CORNERSTONE
EARTH GROUP

TYPE OF SERVICES Design-Level Geotechnical Investigation

PROJECT NAME 31-57 South B Street Mixed-Use Building

LOCATION 31-57 South B Street
San Mateo, California

CLIENT Harvest Properties, Inc.

PROJECT NUMBER 480-6-1

DATE March 31, 2022

GEOTECHNICAL

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Client	**Harvest Properties, Inc.**
Client Address	**180 Grand Avenue, Suite 1400** **Oakland, California**
Project Number	**480-6-1**
Date	**March 31, 2022**

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SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Harvest Properties, Inc. for the 31-57 South B Street Mixed-Use Building project in San Mateo, California. As you know, we previously prepared a preliminary geotechnical report titled, “Preliminary Geotechnical Investigation, 31-57 South B Street Proposed Mixed-Use Building, 31-57 South B Street, San Mateo, California,” dated June 24, 2019. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- A set of conceptual plans titled “Corner Plaza – Tri-Terrace, Downtown San Mateo, Donut Delite, Massing Studies,” prepared by RMW Architecture, dated July 20, 2021.

1.1 PROJECT DESCRIPTION

We understand the planned project will consist of an at-grade, four-story mixed-use office building. We understand the first floor will consist of retail space and a lobby. The upper three floors will consist of office space with terraces. We anticipate the planned development will be of wood or steel-frame construction. Appurtenant parking, utilities, landscaping and other improvements necessary for site development are also planned.

Structural loads were not available at the time of our report; however, structural loads are expected to be representative of this type of structure. We anticipate cuts and fills will be minor and on the order of 2 to 3 feet.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated March 2, 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 two borings drilled on February 16, 2022 with truck-mounted, hollow-stem auger drilling equipment. The borings were drilled to depths ranging from 40 to 59½ feet. The borings were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions. The approximate locations of our exploratory borings are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

1.4 PREVIOUS FIELD EXPLORATION

Our previous field exploration consisted of two borings drilled on May 31, 2019 with truck-mounted, hollow-stem auger drilling equipment. The borings were drilled to depths ranging from 40 to 80 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 previous exploratory borings are also shown on the Site Plan, Figure 2. Details regarding our previous field program are included in Appendix A.

1.5 LABORATORY TESTING PROGRAM

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

1.6 CORROSION EVALUATION

Two samples from our previous borings from depths of 5½ to 19 feet were tested for saturated resistivity, pH, and soluble sulfates and chlorides. In general, the on-site soils can be characterized as moderately to severely corrosive to buried metal, and not corrosive to buried concrete.

1.7 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

The San Francisco peninsula is a relatively narrow band of rock at the north end of the Santa Cruz Mountains separating the Pacific Ocean from San Francisco Bay. This represents one mountain range in a series of northwesterly-aligned mountains forming the Coast Ranges 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. Locally these basement rocks are capped by younger sedimentary and volcanic rocks. Most of the Coast Ranges are covered by still younger surficial deposits that reflect geologic conditions of the last million years or so.

Movement on the many splays within 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 branch faults are 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 no being identified also.

2.2 REGIONAL SEISMICITY

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

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

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
San Andreas (1906)	3.5	5.6
Monte Vista-Shannon	9.9	16.0
San Gregorio	10.4	16.7
Hayward (Total Length)	14.9	23.9

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 SURFACE DESCRIPTION

The approximately 13,860 square foot project site is located at 31 to 57 South B Street in San Mateo, California. The site is comprised of several parcels and is currently occupied by several one-story retail buildings and common driveway/parking areas. The site is relatively level ranging from about Elevation 29 to 30 feet based on Google Earth. The site is bounded by a private alley and San Mateo Transit Center to the northeast, First Street to the southeast, South B Street to the southwest, and commercial development to the northwest.

Surface pavements generally consisted of 6 to 8 inches of Portland cement concrete over 2 to 6 inches of aggregate base. Boring EB-3 (within South B Street) also encountered 3 inches of asphalt concrete over the Portland cement concrete. Based on visual observations, the existing pavements are in fair to poor condition.

3.2 SUBSURFACE CONDITIONS

Below the surface pavements, our explorations generally encountered 1½ to 4½ feet of undocumented fill consisting of loose to medium dense clayey sands with varying amounts of gravel and very stiff lean clays with varying amounts of sand. Beneath the undocumented fills, our explorations primarily encountered interbedded layers of stiff to hard lean clays with varying amounts of sand and medium dense to very dense clayey sands with varying amounts of gravel to the maximum depth explored of 80 feet. Boring EB-1 also encountered a layer of medium dense poorly graded sand with silt between about 16 to 20 feet and layers of very dense poorly graded sand with clay between about 61 to 67½ feet and about 77 to 80 feet. Boring EB-3 also encountered a layer of medium dense silty sand between about 22½ to 25 feet.

3.2.1 Plasticity/Expansion Potential

We performed three Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of surficial soils, and the plasticity of the fines in potentially liquefiable layers. The results of the surficial PI tests both indicated a PI of 9, indicating low expansion potential to wetting and drying cycles. The results of the PI tests in the potentially liquefiable layers indicated a PI of 18.

3.2.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from about optimum to about 8 percent over the estimated laboratory optimum moisture.

3.3 GROUNDWATER

Groundwater was encountered in our explorations at depths ranging from 21 to 24 feet below current grades. All measurements were taken at the time of drilling and may not represent the stabilized levels that can be higher than the initial levels encountered.

Based on our previous explorations in the area and groundwater data reported on GeoTracker, we estimate stabilized levels of groundwater are estimated to be on the order of 14 to 15 feet below existing grade. Mapping by CGS (2018) indicates the historic high groundwater in the area of the site is estimated to be between 12 to 15 feet; however, the data provided is very difficult to interpolate. We recommend a high groundwater level of 14 feet be used for design. We note this value does not include any additional “free” board to account for potential future fluctuations of the groundwater depth.

Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

3.4 CORROSION SCREENING

We tested two samples collected at depths of 5½ and 19 feet for resistivity, pH, soluble sulfates, and chlorides. The laboratory test results are summarized in Table 2A.

Table 2A: Summary of Corrosion Test Results

Sample Location	Depth (feet)	Soil pH[1]	Resistivity[2] (ohm-cm)	Chloride[3] (mg/kg)	Sulfate[4,5] (mg/kg)
EB-1 / 3A	5½	6.8	1,106	5	119
EB-2 / 6A	19	7.5	4,812	7	17

Notes:
[1] ASTM G51
[2] ASTM G57 - 100% saturation
[3] ASTM D3427/Cal 422 Modified
[4] ASTM D3427/Cal 417 Modified
[5] 1 mg/kg = 0.0001 % by dry weight

Many factors can affect the corrosion potential of soil including moisture content, resistivity, permeability, and pH, as well as chloride and sulfate concentration. Typically, soil resistivity, which is a measurement of how easily electrical current flows through a medium (soil and/or water), is the most influential factor. In addition to soil resistivity, chloride and sulfate ion concentrations, and pH also contribute in affecting corrosion potential.

3.4.1 Preliminary Soil Corrosion Screening

Based on the laboratory test results summarized in Table 2A and published correlations between resistivity and corrosion potential, the soils may be considered moderately to severely corrosive to buried metallic improvements (Chaker and Palmer, 1989).

In accordance with the 2019 CBC Section 1904.1, alternative cementitious materials for different exposure categories and classes shall be determined in accordance with ACI 318-19 Table 19.3.1.1, Table R19.3.1, and Table 19.3.2.1. Based on the laboratory sulfate test results, a cement type restriction is not required, although, in our opinion, it is generally a good idea to include some sulfate resistance and to maintain a relatively low water-cement ratio. We have summarized applicable exposure categories and classes from ACI 318-19, Table 19.3.1.1 below in Table 2B.

Table 2B: ACI 318-19 Table 19.3.1.1 Exposure Categories and Classes

Freezing and Thawing (F)	Sulfate (S, soil)	In Contact with Water (W)	Corrosion Protection of Reinforcement (C)
F0[1]	S0[2]	W0[3]	C0[4]

[1] (F0) “Concrete not exposed to freezing-and-thawing cycles” (ACI 318-19)
[2] (S0) “Water soluble sulfate in soil, percent by mass” (ACI 318-19)
[3] (W0) “Concrete dry in service” (ACI 318-19)
[4] (C0) “Concrete dry or protected from moisture” (ACI 318-19)

We recommend the structural engineer and a corrosion engineer be retained to confirm the above information and provide additional recommendations, as needed.

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 northwestern side of the site is within a State-designated Liquefaction Hazard Zone (CGS, San Mateo Quadrangle, 2018). Our field and laboratory programs addressed this issue by testing and sampling potentially liquefiable layers to depths of at least 50 feet, performing visual classification on sampled materials, and performing various tests to further classify 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 groundwater depth of 14 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 blow counts and laboratory testing on samples retrieved from our borings. SPT "N" values obtained from hollow-stem auger borings were used in our analyses.

The results of our analyses (EB-1 through EB-4) are presented on Figures 4A through 4D of this report.

