure above.

If shotcrete lagging is used for the shoring facing, the permanent retaining wall drainage materials, as discussed in the “Wall Drainage” section of this report, will need to be installed during temporary shoring construction. At a minimum, 2-foot-wide vertical panels should be placed between soil nails or tiebacks that are spaced at 6-foot centers. For 8-foot centers, 4-foot-wide vertical panels should be provided. A horizontal strip drain connecting the vertical panels should be provided, or pass-through connections should be included for each vertical panel.

We performed our borings with hollow-stem auger drilling equipment and as such were not able to evaluate the potential for caving soils, which can create difficult conditions during soldier beam, or tie-back installation; caving soils can also be problematic during excavation and lagging placement. The contractor is responsible for evaluating excavation difficulties prior to construction. Where relatively clean sands (especially encountered below ground water) or difficult drilling or cobble conditions were encountered during our exploration, pilot holes performed by the contractor may be desired to further evaluate these conditions prior to the finalization of the shoring budget.

In addition to anticipated deflection of the shoring system, other factors such as voids created by soil sloughing, and erosion of granular layers due to perched water conditions can create adverse ground subsidence and deflections. The contractor should attempt to cut the excavation as close to neat lines as possible. Where voids are created, they should be backfilled as soon as possible with sand, gravel, or grout.

As previously mentioned, we recommend that a monitoring program be developed and implemented to evaluate the effects of the shoring on adjacent improvements. All sensitive improvements should be located and monitored for horizontal and vertical deflections and distress cracking based on a pre-construction survey. For multi-level excavations, the installation of inclinometers at critical areas may be desired for more detailed deflection monitoring. The monitoring frequency should be established and agree to by the project team prior to start of shoring construction.

The above recommendations are for the use of the design team; the contractor in conjunction with input from the shoring designer should perform additional subsurface exploration they deem necessary to design the chosen shoring system. A California-licensed civil or structural engineer must design and be in responsible charge of the temporary shoring design. The above design parameters are minimums, and may not be suitable for situations other than simple braced systems. The contractor is responsible for means and methods of construction, as well as site safety.

6.5.2 Construction Dewatering

Groundwater levels are expected to be about 12 to 13 feet above the planned excavation bottom; therefore, temporary dewatering will 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 5 feet below the bottom of the mass excavation, and at least 2 feet below localized excavations such as deepened footings, 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.

Temporary draw down of the groundwater table can cause the subsidence outside the excavation area, causing settlement of adjacent improvements. Our preliminary analysis for assumed a dewatering depth of 35 feet, resulting in about ¾ inch of settlement near well points, decreasing at greater distances from well points. If this settlement is considered tolerable to the adjacent structure as well as City improvements, dewatering should be feasible. If settlement due to dewatering is not desired, the shoring can be designed as undrained cutoff walls, with secant soil-cement columns or similar.

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 6 inches, moisture conditioned, and compacted in accordance with the “Compaction” section below.

Due to the sandy soils likely to be encountered at the subgrade elevation, we recommend that subgrade compaction and proof rolling be performed within 24 hours of capillary break layer or slab-on-grade construction.

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 1 to 12 percent over the estimated laboratory optimum in the upper 24 to 28 feet of the soil profile. The contractor should anticipate drying the soils prior to reusing them as fill. In addition, as mentioned above, the bottom of the garage excavation may consist of saturated native soils; repetitive rubber-tire loading will likely de-stabilize the 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

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 on-site or 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 Below-Grade Excavation Stabilization

Dewatering at the site will lower the groundwater level to below the bottom of the planned basement excavation; however, saturated native soils may still be encountered at the bottom of the garage excavation. Therefore, we recommend that the contractor plan to excavate an additional 12 to 18 inches below subgrade, place a layer of stabilization fabric (Mirafi HP270/HP370/RS580i, or equivalent) at the bottom, and backfill with clean, crushed rock. The crushed rock should be consolidated in place with light vibratory equipment. Rubber-tire equipment should not be allowed to operate on the exposed subgrade; the crushed rock should be stockpiled and pushed out over the stabilization fabric.

As an alternative, the basement subgrade could possibly be over-excavated neat and covered with a minimum 3 or 4-inch-thick cement-sand slurry.

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 Portland Cement Concrete (PCC) will be generated during site demolition. If the AB is separated, it may be reused within the new pavement and flatwork structural sections). AC grindings may not be reused within the habitable building areas. Laboratory testing will be required to confirm the grindings meet project specifications.

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 habitable building areas, 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 within the pavement structural sections. PCC grindings also make good winter construction access roads, similar to a cement-treated base (CTB) section.

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. Imported soil for use as general fill 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 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 "Subgrade Stabilization Measures" section of this report. Where the soil's PI is 20 or greater, the expansive soil criteria should be used.

Table 3: 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
General Fill (below a depth of 5 feet)	On-Site Expansive Soils	95	>3
	Low Expansion Soils	95	>1
Basement Wall Backfill	Without Surface Improvements	90	>1
	With Surface Improvements	95[4]	>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

Table 3 continues

Table 3: Compaction Requirements (continued)

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
Pavement Subgrade	On-Site Expansive Soils	87 - 92	>3
	Low Expansion Soils	95	>1
Pavement Aggregate Base	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)
4 – Using light-weight compaction or walls should be braced

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.

6.11 SITE DRAINAGE

6.11.1 Surface 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 should be spaced at least 10 feet from buildings, and preferably at least 5 feet from slabs-on-grade or pavements.

SECTION 7: 2019 CBC SEISMIC DESIGN CRITERIA

7.1 SEISMIC DESIGN CRITERIA

We developed site-specific seismic design parameters in accordance with Chapter 16, Chapter 18 and Appendix J of the 2019 California Building Code (CBC) and Chapters 11, 12, 20, and 21 and Supplement No. 1 of ASCE 7-16.

7.1.1 Site Location and Provided Data For 2019 CBC Seismic Design

The project is located at latitude 37.566482° and longitude 122.320021°, which is based on Google Earth (WGS84) coordinates at the approximate center of the site in San Mateo, California. We have assumed that a Seismic Importance Factor (I_e) of 1.00 has been assigned to the structure in accordance with Table 1.5-2 of ASCE 7-16 for structures classified as Risk Category II. The building period has not been provided by the project structural engineer.

7.2 2019 CBC SEISMIC DESIGN CRITERIA

Based on our experience in the project vicinity, geologic mapping, and the alluvial soils encountered within our exploratory borings, we have classified the site as Soil Classification D, which is described as a “stiff soil” profile. 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. Our site-specific ground motion hazard analysis considered a V_{S30} of 280 m/s (918 ft/s).

In accordance with Section 11.4.8 of ASCE 7-16, we performed a ground motion hazard analysis following Chapter 21, Section 21.2 of ASCE 7-16. We evaluated both Probabilistic MCE_R Ground Motions, in accordance with Method 1 and Method 2, and Deterministic MCE_R Ground Motions to generate our recommended design response spectrum for the project, see

Figure 6. The recommended design spectral accelerations and associated periods are provided in graphically on Figure 7.

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

Due to the potential of hydrostatic uplift, in our opinion, the proposed structures may be supported on a mat foundation provided the recommendations in the “Earthwork” section and the sections below are followed.

8.2 MAT FOUNDATIONS

8.2.1 Reinforced Concrete Mat Foundations

As the below grade parking garage will extend below the design ground water level, the below-grade structure should be supported on a mat foundation due to the magnitude of hydrostatic uplift. The mat foundation should bear on natural soil or 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 2019 California Building Code.

To reduce potential differential movement, the mat should be designed for an average aerial bearing pressure of 1,500 psf for dead plus live loads; at column or wall loading, the maximum localized bearing pressure should be limited to 3,000 psf (dead plus live loads). 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 actual average areal bearing pressure is higher than presented above, or if there are other aspects of design not accounted for in this report, please notify us so that we may revise our recommendations.

8.2.2 Mat Foundation Settlement

On a preliminary basis, we estimate static settlements on the order of ½-inch along the edges of the mat, and on the order of 1-inch at the center of the mat. Post construction differential static settlements of ½-inch are anticipated for recompression of the subgrade soils between the center and edges of the mat. As previously mentioned, structural loads were not provided; therefore, we assumed loads based on our experience with similar-type structures. Once structural loads are finalized, we recommend we be retained to review the final layout and loading, and verify the settlement estimates above.

In addition to estimated differential static settlements, the mats should be designed to accommodate an estimated seismic differential movement of ½-inch over a horizontal distance of 50 feet.

8.2.3 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.

8.2.4 Mat Foundation Construction Considerations

Mat subgrade areas should be kept moist until concrete placement by regular sprinkling to prevent desiccation. If deep drying is allowed to occur, several days of moisture conditioning (flooding of the pads is not recommended) may be required to allow the moisture to re-penetrate the subgrade. If sever drying occurs, reworking and moisture conditioning of the pad may be required. Prior to placement of any waterproofing and mat construction, the subgrade should be proof-rolled and visually observed by a Cornerstone representative to confirm stable subgrade conditions. The pad moisture should also be checked at least 24 hours prior to vapor barrier or mat reinforcement placement to confirm that the soil has a moisture content of at least 1 percent over optimum in the upper 12 inches.

8.2.5 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 will require at least one iteration between our soil model and the structural SAFE (or similar) analysis for the mat. As discussed above, the estimated average areal mat pressure is approximately 1,500 psf within the proposed structure. Based on this assumed pressure, we calculated a preliminary modulus of subgrade reaction value for the mat foundation.

For preliminary SAFE runs (or equivalent analysis), we recommend an initial modulus of soil subgrade reaction of 15 pounds per cubic inch (pci) for the mat foundation. As discussed above, the modulus of soil subgrade reaction is intended for use in the first iteration of the structural SAFE analysis for the mat design. Once the initial structural analysis is complete, please forward a color plot of contact pressures for the mat (to scale) so that we can provide a revised plan with updated contours of equal modulus of soil subgrade reaction values.

8.2.6 Hydrostatic Uplift and Waterproofing

Mat foundations that extend below the recommended design groundwater level of 12 feet, should be designed to resist potential hydrostatic uplift pressures. A buoyancy evaluation should be performed by the structural engineer to evaluate the number of uplift ground anchors are needed, if any. Basement walls extending below design groundwater should be designed to resist hydrostatic pressure. Where portions of the walls extend above the design groundwater level, a drainage system may be added as discussed in the "Retaining Wall" section.