4.3.3 Summary

Our analyses indicate that several layers could potentially experience liquefaction triggering that could result in post-liquefaction total settlement at the ground surface ranging from about ⅓ inch to about 1¼ inches based on the Yoshimine (2006) method. As discussed in SP 117A, differential movement for level ground sites over deep soil sites will be up to about two-thirds of the total settlement between independent foundation elements. In our opinion, differential settlements are anticipated to be on the order of less than 1 inch over a horizontal distance of 30 to 40 feet.

4.3.4 Ground Deformation and Surficial Cracking Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground deformation or sand boils. For ground deformation to occur, the pore water pressure within the liquefiable soil layer will need to be great enough to break through the overlying non-liquefiable layer, which could cause significant ground deformation and settlement. The work of Youd and Garris (1995) indicates that the 14-foot-thick layer of non-liquefiable cap is sufficient to prevent ground deformation and significant surficial cracking; therefore, the above total settlement estimates are reasonable.

4.4 LATERAL SPREADING

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

Based on our experience in the area, we understand that an underground culvert is present along North B Street and Baldwin Avenue to the northwest of the project site. Based on our review of aerial images, the nearest point of the culvert is about 60 to 65 feet from the site. Based on our review of available maps and photos it appears the culvert runs along the northeast side of B Street and turns to run along the southeast side of Baldwin Avenue prior to reaching the project site. We understand the culvert is a concrete box type with an open bottom positioned about 12 to 14 feet below the ground surface. As part of our liquefaction analyses, we calculated the Lateral Displacement Index (LDI) for potentially liquefiable layers based on methods presented in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008). LDI is a summation of the maximum shear strains versus depth, which is a measurement of the potential maximum displacement at that exploration location. Summations of the LDI values to a depth equal to twice the open face height were included. Estimated displacements in the vicinity of EB-1 through EB-4 based on the LDI calculations are on the order of a few to several inches.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. We evaluated the potential for seismic compaction of the sands above the design groundwater depth based on the work by Pradel (1998). Our analyses indicate that the proposed building could experience on the order of ¼ inch or less of movement after strong seismic shaking.

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 29 to 30 feet above mean sea level based on Google Earth. In addition, the site is mapped by the State of California as being outside a tsunami hazard area (CGS, 2021). 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, described as “areas of minimal flood hazard.” We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

The Department of Water Resources (DWR), Division of Safety of Dams (DSOD) compiled a database of Dam Failure Inundation Hazard Maps (DSOD, 2015). The generalized hazard maps were prepared by dam owners as required by the State Office of Emergency Services; they are intended for planning purposes only. Based on our review of these maps, the site is located within a dam failure inundation area for the Lower Crystal Springs Reservoir. 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
- Redevelopment considerations
- Presence of undocumented fill
- Soil corrosion potential

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 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 ¼ inch to 1¼ inches could occur, resulting in differential settlement on the order of less than 1 inch over a horizontal distance of 30 to 40 feet. Foundations should be designed to tolerate the anticipated total and differential settlements. Based on our estimated foundation loads, it should be feasible to support the proposed buildings on shallow foundations; however, the building foundations will need to be designed to tolerate total and differential settlement due to static loads and liquefaction-induced settlement. Detailed foundation recommendations are presented in the “Foundations” section.

5.1.2 Redevelopment Considerations

As discussed, the site is currently occupied by existing buildings and appurtenant flatwork, site fixtures, and landscaping. We understand that all of the existing improvements will be demolished for the construction of the building additions. Potential issues that are often associated with redeveloping sites include demolition of existing improvements which may include old basements or sump pits, abandonment of existing utilities, and undocumented fills. Please refer to the “Earthwork” section below for further recommendations.

5.1.3 Undocumented Fill

As previously discussed, up to about 4½ feet of undocumented fills were encountered in our exploratory borings. Additional undocumented fill may be present as a result of prior development grading. To reduce the potential for differential settlement, undocumented fills encountered during site grading 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. 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.

Provided undocumented fills are mitigated by removal and replacement as engineered fill, the potential impact due to undocumented fill should be low.

5.1.4 Soil Corrosion Potential

As discussed, we performed a preliminary soil corrosion screening based on the results of analytical tests on samples of the near-surface soil. In general, the corrosion potential of buried concrete does not warrant the use of sulfate resistance concrete; however, the corrosion potential for buried metallic structures, such as metal pipes, is considered moderately to severely corrosive. As the preliminary soil corrosion screening was based on the results of limited sampling, consideration may be given to collecting additional samples, as well as hiring a corrosion engineer, to confirm the classifications.

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, which are currently present on the site, prior to the start of mass grading or the construction of new improvements for the project.

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

6.1.1 Demolition of Existing Slabs, Foundations and Pavements

All slabs, foundations, and pavements should be completely removed from within planned building areas.

As an owner value-engineered option, existing slabs, foundations, and pavements that extend into planned flatwork, pavement, or landscape areas may be left in place provided there is at least 3 feet of engineered fill overlying the remaining materials, they are shown not to conflict with new utilities, and that asphalt and concrete more than 10 feet square is broken up to allow subsurface drainage. Future distress and/or higher maintenance may result from leaving these prior improvements in place. A discussion of recycling existing improvements is provided later in this report.

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

Existing foundations are typically mat-slabs, shallow footings, or piers/piles. If slab or shallow footings are encountered, they should be completely removed. If drilled piers are encountered, they should be cut off at an elevation at least 36-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. A detailed discussion of removal of existing fills is provided later in this report. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight.

6.2.2 Tree and Shrub Removal

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

6.3 MITIGATION OF UNDOCUMENTED FILLS

As discussed above, our exploratory borings encountered up to approximately 4½ feet of undocumented fill. In addition, due to existing site development, additional fills may be present. All undocumented fills should be over-excavated and re-compacted 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 10 feet at the site may be classified as OSHA Soil Type C materials.

Excavations performed during site demolition and fill removal should be sloped at 3:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Actual excavation inclinations should be reviewed in the field during construction, as needed. Excavations below building subgrade and excavations in pavement and flatwork areas should be sloped in accordance with OSHA soil classification requirements.

6.5 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.

6.6 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 up to about 8 percent over the estimated laboratory optimum in the upper 10 feet of the soil profile. The contractor should anticipate drying the soils prior to reusing them as fill. In addition, 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.

6.6.1 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.

6.6.2 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.

6.6.3 Chemical Treatment

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

6.7 MATERIAL FOR FILL

6.7.1 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.

6.7.2 Re-Use of On-Site Site Improvements

We anticipate that minor to moderate quantities of Portland concrete cement (PCC) grindings and aggregate base (AB) will be generated during site demolition. If the site area allows for on-site pulverization of PCC and provided the PCC is pulverized to meet the “Material for Fill” requirements of this report, it may be used as select fill within the proposed mixed-use 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.

6.7.3 Potential Import Sources

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.

6.8 COMPACTION REQUIREMENTS

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

Table 3: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Soils	90	>1
Trench Backfill	On-Site Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site 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 Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	NA

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

6.9 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.

6.10 SITE DRAINAGE

Ponding should not be allowed adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 2 percent towards suitable discharge facilities; landscape areas should slope at least 3 percent towards suitable discharge facilities. Roof runoff should be directed away from building areas in closed conduits, to approved infiltration facilities, or on to hardscaped surfaces that drain to suitable facilities. Retention, detention or infiltration facilities should be spaced at least 10 feet from buildings, and preferably at least 5 feet from slabs-on-grade or pavements. However, if retention, detention or infiltration facilities are located within these zones, we recommend that these treatment facilities meet the requirements in the Storm Water Treatment Design Considerations section of this report.

6.11 LOW-IMPACT DEVELOPMENT (LID) IMPROVEMENTS

The Municipal Regional Permit (MRP) requires regulated projects to treat 100 percent of the amount of runoff identified in Provision C.3.d from a regulated project’s drainage area with low impact development (LID) treatment measures onsite or at a joint stormwater treatment facility. LID treatment measures are defined as rainwater harvesting and use, infiltration, evapotranspiration, or biotreatment. A biotreatment system may only be used if it is infeasible to implement harvesting and use, infiltration, or evapotranspiration at a project site.

Technical infeasibility of infiltration may result from site conditions that restrict the operability of infiltration measures and devices. Various factors affecting the feasibility of infiltration treatment may create an environmental risk, structural stability risk, or physically restrict infiltration. The presence of any of these limiting factors may render infiltration technically infeasible for a proposed project. To aid in determining if infiltration may be feasible at the site, we provide the following site information regarding factors that may aid in determining the feasibility of infiltration facilities at the site.

- The near-surface soils at the site are clayey and categorized as Hydrologic Soil Group D, and is expected to have infiltration rates of less than 0.2 inches per hour. In our opinion, these clayey soils will significantly limit the infiltration of stormwater.

- Locally, seasonal high groundwater is mapped at a depth of about 12 to 15 feet, and therefore may be within 10 feet of the base of the infiltration measure.

6.11.1 Storm Water Treatment Design Considerations

If storm water treatment improvements, such as shallow bio-retention swales, basins or pervious pavements, are required as part of the site improvements to satisfy Storm Water

Quality (C.3) requirements, we recommend the following items be considered for design and construction.

6.11.1.1 General Bioswale Design Guidelines

- If possible, avoid placing bioswales or basins within 10 feet of the building perimeter or within 5 feet of exterior flatwork or pavements. If bioswales must be constructed within these setbacks, the side(s) and bottom of the trench excavation should be lined with 10-mil visqueen to reduce water infiltration into the surrounding expansive clay.

- Bioswales constructed within 3 feet of proposed buildings may be within the foundation zone of influence for perimeter wall loads. Therefore, where bioswales will parallel foundations and will extend below the “foundation plane of influence,” an imaginary 1:1 plane projected down from the bottom edge of the foundation, the foundation will need to be deepened so that the bottom edge of the bioswale filter material is above the foundation plane of influence.

- The bottom of bioswale or detention areas should include a perforated drain placed at a low point, such as a shallow trench or sloped bottom, to reduce water infiltration into the surrounding soils near structural improvements, and to address the low infiltration capacity of the on-site clay soils.

6.11.1.2 Bioswale Infiltration Material

- Gradation specifications for bioswale filter material, if required, should be specified on the grading and improvement plans.

- Compaction requirements for bioswale filter material in non-landscaped areas or in pervious pavement areas, if any, should be indicated on the plans and specifications to satisfy the anticipated use of the infiltration area.

- If bioswales are to be vegetated, the landscape architect should select planting materials that do not reduce or inhibit the water infiltration rate, such as covering the bioswale with grass sod containing a clayey soil base.

- Due to the relatively loose consistency and/or high organic content of many bioswale filter materials, long-term settlement of the bioswale medium should be anticipated. To reduce initial volume loss, bioswale filter material should be wetted in 12-inch lifts during placement to pre-consolidate the material. Mechanical compaction should not be allowed, unless specified on the grading and improvement plans, since this could significantly decrease the infiltration rate of the bioswale materials.

- It should be noted that the volume of bioswale filter material may decrease over time depending on the organic content of the material. Additional filter material may need to be added to bioswales after the initial exposure to winter rains and periodically over the life of the bioswale areas, as needed.

6.11.1.3 Bioswale Construction Adjacent to Pavements

If bio-infiltration swales or basins are considered adjacent to proposed parking lots or exterior flatwork, we recommend that mitigative measures be considered in the design and construction of these facilities to reduce potential impacts to flatwork or pavements. Exterior flatwork, concrete curbs, and pavements located directly adjacent to bio-swales may be susceptible to settlement or lateral movement, depending on the configuration of the bioswale and the setback between the improvements and edge of the swale. To reduce the potential for distress to these improvements due to vertical or lateral movement, the following options should be considered by the project civil engineer:

- Improvements should be setback from the vertical edge of a bioswale such that there is at least 1 foot of horizontal distance between the edge of improvements and the top edge of the bioswale excavation for every 1 foot of vertical bioswale depth, or

- Concrete curbs for pavements, or lateral restraint for exterior flatwork, located directly adjacent to a vertical bioswale cut should be designed to resist lateral earth pressures in accordance with the recommendations in the “Retaining Walls” section of this report, or concrete curbs or edge restraint should be adequately keyed into the native soil or engineered to reduce the potential for rotation or lateral movement of the curbs.

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 37.567425° and longitude -122.324072°, which is based on Google Earth (WGS84) coordinates at the approximate center of the project site located at 31-57 South B Street 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

As discussed in the “Subsurface” of our report, our exploratory borings encountered medium dense to very dense sands and stiff to hard clay deposits to a depth of 80 feet, the maximum depth explored. Based on our review of local geology and soils encountered, an estimated time-averaged shear wave velocity for the top 30 meters (V_{S30}) of 308 meters per second (1011 feet per second), for the upper 100 feet based on correlated SPT “N” values was used for our analysis.

7.2.1 2019 CBC Seismic Design

As our borings encountered deep alluvial soils with typical SPT "N" values between 15 and 50 blows per foot, per section 20.3.2 of ASCE 7-16, we have classified the site as Soil Classification D, which is described as a "stiff soil" profile, with a correlated/estimated shear wave velocity for the upper 30 meters (V_{S30}) of 308 m/s (1011 ft/s) based on our exploratory boring SPT blow counts. 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. We used a correlated/estimated V_{S30} of 308 m/s (1011 ft/s) for our site-specific ground motion hazard analysis.

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 Deterministic MCE_R Ground Motions to generate our recommended design response spectrum for the project, see Figure 5.

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed structures may be supported on shallow foundations provided the recommendations in the "Earthwork" section and the sections below are followed.

8.2 SHALLOW FOUNDATIONS

8.2.1 Conventional Shallow Footings

Conventional shallow footings should bear on natural, undisturbed soil or engineered fill, be at least 18 inches wide, and extend at least 18 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil.

Footings constructed to the above dimensions and in accordance with the "Earthwork" recommendations of this report are capable of supporting maximum allowable bearing pressures of 2,000 psf for dead loads, 3,000 psf for combined dead plus live loads, and 4,000 psf for all loads including wind and seismic. These pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected for the portion of the footing extending below grade (typically, the full footing depth). Top and bottom mats of reinforcing steel should be included in continuous footings to help span irregularities and differential settlement.

8.2.2 Footing Settlement

Structural loads were not provided to us at the time this report was prepared; therefore, we assumed the typical loading in the following table.

Table 4: Assumed Structural Loading

Foundation Area	Range of Assumed Loads
Interior Isolated Column Footing	200 to 400 kips
Exterior Isolated Column Footing	50 to 150 kips
Perimeter Strip Footing	5 to 7 kips per lineal foot

Based on the above loading and the allowable bearing pressures presented above, we estimate that the total static footing settlement will be on the order of ¾-inch or less, with about ⅓-inch or less of post-construction differential settlement between adjacent foundation elements. In addition, we estimate that differential seismic movement will be on the order of ¼ inch to less than 1 inch over a horizontal distance of 30 to 40 feet, resulting in a total estimated differential footing movement of on the order of about 1 to 1¼ inch or less between foundation elements, assumed to be on the order of 30 feet. As our footing loads were assumed, we recommend we be retained to review the final footing layout and loading and verify the settlement estimates above.

8.2.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of footing and the supporting subgrade, and also by passive pressures generated against footing sidewalls. An ultimate frictional resistance of 0.40 applied to the footing 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. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

8.2.4 Conventional Shallow Footing Construction Considerations

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.

Footing excavations should be filled as soon as possible or be kept moist until concrete placement by regular sprinkling to prevent desiccation. A Cornerstone representative should observe all footing excavations prior to placing reinforcing steel and concrete. If there is a significant schedule delay between our initial observation and concrete placement, we may need to re-observe the excavations.

Due to the presence of localized clayey sand with varying amounts of fines, footing excavation walls may not stand vertical and may need to be sloped to a minimum 1:1 inclination or Stay-Form or similar may need to be placed within the footing excavations as they are excavated during construction of the foundation elements. Granular material encountered in the footing bottoms will likely be disturbed to a depth of 6 to 8 inches following excavation and will need to be compacted to 90 percent relative compaction prior to steel placement. Care should be taken to not disturb the compacted granular material during steel placement. We should re-observe the footing excavations in granular materials after reinforcing steel has been placed and just prior to concrete placement. Footing excavations should also be kept moist by regular sprinkling with water to prevent desiccation and potential raveling of the granular materials. As an alternative, a rat slab can be placed over the granular material after we have observed the footing excavation to protect the granular material prior to steel placement.

8.2.5 Reinforced Concrete Mat Foundations

As an alternative to conventional shallow footings, the proposed structure may be supported on a mat foundation bearing 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.

For our analysis, we assumed an average allowable bearing pressure of 600 to 750 psf for dead plus live loads; at column or wall loading, the maximum localized bearing pressure should be limited to 3,000 psf. 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.

8.2.6 Mat Foundation Settlement

As discussed above, for our analysis we assumed average areal bearing pressures of about 600 to 750 psf for dead plus live loads across the mat foundation. Based on the assumed bearing pressures, we estimate the total static settlements for the mixed-use structure would be on the order of about ⅔ to 1 inch near the center of the mat and on the order of ¼ inch or less at the mat edges and corner. Differential seismic settlements on the order of ¼ inch to less than 1 inch across a horizontal distance of 30 to 40 feet may occur anywhere within the mat. Accounting for both seismic induced and static differential settlement, the mat foundation may experience combined static and seismic differential settlements on the order of about 1 to 1¼ inches. As our loads were assumed, we recommend we be retained to review the final foundation plan and loading and verify the settlement estimates above.

8.2.7 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 0.40 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.

8.2.8 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 600 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 10 pounds per cubic inch (pci) for the center of the mat foundation and 20 pounds per cubic inch (pci) at the edges and corners. 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.

8.2.9 Mat Foundation Construction Considerations

Prior to placement of any vapor retarder and mat construction, the subgrade should be proof-rolled and visually observed by a Cornerstone representative to confirm stable subgrade conditions.

SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

9.1 INTERIOR SLABS-ON-GRADE

As the Plasticity Index (PI) of the surficial soils is 15 or less, the proposed slabs-on-grade may be supported directly on subgrade prepared in accordance with the recommendations in the "Earthwork" section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the "Interior Slabs Moisture Protection Considerations" section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and slab-on-grade construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to near optimum moisture content.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying soils. For unreinforced concrete slabs, ACI 302.1R recommends limiting control joint spacing to 24 to 36 times the slab thickness in each direction, or a maximum of 18 feet.