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

8.3 UPLIFT GROUND ANCHORS

We understand that ground anchors may be used to resist hydrostatic uplift due to the water table. The following presents preliminary ultimate uplift capacities. The structural engineer should apply an appropriate factor of safety to the ultimate values. The following values are for post-grouted anchors with pressures exceeding about 150 psi based on information in Section 5.9 of FHWA's Geotechnical Engineering Circular No. 4 (1999), "Ground Anchors and Anchored Systems." All anchors should be load tested to confirm design capacity in accordance with FHWA recommendations. We assume at least one verification test will be performed and the remaining anchors will be proof tested. Ground anchors should be spaced at a minimum of 3 diameters otherwise, a reduction for group effects may be required and structural engineer should check group effects. Construction tolerances for vertical alignment should be specified such that there will not be overlap at the anchor tips. We recommend the post grout injection ports be spaced every 5 feet.

The excavation of all drilled shafts should be observed by a Cornerstone representative to confirm the soil profile and that the ground anchors are constructed in accordance with our recommendations and project requirements. The drilled ground anchors should be straight, dry, and relatively free of loose material before grout and reinforcing steel is placed. If ground water cannot be removed from the excavations prior to grout placement, casing or drilling slurry may be required to stabilize the shaft and the grout should be placed using a tremie pipe, keeping the tremie pipe below the surface of the grout to avoid entrapment of water or drilling slurry in the grout.

Due to the loose nature of the cleaner sand layers (fines content between 9 and 12 percent) documented in the previous borings, the use of casing, drilling slurry, or other methods to stabilize the hole of each drilled shaft may be required.

Table 4: Recommended Ground Anchor Ultimate Capacities

Anchor Length Below Footing (feet)	Ultimate Uplift Capacity (kips)	
	6-inch Diameter	8-inch Diameter
20	62	84
35	110	147
50	157	209

We recommend a ground anchor design-build contractor be retained to confirm the information provided and for additional recommendations, as required.

SECTION 9: VEHICULAR PAVEMENTS

9.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 5. Patching of existing asphalt concrete pavements in the public right-of-way should match in kind the existing structural section, or conform to a minimum section provided by the City.

Table 5: Asphalt Concrete Pavement Recommendations

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base[1] (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.5	10.0
4.5	2.5	9.5	12.0
5.0	3.0	10.0	13.0
5.5	3.0	12.0	15.0
6.0	3.5	13.0	16.5
6.5	4.0	14.0	18.0

[1] Caltrans Class 2 aggregate base; minimum R-value of 78; subgrade R-value of 5

9.2 PORTLAND CEMENT CONCRETE

Portland Cement Concrete (PCC) driveway entrances to the site in the public right-of-way should be designed and constructed in accordance with City requirements. Any portion of a concrete driveway on grade within private property should have a structural section of at least 6 inches of concrete overlying at least 6 inches of Class 2 aggregate base compacted as recommended in the “Earthwork” section. The concrete should have a compressive strength of at least 3,500 psi and be laterally restrained with curbs or concrete shoulders. Adequate expansion and control joints should be included. 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. If there is an at-grade concrete trash enclosure slab where the large dumpsters are stored, it should be at least 8 inches thick overlying at least 6 inches of Class 2 aggregate base.

9.3 PAVEMENT CUTOFF

Surface water penetration into the pavement section can significantly reduce the pavement life, due to the native expansive clays. While quantifying the life reduction is difficult, a normal 20-year pavement design could be reduced to less than 10 years; therefore, increased long-term maintenance may be required.

It would be beneficial to include a pavement cut-off, such as deepened curbs, redwood-headers, or “Deep-Root Moisture Barriers” that are keyed at least 4 inches into the pavement subgrade. This will help limit the additional long-term maintenance.

9.4 EXTERIOR 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 8 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the “Earthwork” recommendations of this report. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. 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 should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

SECTION 10: RETAINING WALLS

10.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall 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. If traffic lanes are within 10 feet of the basement wall, an additional horizontal surcharge of 125 psf should be applied in the upper 5 feet of 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:

Table 6: Recommended Lateral Earth Pressures

Wall Condition	Lateral Earth Pressure*	Additional Surcharge Loads
Unrestrained – Cantilever Wall	45 pcf	⅓ of vertical loads at top of wall
Restrained – Braced Wall	45 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
** H is the distance in feet between the bottom of footing and top of retained soil

Basement walls should be designed as restrained walls with hydrostatic fluid pressures below the design groundwater of 12 feet below current grade. If adequate drainage cannot be provided behind the wall, an additional equivalent fluid pressure of 40 pcf should be added to the values above for both restrained and unrestrained walls for the portion of the wall that will not have drainage. Damp proofing or waterproofing of the walls may be considered where moisture penetration and/or efflorescence are not desired. If a perimeter drainage system will not be included above 12 feet below current grade, the full height of the walls should include the additional 40 pcf equivalent fluid pressure.

10.2 SEISMIC LATERAL EARTH PRESSURES

The 2019 California Building Code (CBC) states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. We checked seismic earth pressures for the proposed restrained and unrestrained (cantilever) retaining walls in accordance with CBC 1803.5.12 and ASCE 7-16 Section 11.8.3 using the Design level earthquake. We developed seismic earth pressures for the proposed basement using interim recommendations generally based on refinement of the Mononobe-Okabe method (Lew et al., SEAOC 2010).

Because the walls are greater than 12 feet in height, and peak ground accelerations are greater than 0.40g, we checked the result of the seismic increment when added to the recommended active earth pressure against the recommended fixed wall earth pressures. Basement walls are not free to deflect, and should therefore be designed for static conditions as a restrained wall, which is also a CBC requirement. Based on current recommendations for seismic earth pressures, it appears that active earth pressures plus a seismic increment exceed the restrained (i.e. at-rest), static wall earth pressures. Therefore, we recommend checking the walls for the seismic condition in accordance with the interim recommendations of the above referenced paper and the 2019 CBC.

The CBC prescribes basic load combinations for structures, components and foundations with the intention that their design strength equals or exceeds the effects of the factored loads. With respect to the load from lateral earth pressure and ground water pressure, the CBC prescribes the basic combinations shown in CBC equations 16-2 and 16-7 below.

$$1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) \qquad \text{[Eq. 16-2]}$$

In Eq. 16-2: H - should represent the total static lateral earth pressure, which for the basement wall will be restrained (use 45 pcf + 8H psf)

0.9(D + F) + 1.0E + 1.6H [Eq. 16-7]

In Eq. 16-7: H - should represent the static “active” earth pressure component under seismic loading conditions (use 45 pcf)

E - should represent the seismic increment component in Eq. 16-7, a triangular load with a resultant force of $14H^2$, which should be applied one third of the height up from the base of the wall (and which can also be expressed as an equivalent fluid pressure equal to 28 pcf).

The interim recommendations in the SEAOC paper more appropriately split out "active" earth pressure (and not the restrained "at-rest" pressure) from our report and provide the total seismic increment so that different load factors can be applied in accordance with different risk levels.

10.3 WALL DRAINAGE

10.3.1 At-Grade Site Walls

Adequate drainage should be provided by a subdrain system behind all walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 12 inches out from the wall and to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or approved equivalent. The upper 2 feet of wall backfill should consist of compacted on-site soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or equivalent drainage matting can be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. Horizontal strip drains connecting to the vertical drainage matting may be used in lieu of the perforated pipe and crushed rock section. The vertical drainage panel should be connected to the perforated pipe or horizontal drainage strip at the base of the wall, or to some other closed or through-wall system such as the TotalDrain system from AmerDrain. Sections of horizontal drainage strips should be connected with either the manufacturer’s connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path.

Drainage panels should terminate 18 to 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

10.3.2 Below-Grade Walls

Miradrain, AmerDrain or other equivalent drainage matting should be used for wall drainage where below-grade walls are temporarily shored and the shoring will be flush with the back of the permanent walls. The drainage panel should be connected at the base of the wall by a horizontal drainage strip and closed or through-wall system such as the TotalDrain system from AmerDrain.

Sections of horizontal drainage strips should be connected with either the manufacturer's connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path. In addition, where drainage panels will connect from a horizontal application to vertical basement wall drainage panels, the drainage path must be maintained.

Drainage panels should terminate 18 to 24 inches from final exterior grade unless capped by hardscape. The drainage panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil. If the shoring system will be offset behind the back of permanent wall, the drainage systems discussed in the "At-Grade Site Walls" section may also be used.

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.

As discussed previously, consideration should be given to the transitions from on-grade to on-structure. Providing subslabs or other methods for reducing differential movement of flatwork or pavements across this transition should be included in the project design.

10.5 FOUNDATIONS

We anticipate that the walls of the below-grade parking structure will be supported on the mat foundation as mentioned above.

SECTION 11: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Windy Hill Property Ventures specifically to support the design of the Block 21 project in San Mateo, 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 ground water 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.