9.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

The following general guidelines for concrete slab-on-grade construction where floor coverings are planned are presented for the consideration by the developer, design team, and contractor. These guidelines are based on information obtained from a variety of sources, including the American Concrete Institute (ACI) and are intended to reduce the potential for moisture-related problems causing floor covering failures, and may be supplemented as necessary based on project-specific requirements. The application of these guidelines or not will not affect the geotechnical aspects of the slab-on-grade performance.

- Place a minimum 15-mil vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab; the vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements. A 4-inch-thick capillary break, consisting of crushed rock should be placed below the vapor retarder and consolidated in place with vibratory equipment. The mineral aggregate shall be of such size that the percentage composition by dry weight as determined by laboratory sieves will conform to the following gradation:

Sieve Size	Percentage Passing Sieve
1"	100
¾"	90 – 100
No. 4	0 – 10
No. 200	0 – 5

- The concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement.

- Water should not be added after initial batching unless the slump is less than specified and/or the resulting water:cement ratio will not exceed 0.45.

- Polishing the concrete surface with metal trowels is not recommended.

- Where floor coverings are planned, all concrete surfaces should be properly cured.

- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869-98 and F710-98 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

9.3 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 4 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below. 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: VEHICULAR PAVEMENTS

10.1 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. The design R-value was chosen based on engineering judgement considering the variable and clayey soil conditions.

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	12.5	16.0
6.5	4.0	14.0	18.0

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

Frequently, the full asphalt concrete section is not constructed prior to construction traffic loading. This can result in significant loss of asphalt concrete layer life, rutting, or other pavement failures. To improve the pavement life and reduce the potential for pavement distress through construction, we recommend the full design asphalt concrete section be constructed prior to construction traffic loading. Alternatively, a higher traffic index may be chosen for the areas where construction traffic will use the pavements.

10.2 PORTLAND CEMENT CONCRETE

The Portland Cement Concrete (PCC) pavement recommendations outlined below are based on methods presented in American Concrete Pavement Association (ACPA, 2006). We have provided a few pavement alternatives as an anticipated Average Daily Truck Traffic (ADTT) was not provided.

The following table presents minimum PCC pavements thicknesses for various traffic loading categories and the anticipated maximum Average Daily Truck Traffic (ADTT).

Table 6: PCC Pavement Recommendations

Traffic Category	Minimum PCC Thickness[1] (inches)	Class 2 Aggregate Base (inches)
Maximum ADTT = 10	5.5	6.0
Maximum ADTT = 20	6.0	6.0
Maximum ADTT = 50	6.5	6.0

[1] Subgrade design R-Value = 5

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi, and laterally restraining the PCC 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.

10.2.1 Stress Pads for Trash Enclosures

Pads where trash containers will be stored, and where garbage trucks will park while emptying trash containers, should be constructed on Portland Cement Concrete. We recommend that the trash enclosure pads and stress (landing) pads where garbage trucks will store, pick up, and empty trash be increased to a minimum PCC thickness of 7 inches. The compressive strength, underlayment, and construction details should be consistent with the above recommendations for PCC pavements.

10.3 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.

SECTION 11: RETAINING WALLS

11.1 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. 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 7: Recommended Lateral Earth Pressures

Wall Condition	Lateral Earth Pressure*	Additional Surcharge Loads
Unrestrained – Cantilever Wall	40 pcf	⅓ of vertical loads at top of wall
Restrained – Braced Wall	40 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

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.

11.2 SEISMIC LATERAL EARTH PRESSURES

11.2.1 Site Walls

The 2019 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. At this time, we are not aware of any retaining walls for the project. However, minor landscaping walls (i.e. walls 6 feet or less in height) may be proposed. In our opinion, design of these walls for seismic lateral earth pressures in addition to static earth pressures is not warranted.

11.3 WALL DRAINAGE

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.

11.4 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.

11.5 FOUNDATIONS

Retaining walls may be supported on a continuous and or spread footing designed in accordance with the recommendations presented in the “Foundations” section of this report.

SECTION 12: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Harvest Properties, Inc. specifically to support the design of the 31-57 South B Street Mixed-Use Building 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 groundwater conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

Harvest Properties, Inc. may have provided Cornerstone with plans, reports and other documents prepared by others. Harvest Properties, Inc. 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 13: REFERENCES

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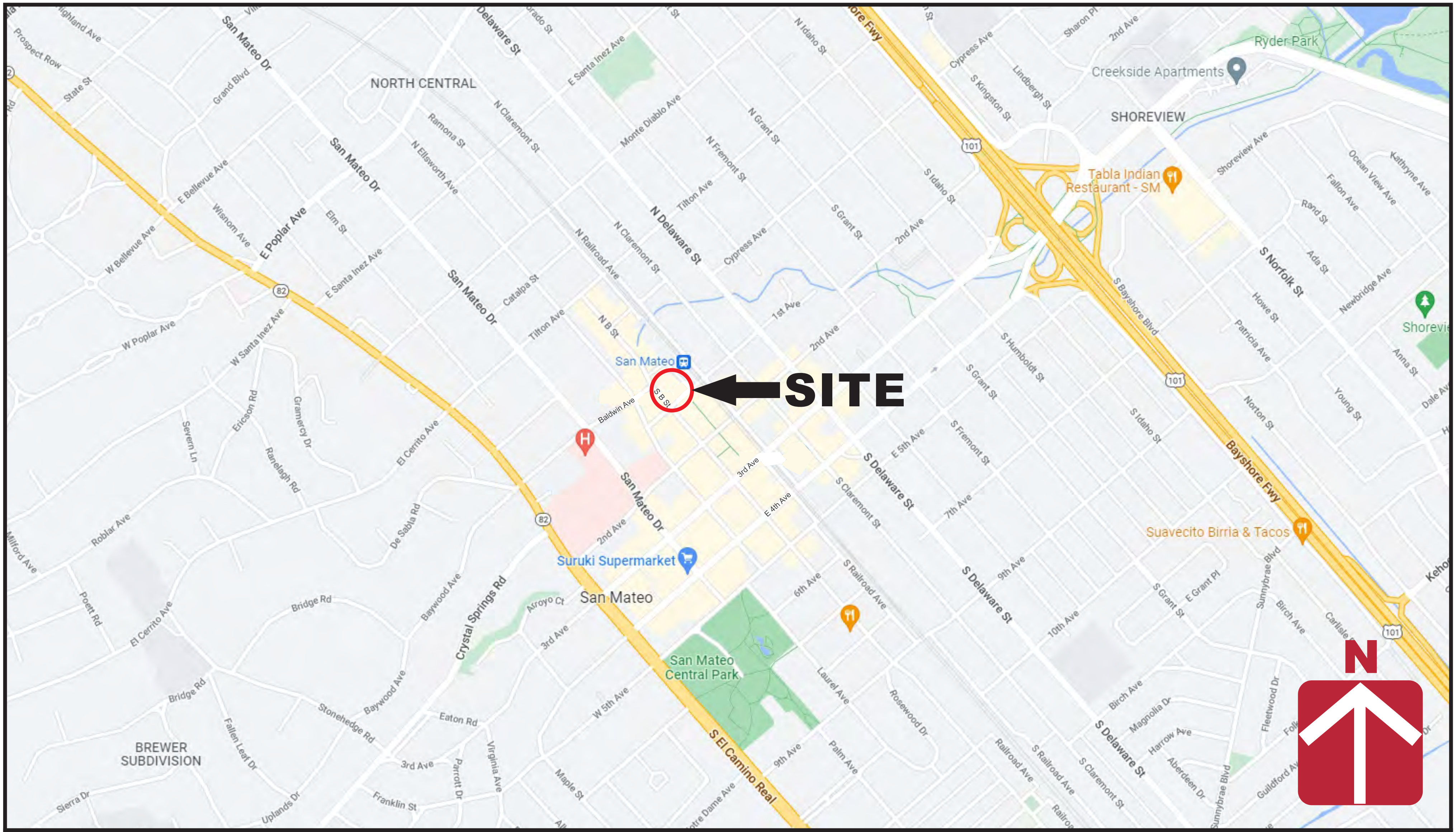


Vicinity Map

CORNERSTONE EARTH GROUP

31 to 57 South B Street
San Mateo, CA

Project Number
480-6-1

Figure Number
Figure 1

Date
March 2022

Drawn By
RRN

Legend

⌖ Approximate location of exploratory boring (EB) (Cornerstone, June 2019)

⌖ Approximate location of exploratory boring (EB) (Cornerstone, current investigation)

0 20 40
APPROXIMATE SCALE (FEET)

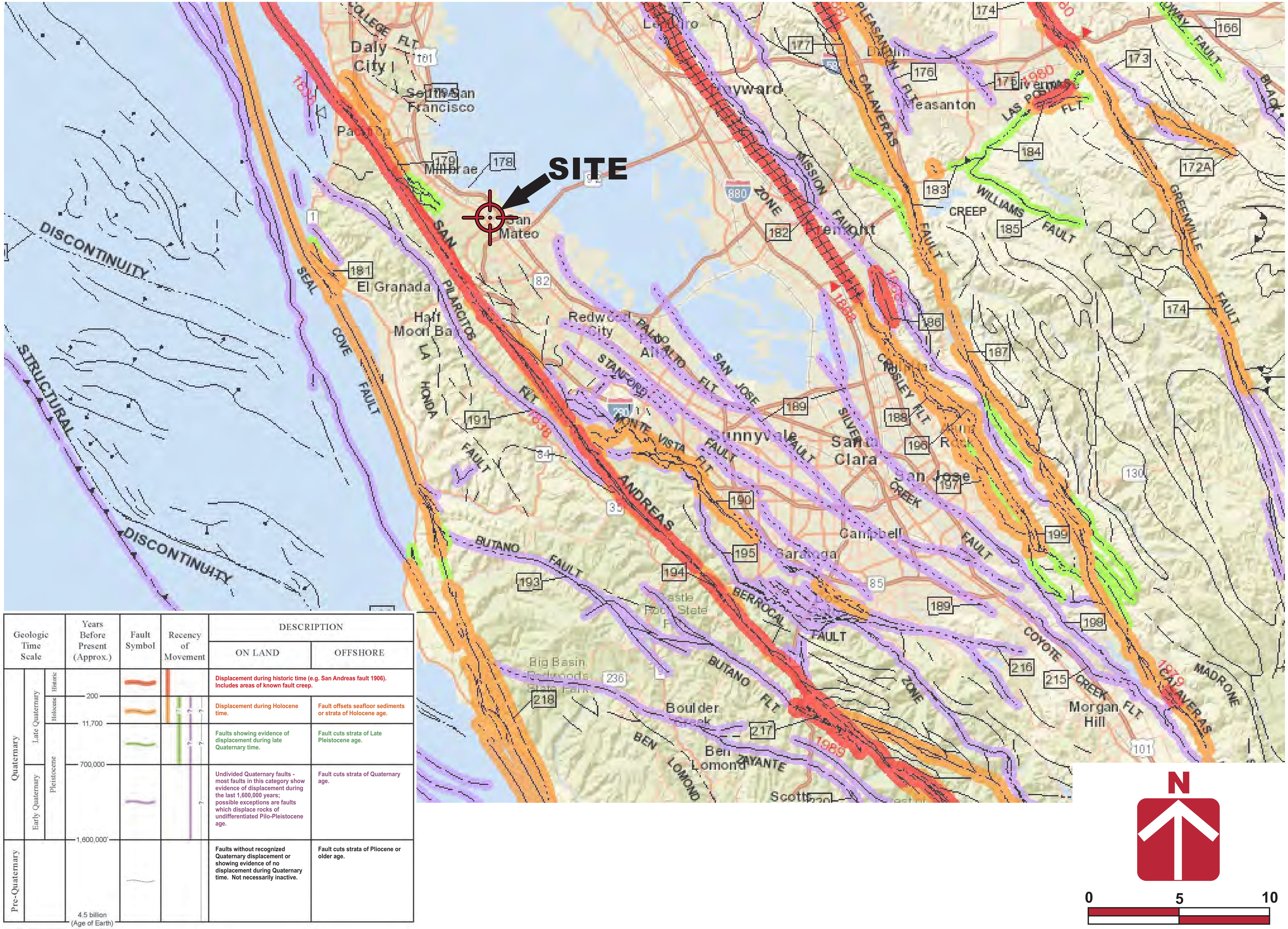
Base by Google Earth, dated 09/26/2020

Project Number	480-6-1
Figure Number	Figure 2
Date	March 2022
Drawn By	RRN

Site Plan

31 to 57 South B Street
San Mateo, CA

CORNERSTONE
EARTH GROUP



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)

Project Number
480-6-1

Figure Number
Figure 3

Date
March 2022

Drawn By
RRN

Regional Fault Map

31 to 57 South B Street
San Mateo, CA

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Boring **EB-1**

SPT ANALYSIS RESULTS

PROJECT INFORMATION

Project Title: **31-57 South B Street Prelim**

Project No.: **307-25-1**

Project Manager: **MFR**

SEISMIC PARAMETERS

Controlling Fault: **San Andreas**

Earthquake Magnitude (Mw): **7.6**

PGA (Amax): **0.9** (g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet): **22**

Design Water Depth (feet): **14**

Ave. Unit Weight Above GW (pcf): **125**

Ave. Unit Weight Below GW (pcf): **135**

MAGNITUDE SCALING FACTOR

0.974

CUMULATIVE SETTLEMENT FROM 50 FEET

0.79 (Inches)

LATERAL DISPLACEMENT

^{2}LDI 0.14 L/H 12.9

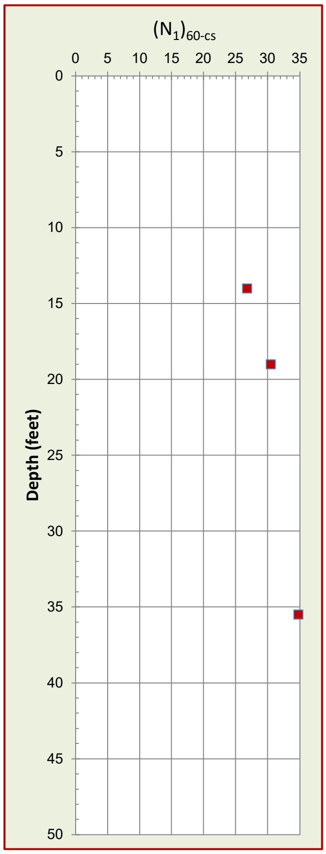
LDI$^1_{\text{Corrected for Distance}}$ 0.11 (4 < L/H < 40)

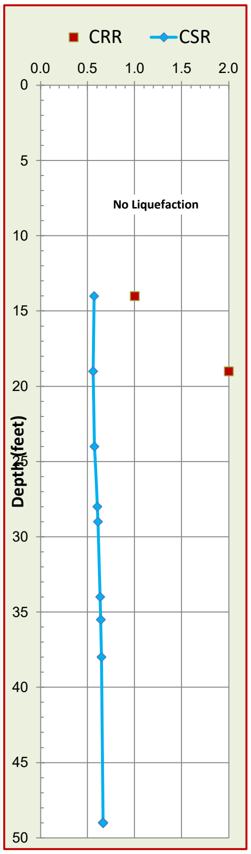
EXPECTED RANGE OF DISPLACEMENT

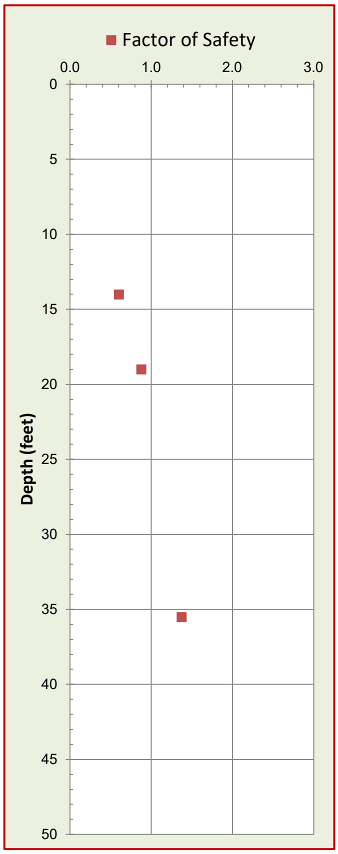
0.1 to 0.2 feet

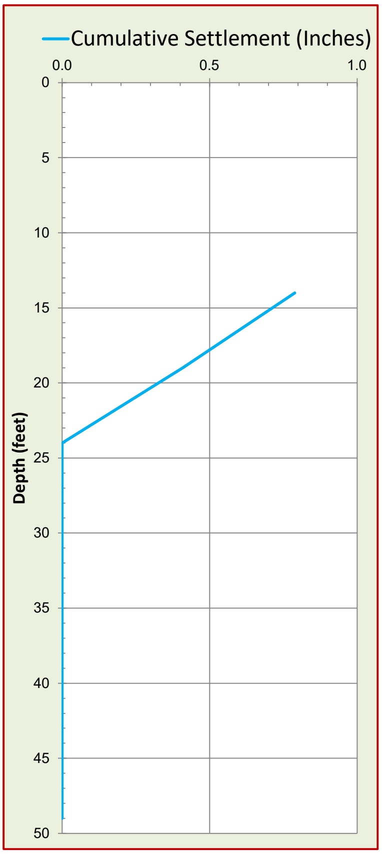
[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.