Windy Hill Property Ventures may have provided Cornerstone with plans, reports and other documents prepared by others. Windy Hill Property Ventures 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 his 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

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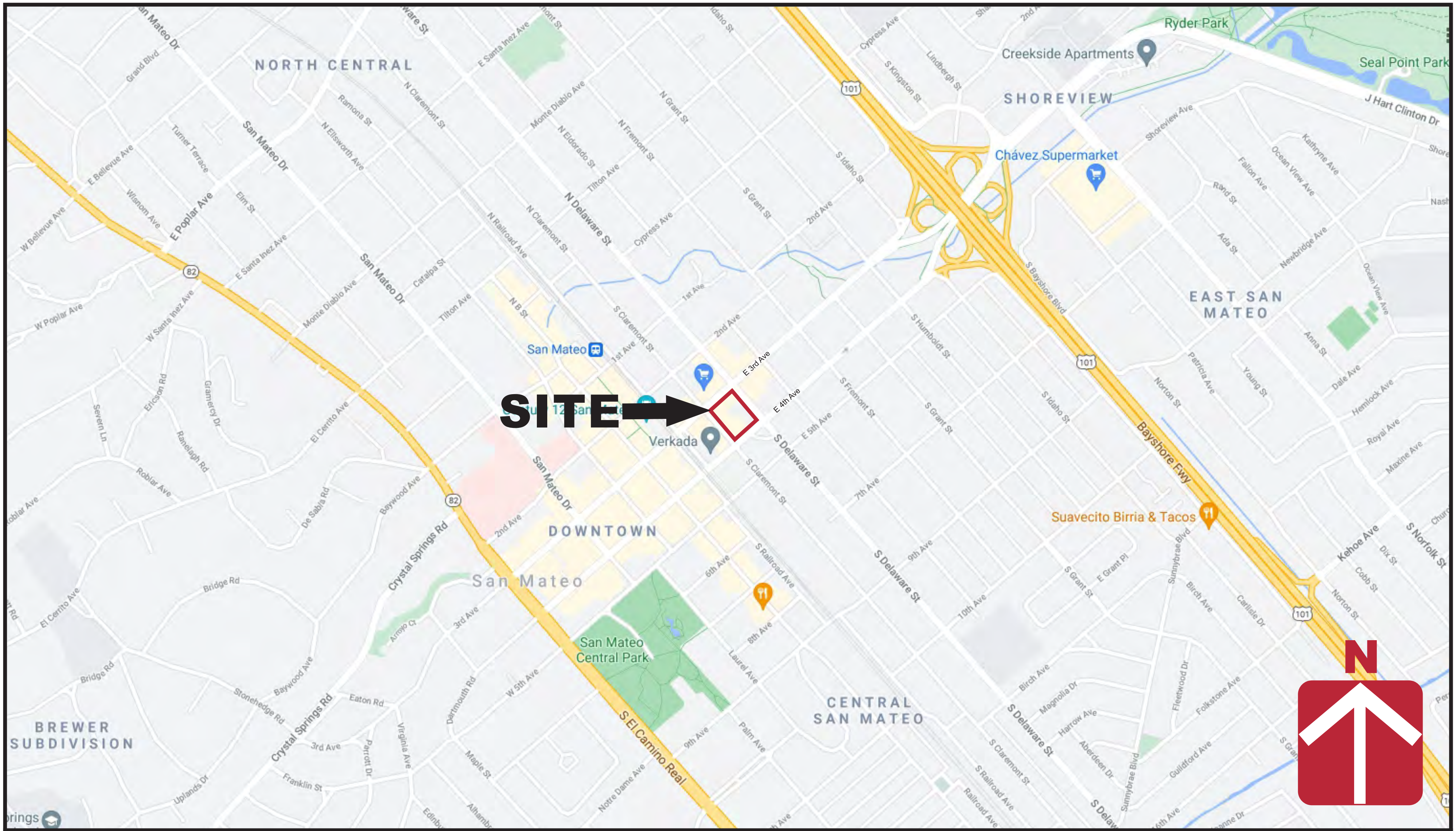

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Vicinity Map

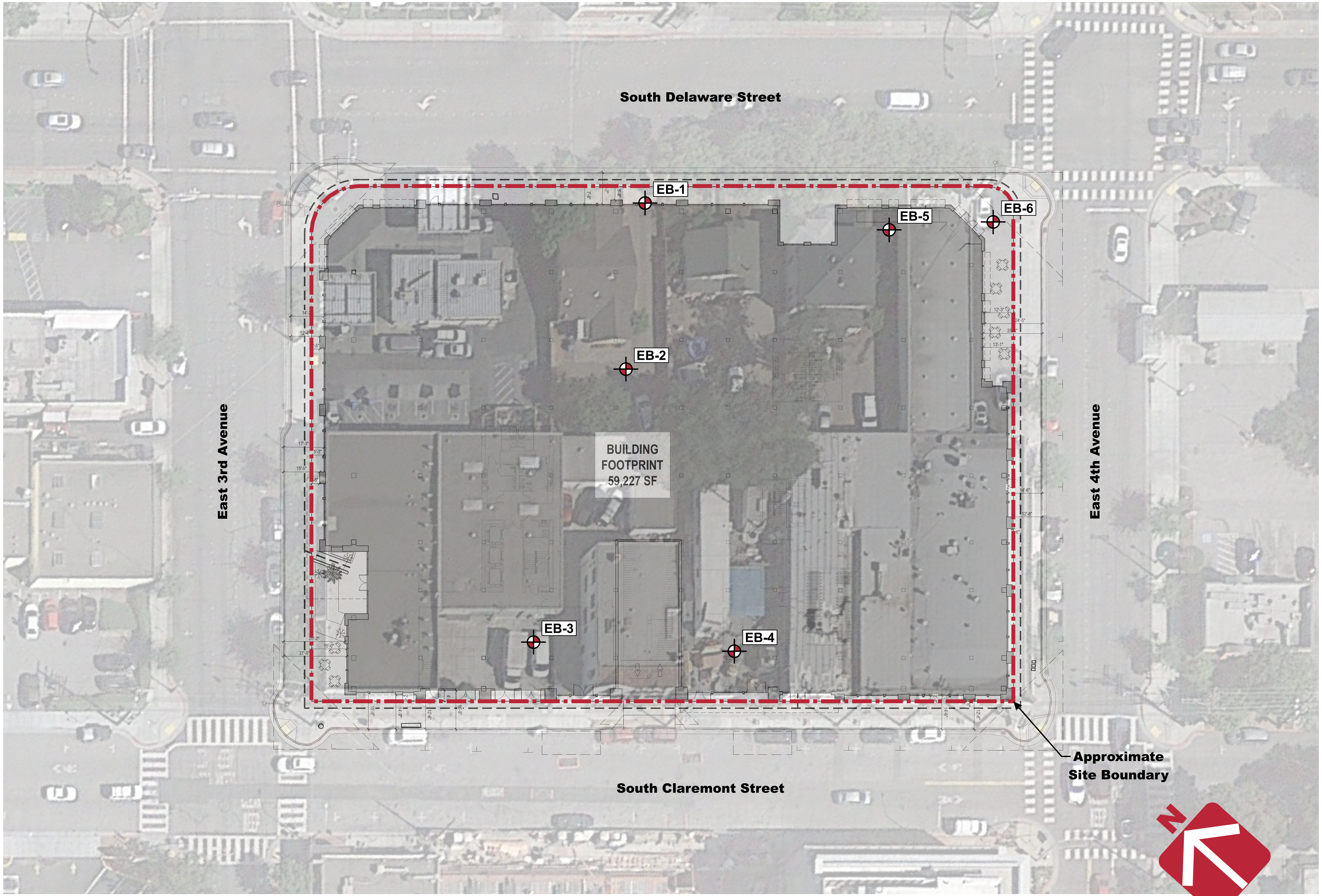
Block 21
East 3rd Avenue and South Delaware Street Office Development
San Mateo, CA

Project Number
803-11-1

Figure Number
Figure 1

Date
August 2021

Drawn By
RRN



Legend

⌖ Approximate location of exploratory boring (EB)

Base by Google Earth, dated 09/26/2020
Overlay by Architectural Technologies, Site Plan - A 1.01, dated 04/21/2021

Project Number
803-11-1

Figure Number
Figure 2

Date
August 2021

Drawn By
RRN

Site Plan

Block 21
East 3rd Avenue and South Delaware Street
Office Development
San Mateo, CA

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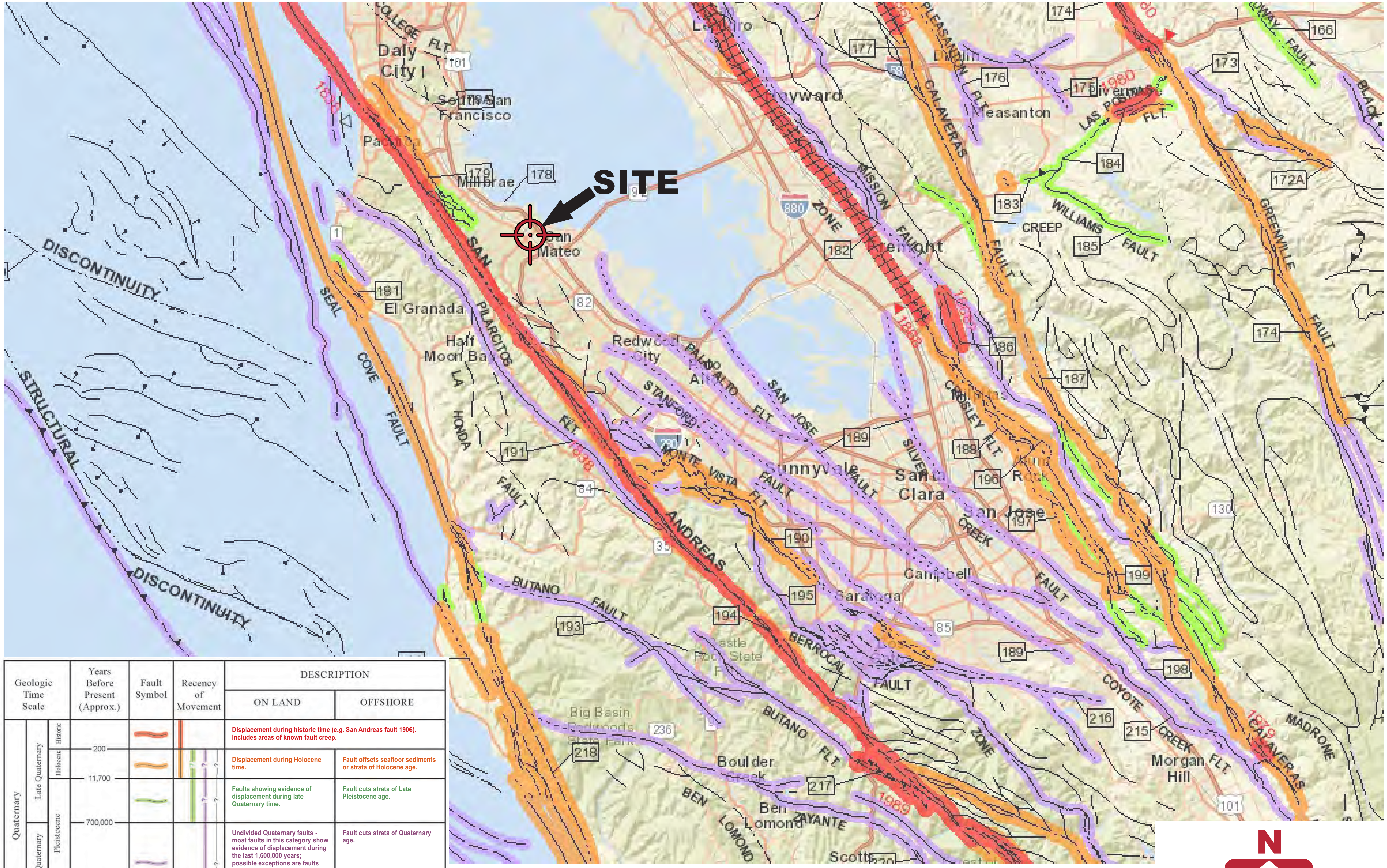