CORNERSTONE
EARTH GROUP

Boring **EB-2**

PROJECT INFORMATION

Project Title **31-57 South B Street Prelim**

Project No. **307-25-1**

Project Manager **MFR**

SPT ANALYSIS RESULTS

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.6**

PGA (Amax) **0.9** (g)

MAGNITUDE SCALING FACTOR

0.974

CUMULATIVE SETTLEMENT FROM 50 FEET

1.26 **(Inches)**

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **22**

Design Water Depth (feet) **14**

Ave. Unit Weight Above GW (pcf) **125**

Ave. Unit Weight Below GW (pcf) **135**

LATERAL DISPLACEMENT

^{2}LDI 0.25 **L/H** 8.6

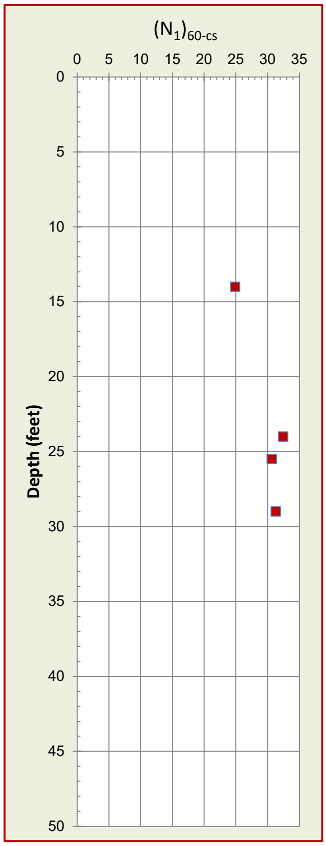
LDI^{1}$_{\text{Corrected for Distance}}$ 0.27 (4 < L/H < 40)

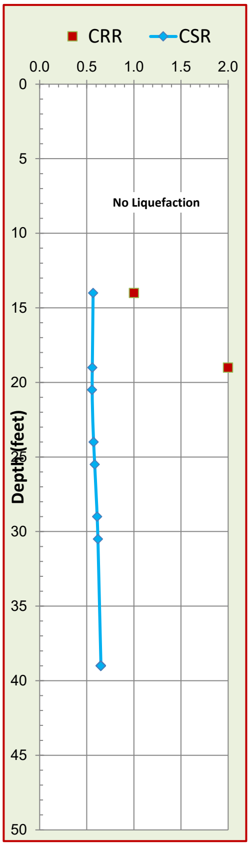
EXPECTED RANGE OF DISPLACEMENT

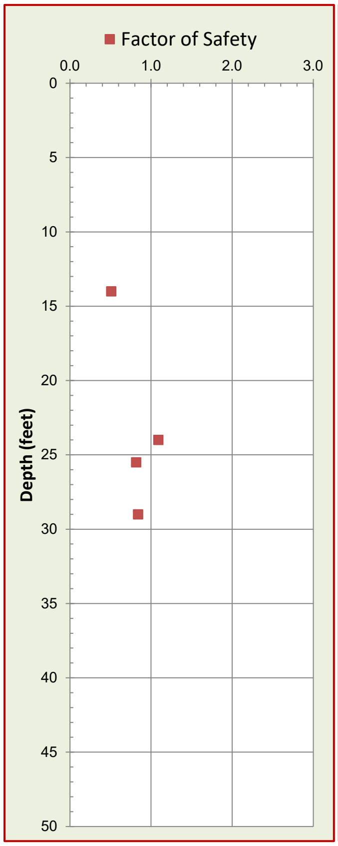
0.1 **to** **0.5** **feet**

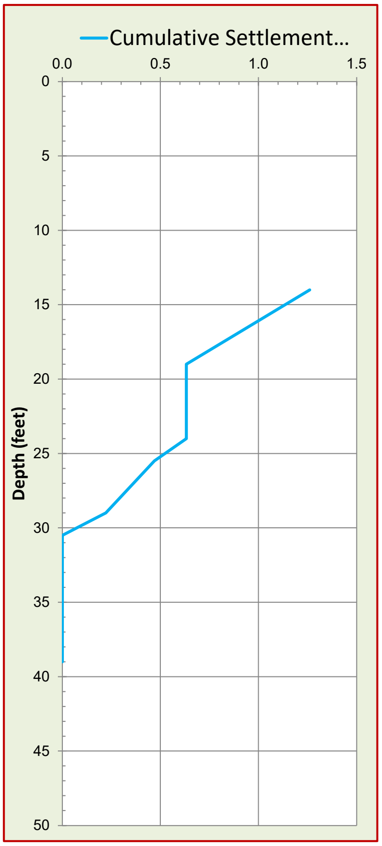
[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

Boring **EB-3**

SPT ANALYSIS RESULTS

PROJECT INFORMATION

Project Title	**31-57 South B Street DL**
Project No.	**480-6-1**
Project Manager	**MFR**

SEISMIC PARAMETERS

Controlling Fault	**San Andreas**	
Earthquake Magnitude (Mw)	**7.6**	
PGA (Amax)	**0.9**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**21**
Design Water Depth (feet)	**14**
Ave. Unit Weight Above GW (pcf)	**125**
Ave. Unit Weight Below GW (pcf)	**135**

MAGNITUDE SCALING FACTOR

0.974

CUMULATIVE SETTLEMENT FROM 50 **FEET**

0.95 **(Inches)**

LATERAL DISPLACEMENT

^{2}LDI 0.07 **L/H** 4.6

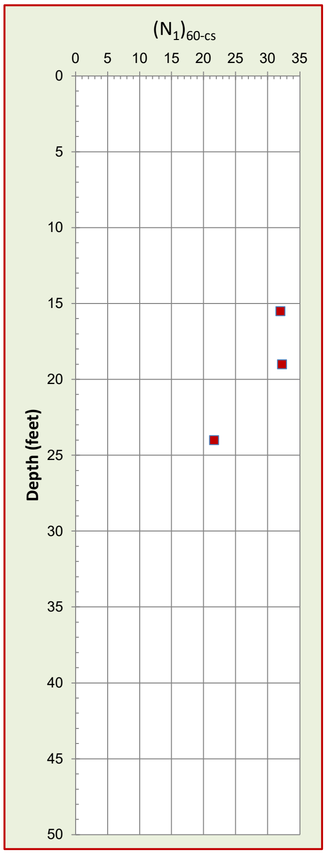
LDI^1$_{\text{Corrected for Distance}}$ 0.12 (4 < L/H < 40)

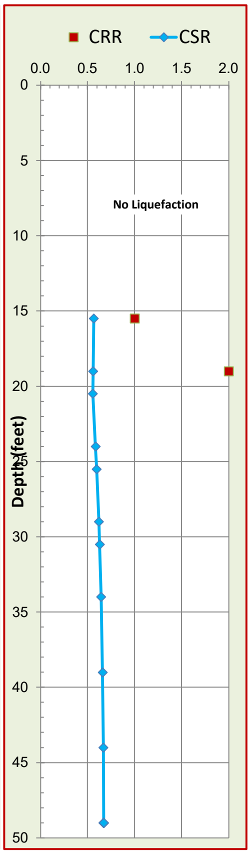
EXPECTED RANGE OF DISPLACEMENT

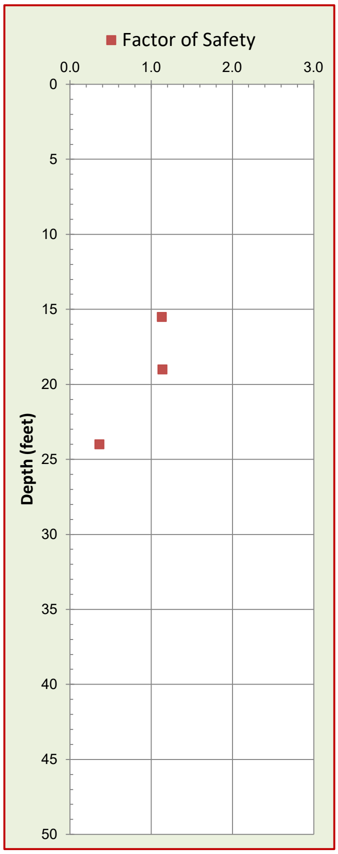
0.1 **to** **0.2** **feet**

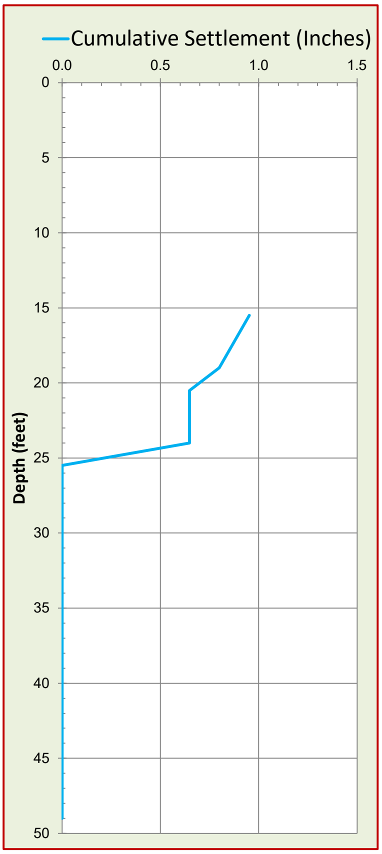
[1] Not Valid for L/H Values < 4 and > 40.

[2] LDI Values Only Summed to 2H Below Grade.









CORNERSTONE EARTH GROUP

Boring **EB-4**

PROJECT INFORMATION

Project Title **31-57 South B Street DL**

Project No. **480-6-1**

Project Manager **MFR**

SPT ANALYSIS RESULTS

MAGNITUDE SCALING FACTOR

0.899

SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.9**

PGA (Amax) **0.888** (g)

CUMULATIVE SETTLEMENT FROM 50 **FEET**

0.29 **(Inches)**

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **24**

Design Water Depth (feet) **14**

Ave. Unit Weight Above GW (pcf) **125**

Ave. Unit Weight Below GW (pcf) **135**

LATERAL DISPLACEMENT

[2]LDI 0.00 **L/H** 15.7

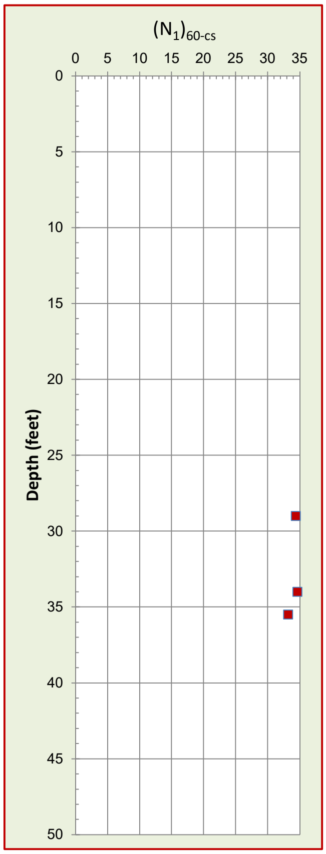
LDI[1] Corrected for Distance 0.00 (4 < L/H < 40)

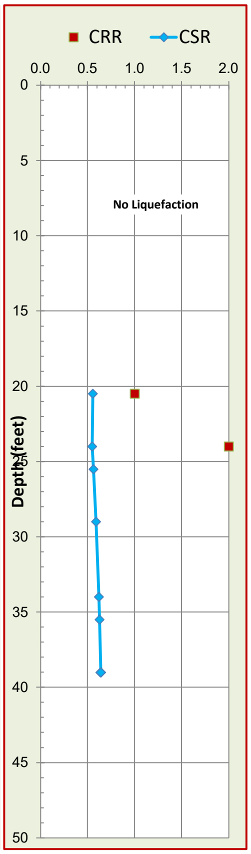
EXPECTED RANGE OF DISPLACEMENT

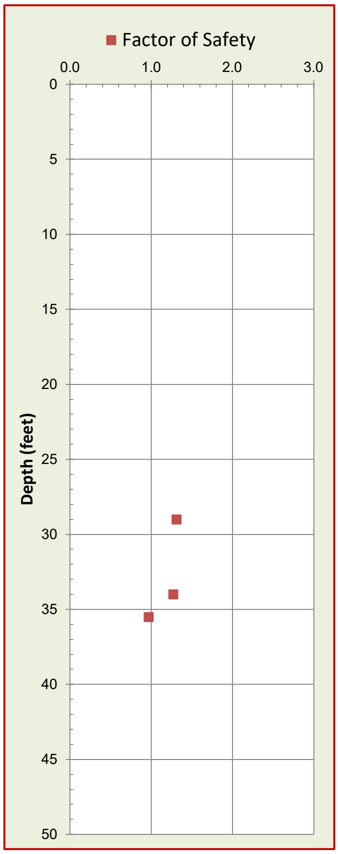
0.0 **to** **0.0** **feet**

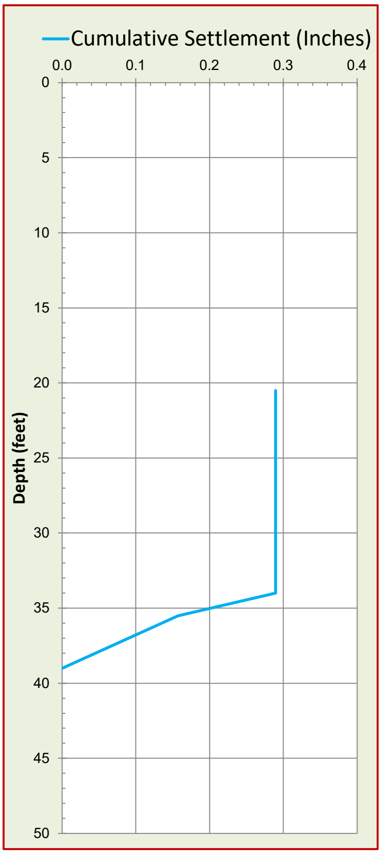
[1] Not Valid for L/H Values < 4 and > 40.

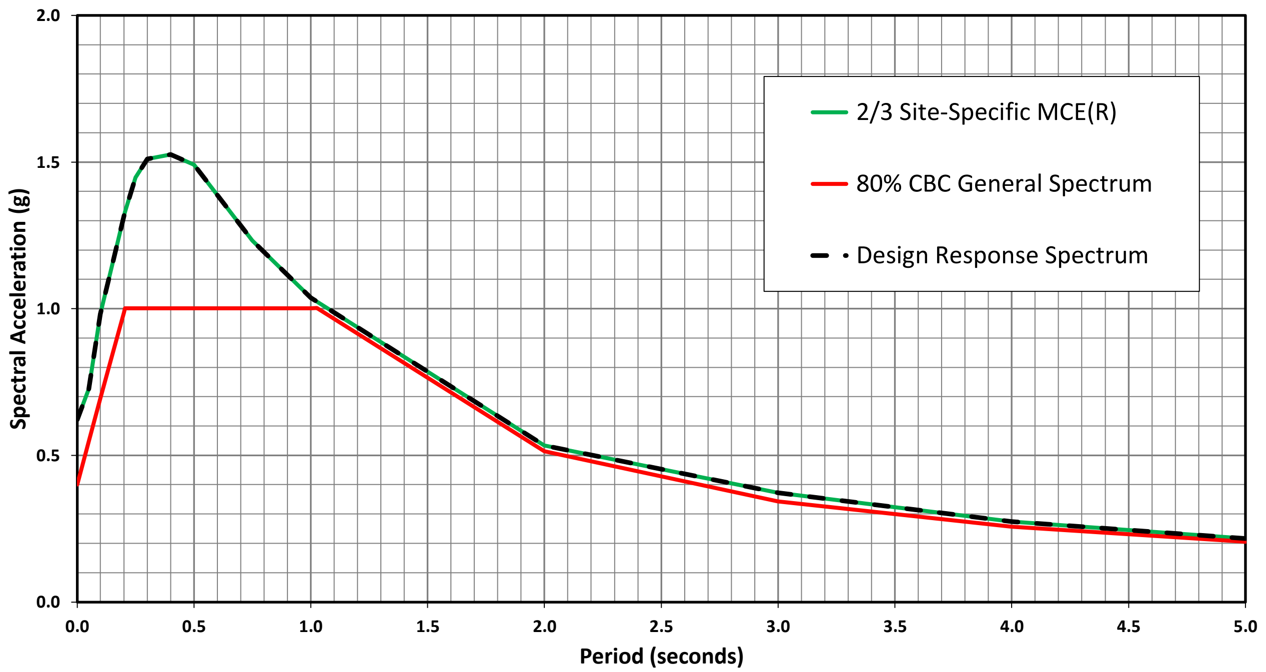
[2] LDI Values Only Summed to 2H Below Grade.











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	0.622
0.05	0.726
0.10	0.981
0.15	1.149
0.20	1.316
0.21	1.329
0.25	1.447
0.30	1.510
0.40	1.526
0.50	1.491
0.75	1.233
1.00	1.038
1.03	1.024
2.00	0.533
3.00	0.372
4.00	0.274
5.00	0.217

Site Design	Design Values
Site Class (Per Chapter 20 ASCE 7-16)	D
Shear Wave Velocity, V_{S30} (m/sec)	308
Site Latitude (degrees)	37.567425
Site Longitude (degrees)	-122.324072
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}	1.373
S_{D1}	1.115
S_{MS}	2.060
S_{M1}	1.673

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

31-57 South B St D-L GI
31-57 South B St
San Mateo, CA

FIGURE 5

PROJECT NO. 480-6-1

March 7, 2022 | BCG

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling. Four 8-inch-diameter exploratory borings were drilled on May 31, 2019 and February 16, 2022 to depths of 40 to 80 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 and bedrock, are included as part of this appendix.

Boring locations were approximated using existing site boundaries, a hand held GPS unit, 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. 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-2487-98)

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

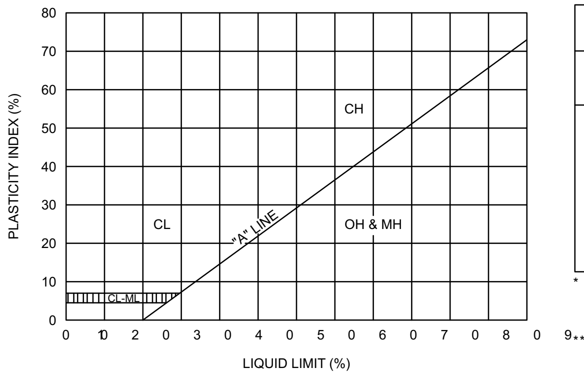
SAMPLER TYPES

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

ADDITIONAL TESTS

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

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 3

CORNERSTONE EARTH GROUP

PROJECT NAME 31-57 South B Street
PROJECT NUMBER 307-25-1
PROJECT LOCATION San Mateo, CA

DATE STARTED 5/31/19 **DATE COMPLETED** 5/31/19
GROUND ELEVATION +/- 29 ft. **BORING DEPTH** 80 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.567467° **LONGITUDE** -122.323920°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY DL
▽ **AT TIME OF DRILLING** 22 ft.
NOTES
▼ **AT END OF DRILLING** 30 ft.