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
						ON LAND	OFFSHORE
Quaternary	Late Quaternary	Historic		🟥		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200	🟧		Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700	🟩		Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000	🟪		Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Pilo-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000 4.5 billion (Age of Earth)	—		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

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

N

0 5 10

APPROXIMATE SCALE (MILES)

Regional Fault Map

Block 21
East 3rd Avenue and South Delaware Street
Office Development
San Mateo, CA

Project Number 803-11-1

Figure Number Figure 3

Date August 2021

Drawn By RRN

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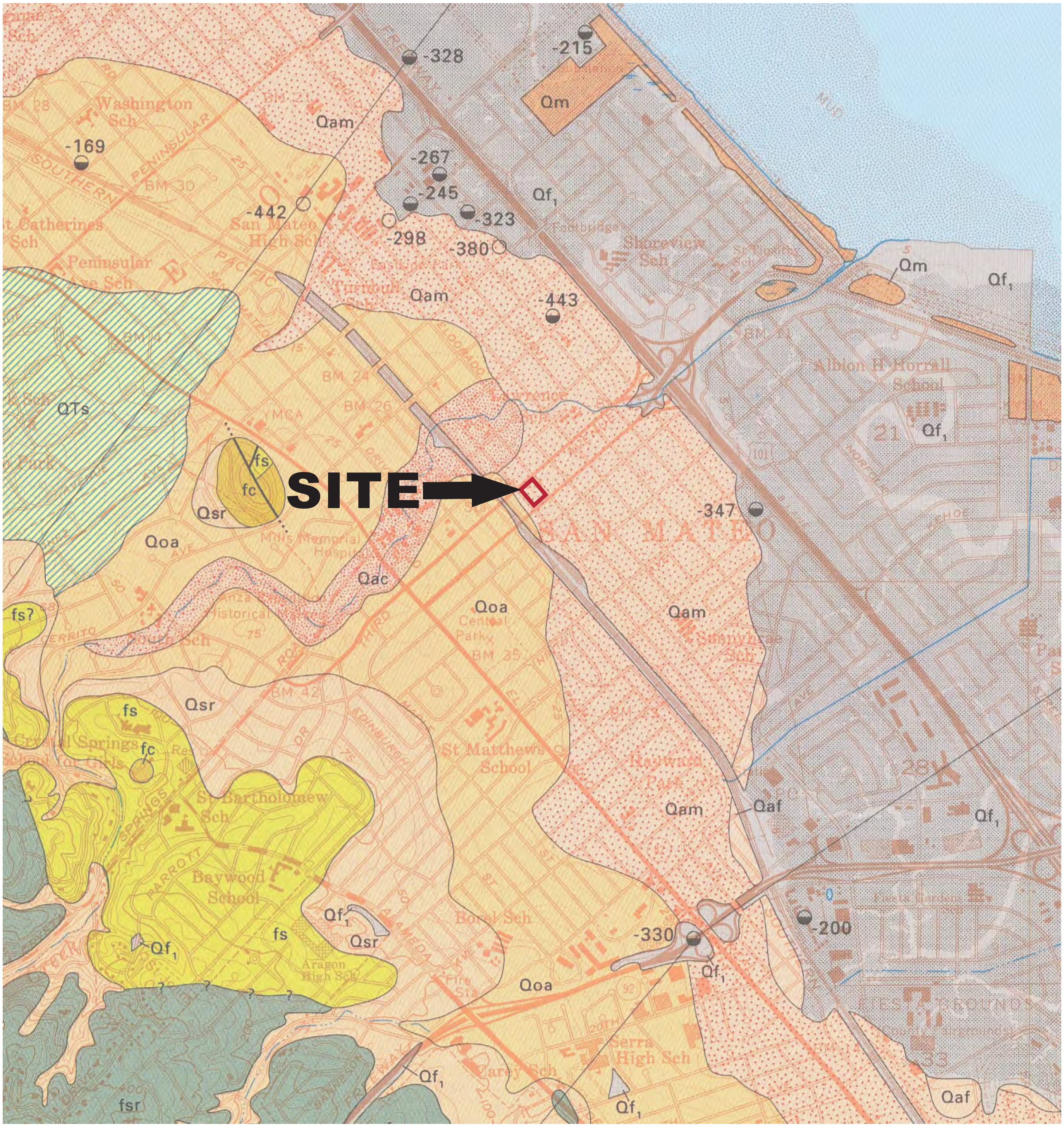

Geologic Units

Qam	Medium-grained alluvium (Holocene)
Qac	Coarse-grained alluvium (Holocene)
Qf_1	Artificial fill (Holocene)
Qoa	Older alluvium (Pleistocene)

Explanation

Contact- dashed where approximate, dotted where concealed

N

0 2000 4000

APPROXIMATE SCALE (FEET)

Base: Geologic Map of the Montara Mountain and San Mateo 7.5' Quadrangles, San Mateo County, California, by Pampeyan, 1994

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Vicinity Geologic Map

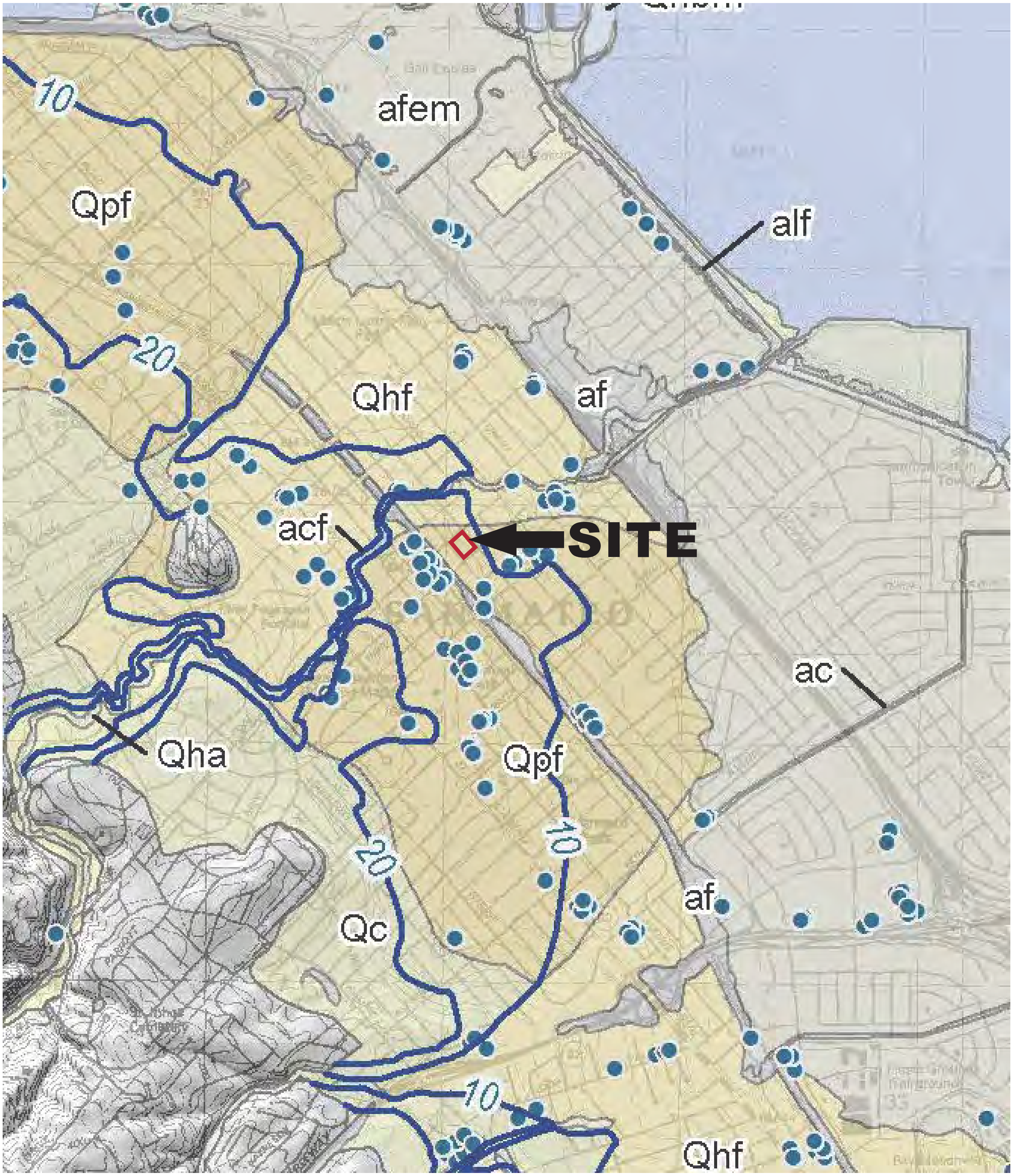
Block 21
East 3rd Avenue and South Delaware Street
Office Development
San Mateo, CA

Project Number 803-11-1

Figure Number Figure 4

Date August 2021

Drawn By RRN



Legend

● Groundwater measurement location

—10— Depth to groundwater (in feet)

N

0 2000 4000

APPROXIMATE SCALE (FEET)

Base: Plate 1.2 Depth to Historic-High Groundwater Levels in Quaternary Alluvial
Deposits and Groundwater Measurement Locations, San Mateo Quadrangle, California

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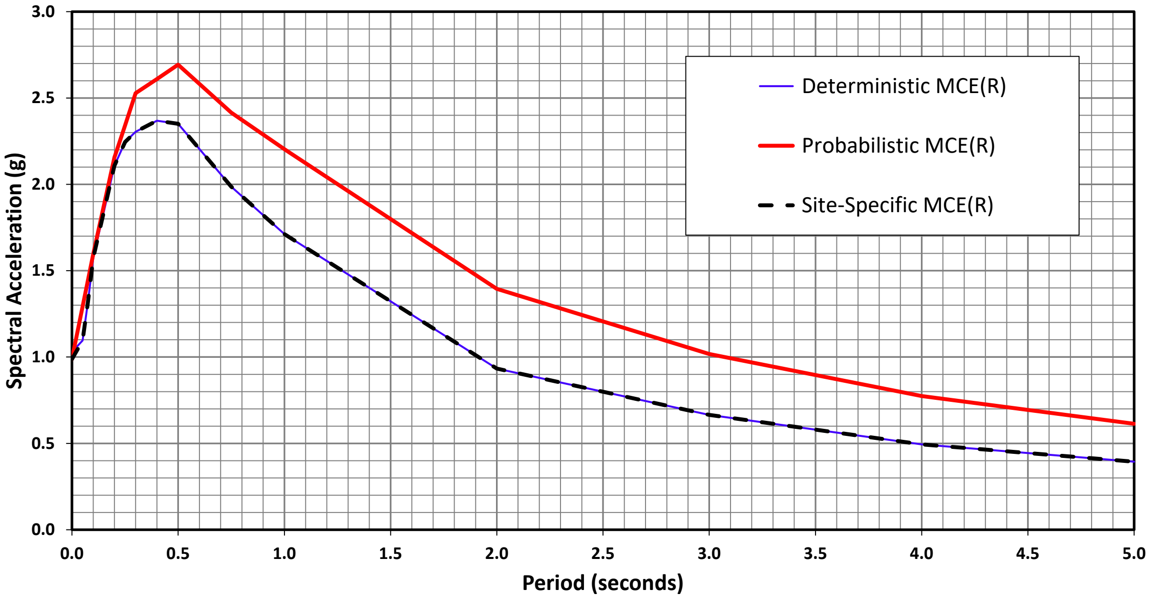
Depth to Historically High Groundwater

Block 21
East 3rd Avenue and South Delaware Street
Office Development
San Mateo, CA

Project Number	803-11-1
Figure Number	Figure 5
Date	September 2021
Drawn By	RRN



The Site-Specific Maximum Considered Earthquake (MCE_R) is defined as the lesser of the following at all periods:

- Deterministic MCE_R – maximum 84th percentile deterministic, or
- Probabilistic MCE_R – defined as the 2,475–year ground motion.

Site-Specific MCE_R	
Period (Seconds)	Spectral Acceleration (g)
0.00	0.988
0.05	1.096
0.08	1.318
0.10	1.574
0.20	2.108
0.21	2.122
0.25	2.245
0.30	2.304
0.40	2.368
0.50	2.351
0.75	1.987
1.00	1.712
1.03	1.693
2.00	0.933
3.00	0.665
4.00	0.494
5.00	0.394

References:

ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Strutures with Supplement No. 1.
2019 California Building Code, Title 24, Part 2, Volume 2

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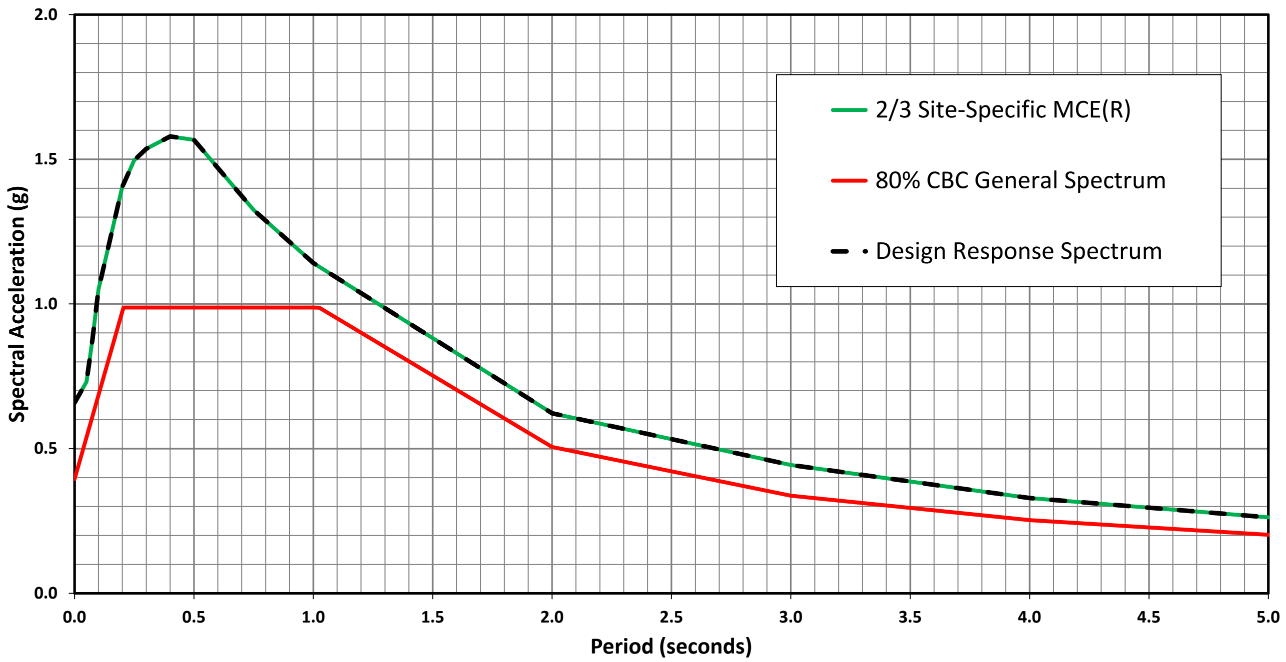
MCE_R RESPONSE SPECTRA

ck 21, E 3rd Ave, Delaware St Office Developm
315 South Claremont
San Mateo, CA

FIGURE 6

PROJECT NO. 803-11-1

August 10, 2021 | SCO



The Site-Specific Design Response Spectrum per Section 21.2, 21.3 and 21.4 of ASCE 7-16 is defined as the greater of the following at all periods:

- 2/3 of the Site-Specific MCE_R, or
- 80% of the CBC General Spectrum.

Design Response Spectra	
Period (Seconds)	Spectral Acceleration (g)
0.00	0.659
0.05	0.731
0.08	0.879
0.10	1.049
0.20	1.405
0.21	1.415
0.25	1.497
0.30	1.536
0.40	1.579
0.50	1.567
0.75	1.325
1.00	1.141
1.03	1.129
2.00	0.622
3.00	0.444
4.00	0.330
5.00	0.263

Site Design	Design Values
Site Class (Per Chapter 20 ASCE 7-16)	D
Shear Wave Velocity, V_{S30} (m/sec)	280
Site Latitude (degrees)	37.566482
Site Longitude (degrees)	-122.320021
Risk Category	II
Building Period (sec)	Unknown
Importance Factor, I_e	1
[1]Site Specific PGA_M (g)	0.90

[1] Lower of Deterministic and Probabilistic, but not less than 80% of mapped value of FM x PGA, determined in accordance with Section 21.5 of ASCE 7-16.

Design Acceleration Parameters[1]	
S_{DS}	1.421
S_{D1}	1.331
S_{MS}	2.132
S_{M1}	1.996

References:

ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Strutures with Supplement No. 1.
2019 California Building Code, Title 24, Part 2, Volume 2

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DESIGN RESPONSE SPECTRA

k 21, E 3rd Ave, Delaware St Office Developm
315 South Claremont
San Mateo, CA

FIGURE 7

PROJECT NO. 803-11-1

August 10, 2021 | SCO

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem, limited-access auger drilling equipment. Six 8-inch-diameter exploratory borings were drilled on July 21 to depths of 40 to 60 feet. The approximate locations of exploratory borings 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, are included as part of this appendix.

Boring locations were approximated using existing site boundaries and other site features as references. Boring elevations were not determined. The locations of the borings 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.

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 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 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.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	Cu>4 AND 1<Cc<3	GW	WELL-GRADED GRAVEL
			Cu>4 AND 1>Cc>3	GP	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	Cu>6 AND 1<Cc<3	SW	WELL-GRADED SAND
			Cu>6 AND 1>Cc>3	SP	POORLY-GRADED SAND
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY
			PI>4 AND PLOTS<"A" LINE	ML	SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY
			PI PLOTS <"A" LINE	MH	ELASTIC SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT

OTHER MATERIAL SYMBOLS

- Poorly-Graded Sand with Clay
- Clayey Sand
- Sandy Silt
- Artificial/Undocumented Fill
- Poorly-Graded Gravelly Sand
- Topsoil
- Well-Graded Gravel with Clay
- Well-Graded Gravel with Silt
- Sand
- Silt
- Well Graded Gravelly Sand
- Gravelly Silt
- Asphalt
- Boulders and Cobble

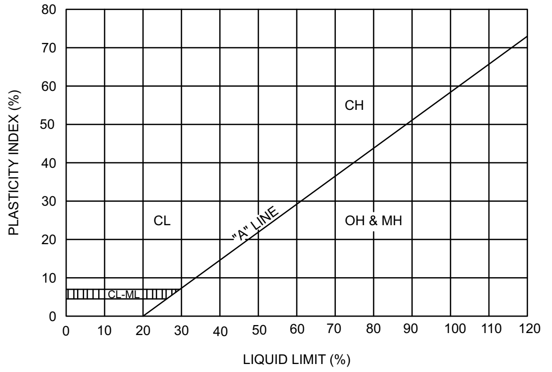
SAMPLER TYPES

- SPT
- Modified California (2.5" I.D.)
- Rock Core
- Shelby Tube
- No Recovery
- Grab Sample

ADDITIONAL TESTS

CA	-	CHEMICAL ANALYSIS (CORROSIVITY)	PI	-	PLASTICITY INDEX
CD	-	CONSOLIDATED DRAINED TRIAXIAL	SW		SWELL TEST
CN	-	CONSOLIDATION	TC	-	CYCLIC TRIAXIAL
CU	-	CONSOLIDATED UNDRAINED TRIAXIAL	TV	-	TORVANE SHEAR
DS	-	DIRECT SHEAR	UC	-	UNCONFINED COMPRESSION
PP	-	POCKET PENETROMETER (TSF)	(1.5)	-	(WITH SHEAR STRENGTH IN KSF)
(3.0)	-	(WITH SHEAR STRENGTH IN KSF)		-	
RV	-	R-VALUE	UU		UNCONSOLIDATED UNDRAINED TRIAXIAL
SA	-	SIEVE ANALYSIS: % PASSING #200 SIEVE		-	
▼	-	WATER LEVEL			

PLASTICITY CHART



PENETRATION RESISTANCE (RECORDED AS BLOWS / FOOT)					
SAND & GRAVEL		SILT & CLAY			
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	STRENGTH** (KSF)	
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25	
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.5	
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	0.5-1.0	
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0	
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0	
		HARD	OVER 30	OVER 4.0	

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

** UNDRAINED SHEAR STRENGTH IN KIPS/SQ. FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST, POCKET PENETROMETER, TORVANE, OR VISUAL OBSERVATION.

CORNERSTONE
EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number
A-1

BORING NUMBER EB-1

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME Block 21, E 3rd Ave & Delaware Mixed-Use Development
PROJECT NUMBER 803-11-1
PROJECT LOCATION San Mateo, CA

DATE STARTED 7/22/21 **DATE COMPLETED** 7/22/21
GROUND ELEVATION **BORING DEPTH** 45 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc
LATITUDE 37.5667° **LONGITUDE** -122.3198°
DRILLING METHOD Mobile B-40, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CRS
▽ **AT TIME OF DRILLING** 22 ft.
NOTES
▼ **AT END OF DRILLING** 20 ft.

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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
3 inches gravel							
Sandy Lean Clay (CL) hard, moist, brown, fine to coarse sand, some fine subangular to subrounded gravel, low to moderate plasticity	62	MC-1B	112	10			○ >4.5
Lean Clay with Sand (CL) hard, moist, brown, fine to coarse sand, moderate plasticity Liquid Limit = 46, Plastic Limit = 16	54	MC-2B	112	14	30		○ >4.5
Clayey Sand with Gravel (SC) dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subrounded to subangular gravel	80	MC-3B	122	9			
	73	MC-4B	113	11			
Well Graded Sand with Clay and Gravel (SW-SC) dense, moist, brown, fine to coarse sand, fine to coarse subrounded to subangular gravel	73	MC-5C	122	8		11	
becomes medium dense	38	MC-6B	114	11			
Clayey Sand with Gravel (SC) medium dense, moist, brown, fine to coarse sand, fine to coarse subrounded to subangular gravel	19	SPT-7		14			

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

Continued Next Page

CORNERSTONE EARTH GROUP

BORING NUMBER EB-1

PAGE 2 OF 2

PROJECT NAME Block 21, E 3rd Ave & Delaware Mixed-Use Development
PROJECT NUMBER 803-11-1
PROJECT LOCATION San Mateo, CA

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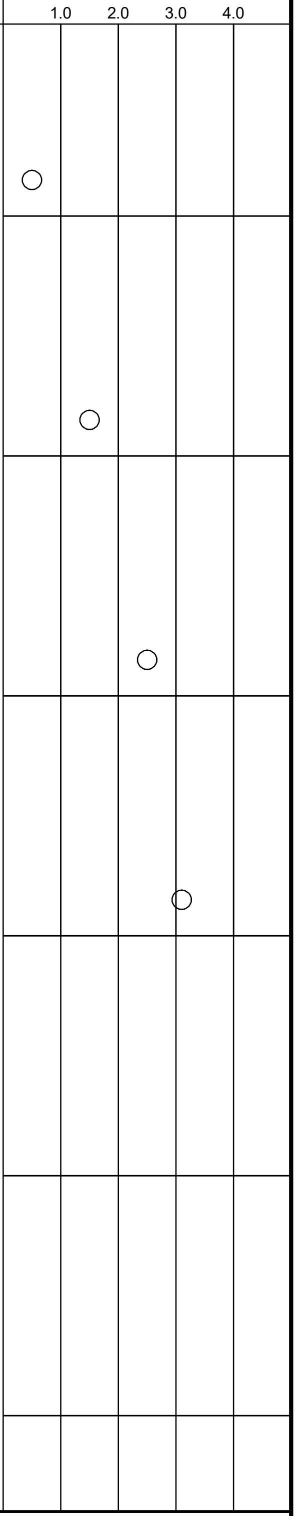
ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
			Sandy Lean Clay (CL) medium stiff, moist, brown, fine to coarse sand, some fine subangular to subrounded gravel, low to moderate plasticity	14	SPT-8		20		
	30				ST				
			becomes stiff	20	SPT-10		16		
	35								
			Lean Clay with Sand (CL) very stiff, moist, brown, fine to coarse sand, some fine gravel, moderate plasticity	45	SPT-11		16		
	40								
				38	SPT-12		19		
	45		Bottom of Boring at 45.0 feet.						
	50								
	55								

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



BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME Block 21, E 3rd Ave & Delaware Mixed-Use Development

PROJECT NUMBER 803-11-1

PROJECT LOCATION San Mateo, CA

DATE STARTED 7/22/21 **DATE COMPLETED** 7/22/21

GROUND ELEVATION **BORING DEPTH** 49.9 ft.

DRILLING CONTRACTOR Exploration Geoservices Inc

LATITUDE 37.5666° **LONGITUDE** -122.3200°

DRILLING METHOD Mobile B-40, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY CRS

▽ **AT TIME OF DRILLING** 22 ft.

NOTES

▼ **AT END OF DRILLING** 22 ft.

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

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	UNDRAINED SHEAR STRENGTH, ksf
	0	3 inches gravel							
		Sandy Lean Clay (CL) hard, moist, brown, fine to coarse sand, some fine subangular gravel, low to moderate plasticity							
			36	MC-1C	105	10			>4.5 ○
			59	MC-2B	113	11			>4.5 ○
	5	**Lean Clay with Sand (CL)** hard, moist, brown, fine to coarse sand, moderate plasticity							
			50/6"	MC-3B	114	15			>4.5 ○
		Clayey Sand with Gravel (SC) very dense, moist, brown with light brown mottles, fine to coarse sand, fine to coarse subrounded to subangular gravel							
			50/5"	MC-4B	107	11			
	10								
			50/4"	MC-5B	115	11			
	15								
		Well Graded Sand with Clay and Gravel (SW-SC) very dense, moist, brown, fine to coarse sand, fine to coarse subrounded to subangular gravel							
				GB-6		7			
			66	SPT-7		9			
	20								
▼									
		becomes dense	43	SPT-8		11		12	
	25								

Continued Next Page

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-2

PAGE 2 OF 2

PROJECT NAME Block 21, E 3rd Ave & Delaware Mixed-Use Development
PROJECT NUMBER 803-11-1
PROJECT LOCATION San Mateo, CA

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

1.0 2.0 3.0 4.0

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
Well Graded Sand with Clay and Gravel (SW-SC) dense, moist, brown, fine to coarse sand, fine to coarse subrounded to subangular gravel	31	SPT-9		16		9	
becomess medium dense Liquid Limit = 41, Plastic Limit = 17	21	SPT-10		17	24		
Sandy Lean Clay (CL) hard, moist, brown with light brown mottles, fine to coarse sand, some fine subangular to subrounded gravel, low to moderate plasticity	73	SPT-11		16			>4.5 ○
Lean Clay with Sand (CL) very stiff, moist, brown, fine to coarse sand, moderate plasticity	52	SPT-12		21			○
Well Graded Sand with Clay and Gravel (SW-SC) very dense, moist, brown, fine to coarse sand, fine subrounded to subangular gravel	50 5"	SPT-13B		12			

Bottom of Boring at 49.9 feet.

BORING NUMBER EB-3

PROJECT NAME Block 21, E 3rd Ave & Delaware Mixed-Use Development
PROJECT NUMBER 803-11-1
PROJECT LOCATION San Mateo, CA
DATE STARTED 7/21/21 **DATE COMPLETED** 7/21/21
GROUND ELEVATION **BORING DEPTH** 60 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc
LATITUDE 37.5665° **LONGITUDE** -122.3205°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CRS
▽ **AT TIME OF DRILLING** 22.5 ft.
NOTES
▼ **AT END OF DRILLING** 22.5 ft.

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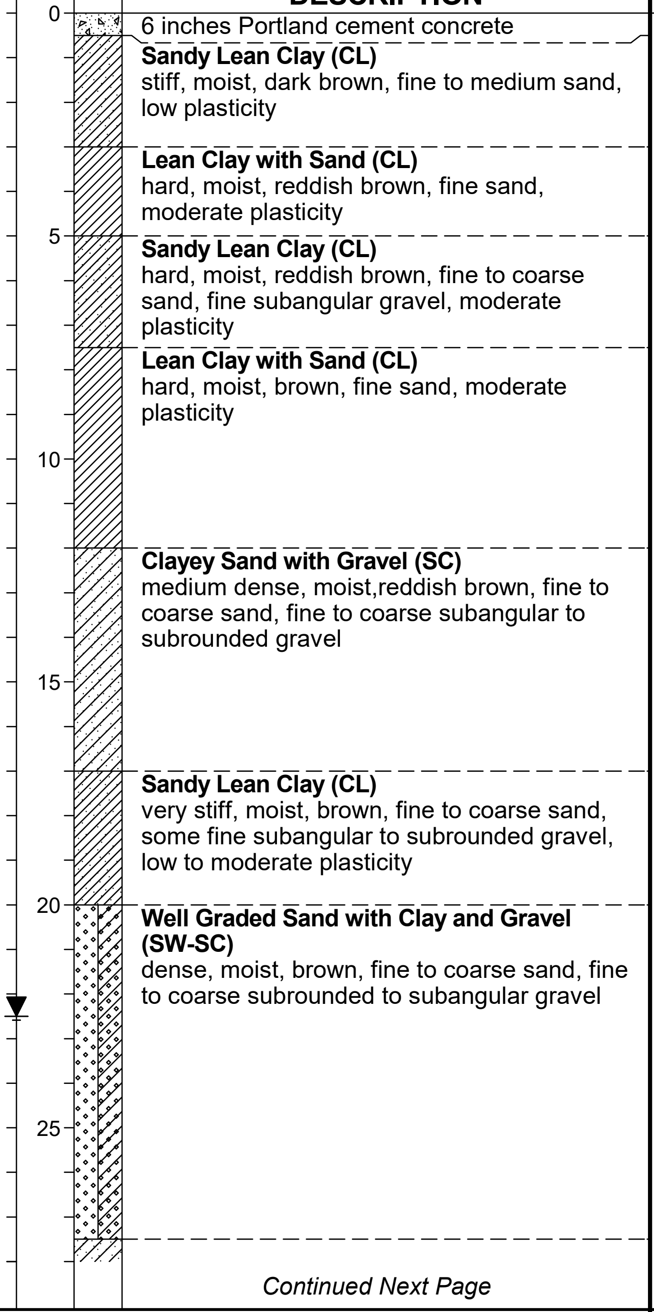


DESCRIPTION

6 inches Portland cement concrete

Sandy Lean Clay (CL)
stiff, moist, dark brown, fine to medium sand, low plasticity

Lean Clay with Sand (CL)
hard, moist, reddish brown, fine sand, moderate plasticity

Sandy Lean Clay (CL)
hard, moist, reddish brown, fine to coarse sand, fine subangular gravel, moderate plasticity

Lean Clay with Sand (CL)
hard, moist, brown, fine sand, moderate plasticity

Clayey Sand with Gravel (SC)
medium dense, moist,reddish brown, fine to coarse sand, fine to coarse subangular to subrounded gravel

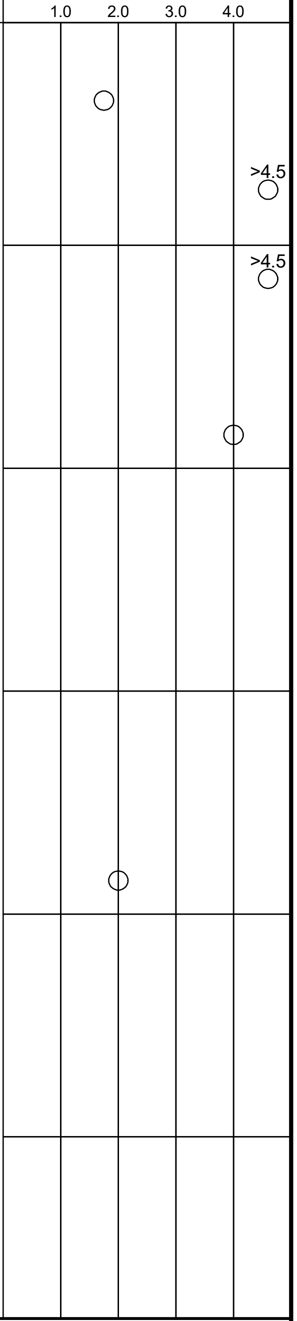
Sandy Lean Clay (CL)
very stiff, moist, brown, fine to coarse sand, some fine subangular to subrounded gravel, low to moderate plasticity

Well Graded Sand with Clay and Gravel (SW-SC)
dense, moist, brown, fine to coarse sand, fine to coarse subrounded to subangular gravel

Continued Next Page

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
9	MC-1B	92	13		
31	MC-2B	112	15		
63	MC-3B	113	16		
34	MC-4B	103	22		
52	MC-5B	121	10		24
39	MC-6B	113	19		
77	MC-7B	126	12		
43	SPT-8		8		

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



BORING NUMBER EB-3

PAGE 2 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME Block 21, E 3rd Ave & Delaware Mixed-Use Development
PROJECT NUMBER 803-11-1
PROJECT LOCATION San Mateo, CA

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ELEVATION (ft) | DEPTH (ft): 30, 35, 40, 45, 50, 55, 60 | SYMBOL

DESCRIPTION

Clayey Sand with Gravel (SC)
medium dense to dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel

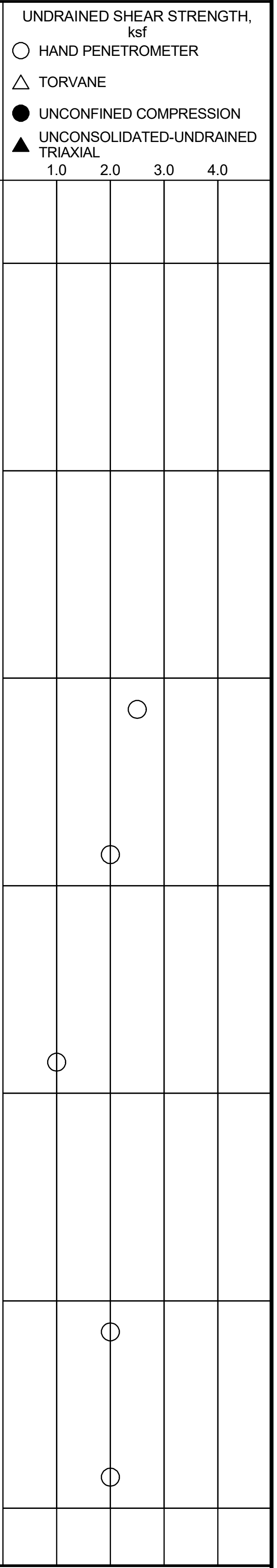
Sandy Lean Clay (CL)
very stiff, moist, reddish brown, fine to coarse sand, some fine subrounded gravel, low plasticity

Lean Clay with Sand (CL)
very stiff to stiff, moist, brown, fine sand, moderate plasticity

Sandy Lean Clay with Gravel (CL)
very stiff, moist, reddish brown, fine to coarse sand, fine subrounded gravel, low to moderate plasticity

Bottom of Boring at 60.0 feet.

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
40	MC-9	120	14		
43	SPT-10		14		
62	MC-11B	116	15		
62	NR				
73	SPT-12		19		
34	SPT-13		22		
15	SPT-14		24		
32	MC				
44	SPT-15		14		
45	SPT-16		18		



BORING NUMBER EB-4

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME Block 21, E 3rd Ave & Delaware Mixed-Use Development
PROJECT NUMBER 803-11-1
PROJECT LOCATION San Mateo, CA
DATE STARTED 7/21/21 **DATE COMPLETED** 7/21/21
GROUND ELEVATION **BORING DEPTH** 40 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc
LATITUDE 37.5663° **LONGITUDE** -122.3202°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CRS
▽ **AT TIME OF DRILLING** 22 ft.
NOTES
▼ **AT END OF DRILLING** 22.5 ft.

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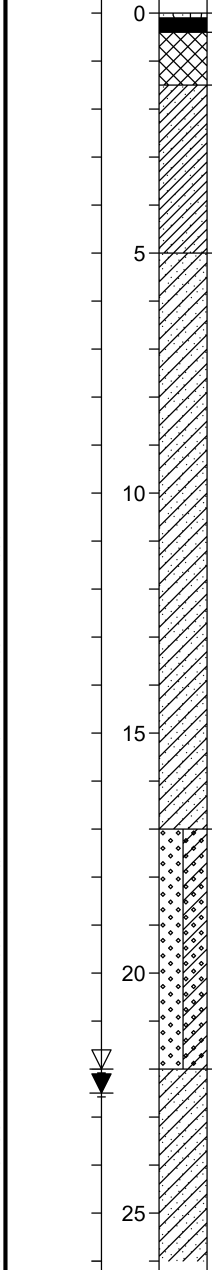

DESCRIPTION

¾ inch gravel over 3 inches asphalt concrete

Clayey Sand with Gravel (SC) [Fill]
loose, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel

Sandy Lean Clay (CL)
stiff to hard, moist, reddish brown, fine to coarse sand, some fine subrounded gravel, low to moderate plasticity

Clayey Sand with Gravel (SC)
dense to medium dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subrounded to subangular gravel

Well Graded Sand with Clay and Gravel (SW-SC)
dense, moist, brown, fine to coarse sand, fine to coarse subrounded to subangular gravel

Clayey Sand with Gravel (SC)
medium dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subrounded to subangular gravel

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	MC-1C	101	10			○
25	MC-2B	109	14			>4.5 ○
77	MC-3B	115	11			
27	MC-4B	110	9			
40	MC-5B	120	10			
60	MC-6B	115	8			
23	MC-7	110	21			
15	SPT-8		13		13	

Continued Next Page

BORING NUMBER EB-4

PAGE 2 OF 2

PROJECT NAME Block 21, E 3rd Ave & Delaware Mixed-Use Development

PROJECT NUMBER 803-11-1

PROJECT LOCATION San Mateo, CA

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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	UNDRAINED SHEAR STRENGTH, ksf (1.0 2.0 3.0 4.0)
		Well Graded Sand with Clay and Gravel (SW-SC) medium dense to dense, moist, brown, fine to coarse sand, fine subrounded to subangular gravel		☒					
	30		43	SPT-9		14		11	
			31	SPT-10		15			
	35								
			32	SPT-11B		16			>4.5 ○
	40	**Sandy Lean Clay (CL)** hard, moist, reddish brown, fine to coarse sand, some fine subrounded gravel, low to moderate plasticity							
		Bottom of Boring at 40.0 feet.							
	45								
	50								
	55								

BORING NUMBER EB-5

CORNERSTONE EARTH GROUP

PROJECT NAME Block 21, E 3rd Ave & Delaware Mixed-Use Development
PROJECT NUMBER 803-11-1
PROJECT LOCATION San Mateo, CA
DATE STARTED 7/23/21 **DATE COMPLETED** 7/23/21
GROUND ELEVATION **BORING DEPTH** 60 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc
LATITUDE 37.5664° **LONGITUDE** -122.3195°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CRS
▽ **AT TIME OF DRILLING** 22 ft.
NOTES
▼ **AT END OF DRILLING** 20 ft.

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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	UNDRAINED SHEAR STRENGTH, ksf 1.0 2.0 3.0 4.0
	0	1 inch asphalt concrete over 3 inches aggregate base							
		Sandy Lean Clay (CL) very stiff, moist, reddish brown, fine sand, low to moderate plasticity	18	MC-1B	92	9			○
		Clayey Sand with Gravel (SC) very dense to medium dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subrounded to subangular gravel	50/5"	MC-2B	109	14			
	5		55	MC-3B	110	10			
	10	**Well Graded Sand with Clay and Gravel (SW-SC)** medium dense, moist, brown, fine to coarse sand, fine to coarse subrounded to subangular gravel	30	MC-4B	116	8			
	15	becomes dense	57	MC-5B	119	7			
▼	20	becomes medium dense	38	MC-6B	114	14			
▽	25	**Well Graded Gravel with Clay and Sand (GW-GC)** dense, wet, brown, fine to coarse subrounded to subangular gravel, fine to coarse sand	59	MC-7B	111	14			

Continued Next Page

BORING NUMBER EB-5

PROJECT NAME Block 21, E 3rd Ave & Delaware Mixed-Use Development
PROJECT NUMBER 803-11-1
PROJECT LOCATION San Mateo, CA

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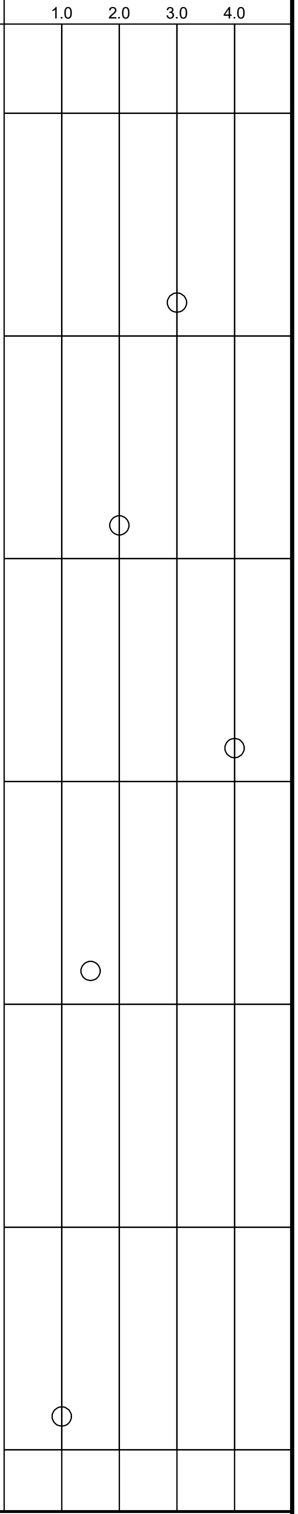
ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
	30		**Clayey Sand with Gravel (SC)** dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subrounded to subangular gravel	44	SPT-8		14		
			Sandy Lean Clay (CL) very stiff, moist, brown, fine to coarse sand, some fine subrounded to subangular gravel, low to moderate plasticity	13	SPT-9		16		
	35								
				26	SPT-10		16		
	40								
				45	SPT-11		18		
	45								
			Lean Clay with Sand (CL) stiff, moist, brown, fine to medium sand, moderate plasticity	35	SPT-12		20		
	50								
			Well Graded Sand with Clay and Gravel (SW-SC) very dense, moist, brown, fine to coarse sand, fine subrounded to subangular gravel	53	SPT-13		15		
	55								
			Sandy Lean Clay (CL) stiff, moist, brown, fine to medium sand, trace fine subrounded to subangular gravel, low to moderate plasticity	30	SPT-14		15		
	60								
			Bottom of Boring at 60.0 feet.						

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



CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 8/24/21 10:19 - P:\DRAFTING\GINT FILES\803-11-1 BLOCK 21.GPJ

BORING NUMBER EB-6

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CORNERSTONE EARTH GROUP

PROJECT NAME Block 21, E 3rd Ave & Delaware Mixed-Use Development
PROJECT NUMBER 803-11-1
PROJECT LOCATION San Mateo, CA
DATE STARTED 7/23/21 **DATE COMPLETED** 7/23/21
GROUND ELEVATION **BORING DEPTH** 40 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc
LATITUDE 37.5664° **LONGITUDE** -122.3196°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CRS
▽ **AT TIME OF DRILLING** 19.5 ft.
NOTES
▼ **AT END OF DRILLING** 19.5 ft.

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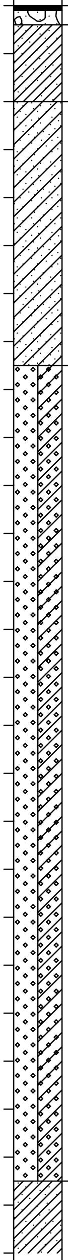
UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0



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	UNDRAINED SHEAR STRENGTH, ksf
	0	1 inch asphalt concrete over 3 inches aggregate base							
		Sandy Lean Clay (CL) hard, moist, reddish brown, fine to medium sand, some fine subrounded to subangular gravel, low to moderate plasticity	73	MC-1B	114	9			>4.5 ○
			50 5"	MC-2	101	11			
		Clayey Sand with Gravel (SC) very dense to medium dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subrounded to subangular gravel							
	5		37	MC-3B	111	8			
		Well Graded Sand with Clay and Gravel (SW-SC) medium dense, moist, brown, fine to coarse sand, fine to coarse subrounded to subangular gravel	39	MC-4B	117	9			
	10								
			49	MC-5B		6			
	15								
▼		becomes wet	40	MC-6B	117	9			
	20								
			13	SPT-7B		25			
	25								

Continued Next Page

BORING NUMBER EB-6

PAGE 2 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME Block 21, E 3rd Ave & Delaware Mixed-Use Development
PROJECT NUMBER 803-11-1
PROJECT LOCATION San Mateo, 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
1.0 2.0 3.0 4.0

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
Sandy Lean Clay (CL) medium stiff, moist, brown, fine to coarse sand, some fine subrounded to subangular gravel, low to moderate plasticity		ST					○
Clayey Sand with Gravel (SC) dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subrounded to subangular gravel	44	SPT-9		14			
Sandy Lean Clay (CL) very stiff, moist, brown, fine to medium sand, some fine subrounded to subangular gravel, low to moderate plasticity	50 6"	SPT-10		17			○
	37	SPT-11		17			○

Bottom of Boring at 40.0 feet.

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.

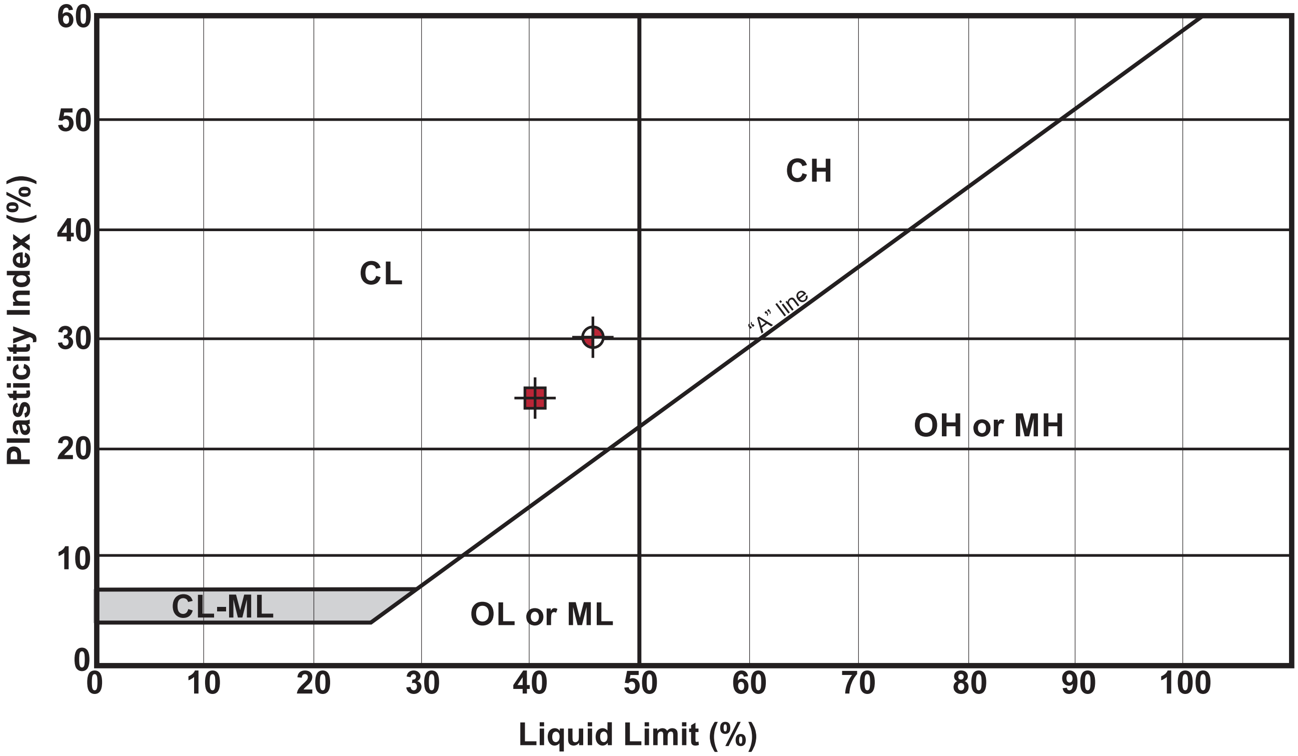
Moisture Content: The natural water content was determined (ASTM D2216) on 75 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 39 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.

Plasticity Index: Two Plasticity Index determinations (ASTM D4318) were performed on samples of the subsurface soils to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index (ASTM D4318) Testing Summary



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
⊕	EB-1	4.0	14	46	16	30	—	Lean Clay with Sand (CL)
■	EB-2	33.5	17	41	17	24	—	Well Graded Sand with Clay (SW-SC) (CL fines)

Samples prepared in accordance with ASTM D421

CORNERSTONE
EARTH GROUP

Plasticity Index Testing Summary

Block 21
East 3rd Avenue and South Delaware Street
Office Development
San Mateo, CA

Project Number
803-11-1

Figure Number
Figure B1

Date
August 2021

Drawn By
FLL