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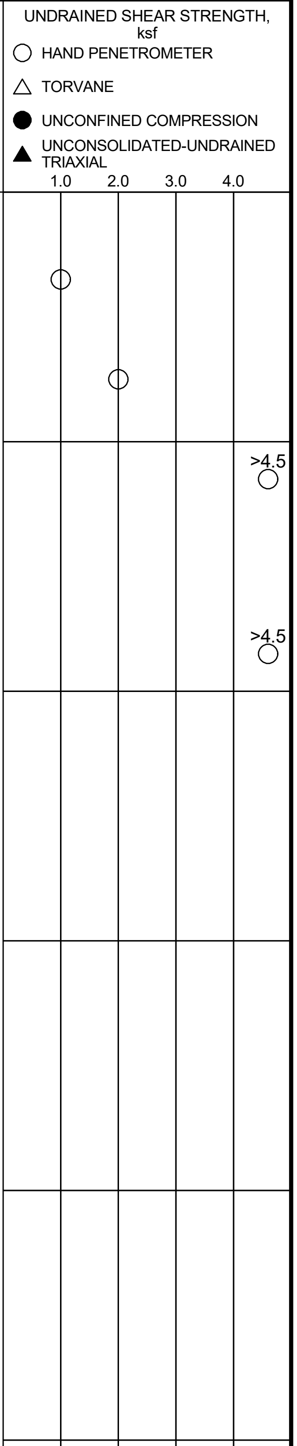
DESCRIPTION

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION
	0	8 inches Portland cement concrete over 2 inches aggregate base
		Clayey Sand (SC) [Fill] medium dense to loose, moist, brown, fine to coarse sand, some charred fragments
		Lean Clay with Sand (CL) stiff, moist, brown, low plasticity Liquid Limit = 24, Plastic Limit = 15
	5	**Sandy Lean Clay (CL)** hard, moist, brown with gray mottles, fine to medium sand, some gravel, low to moderate plasticity
	10	
		Clayey Sand (SC) medium dense, moist, reddish brown, fine sand
	15	
		Poorly Graded Sand with Silt (SP-SM) medium dense, moist, brown, fine to medium sand, some fine subangular to subrounded gravel
	20	**Clayey Sand with Gravel (SC)** dense to very dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel
▽		
	25	

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
15	MC-1B	105	18	9	
20	MC-2B	108	17		
79	MC-3A				
90	MC-4B	114	15		
50	MC				
54	MC-6B	125	11		11
50 2"	MC-7B	116	15		
62	SPT-8		17		



CORNERSTONE EARTH GROUP

BORING NUMBER EB-1

PAGE 2 OF 3

PROJECT NAME 31-57 South B Street

PROJECT NUMBER 307-25-1

PROJECT LOCATION San Mateo, CA

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ELEVATION (ft) | DEPTH (ft) | SYMBOL

DESCRIPTION

25

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

▼ 30

35

40

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

45

Clayey Sand (SC)
very dense, moist, brown, fine to coarse sand, some fine subangular to subrounded gravel

50

Continued Next Page

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

1.0 2.0 3.0 4.0

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
76	MC-9B	131	12			>4.5 ○
61	SPT-10		16			
50/6"	MC-11B	118	15			
60	SPT					
43	SPT-13		19			
50/6"	MC-14B	118	17		14	
70	SPT					
50/3"	MC-16B	104	23			○
50/4"	MC-17B	120	14			

PROJECT NAME 31-57 South B Street

PROJECT NUMBER 307-25-1

PROJECT LOCATION San Mateo, CA

ELEVATION (ft)	DEPTH (ft)	SYMBOL	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. **DESCRIPTION**	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
		▨	**Clayey Sand (SC)** very dense, moist, brown, fine to coarse sand, some fine subangular to subrounded gravel	32	○ NR					
	55	▨		96	☒ SPT-18		16			
		▨		58	☒ SPT-19		16			
	60	▨								
		▨	**Poorly Graded Sand with Clay (SP-SC)** very dense, wet, brown, fine sand, some gravel							
		▨		50/5"	☒ SPT-20		24			
	65	▨								
		▨	**Clayey Sand (SC)** very dense, moist, brown, fine to coarse sand, some fine subangular to subrounded gravel							
		▨		50/5"	☒ SPT-21		15		19	
	70	▨								
		▨		50/5"	☒ SPT-22		12			
	75	▨								
		▨	**Poorly Graded Sand with Clay (SP-SC)** very dense, wet, gray, fine to coarse sand, some fine subangular to subrounded gravel							
		▨		50	☒ SPT-23		15			
	80		Bottom of Boring at 80.0 feet.							

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 6/7/19 13:48 - P:\DRAFTING\GINT FILES\307-25-1 31-57 SOUTH B STREET.GPJ

BORING NUMBER EB-2

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME 31-57 South B Street
PROJECT NUMBER 307-25-1
PROJECT LOCATION San Mateo, CA

DATE STARTED 5/31/19 **DATE COMPLETED** 5/31/19
DRILLING CONTRACTOR Exploration Geoservices, Inc.
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
LOGGED BY DL
NOTES

GROUND ELEVATION +/- 30 ft. **BORING DEPTH** 40 ft.
LATITUDE 37.567309° **LONGITUDE** -122.324168°
GROUNDWATER LEVELS:
∇ **AT TIME OF DRILLING** 21 ft.
▼ **AT END OF DRILLING** 22 ft.

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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 ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
	0	6 inches Portland cement concrete over 5 inches aggregate base							
		Clayey Sand (SC) [Fill] medium dense, moist, brown, fine to coarse sand	25	MC-1B	108	17			○
		Lean Clay with Sand (CL) very stiff to hard, moist, brown, fine to medium sand, low plasticity	74	MC-2B	109	17			>4.5 ○
	5	**Clayey Sand with Gravel (SC)** dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel	60	MC-3B	113	14			
		Lean Clay with Sand (CL) hard, moist, brown with gray mottles, fine sand, moderate plasticity	50/5"	MC					>4.5 ○
	10								
		Clayey Sand with Gravel (SC) medium dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel	38	MC-5B	117	14			
	15								
		becomes dense	66	MC-6B	118	15			
	20								
	∇	Liquid Limit = 35, Plastic Limit = 17	38	SPT-7		18	18	20	
	▼								
			56	MC-8B	115	18			
	25								

Continued Next Page

BORING NUMBER EB-2

PROJECT NAME 31-57 South B Street

PROJECT NUMBER 307-25-1

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)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf 1.0 2.0 3.0 4.0
	25	▨	**Clayey Sand with Gravel (SC)** dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel	32	SPT-9		17			
				32	SPT					
	30			40	SPT-11		16			
		▨	**Lean Clay with Sand (CL)** very stiff, moist, brown, fine sand, low plasticity	40	SPT					○
	35	▨	**Clayey Sand (SC)** very dense, moist, brown, fine to coarse sand, some fine to coarse subangular to subrounded gravel	50 6"	MC-13B	123	15			
				64	SPT-14		14			
	40		Bottom of Boring at 40.0 feet.							
	45									
	50									

BORING NUMBER EB-3

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME 31-57 South B Street
PROJECT NUMBER 480-6-1
PROJECT LOCATION San Mateo, CA

DATE STARTED 2/16/22 **DATE COMPLETED** 2/16/22
GROUND ELEVATION **BORING DEPTH** 59.4 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.567316° **LONGITUDE** -122.324381°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY EA
▽ **AT TIME OF DRILLING** 21 ft.
NOTES
▼ **AT END OF DRILLING** 33 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	3 inches asphalt concrete over 6 inches Portland cement concrete and 6 inches aggregate base							
		Clayey Sand with Gravel (SC) [Fill] medium dense, moist, gray brown, fine to coarse sand, fine to coarse subangular gravel		GB-1		10			
				GB-2		20			
		Lean Clay with Sand (CL) stiff, moist, reddish brown, fine sand, low plasticity							
	5								
		Clayey Sand with Gravel (SC) medium dense, moist, brown to reddish brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	34	MC-3B	115	17			
			51	MC-4B	116	13			
	10								
		Sandy Lean Clay (CL) stiff, moist, reddish brown, fine to medium sand, low plasticity							
			34	MC-5B	111	19			○
	15	**Clayey Sand with Gravel (SC)** medium dense, moist, brown to reddish brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	24	SPT					
			39	MC					
	20								
	▽	becomes dense	39	SPT					
		Silty Sand (SM) medium dense, moist, brown to reddish brown, fine to medium sand	28	MC-9B	100	22			
	25	**Clayey Sand with Gravel (SC)** dense, moist, brown to reddish brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	40	SPT					

Continued Next Page

BORING NUMBER EB-3

PAGE 2 OF 2

PROJECT NAME 31-57 South B Street

PROJECT NUMBER 480-6-1

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	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
		▨	**Clayey Sand with Gravel (SC)** dense, moist, brown to reddish brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	59	MC-11B	118	17			
	30		becomes very dense	58	SPT					
	▼									
				60	MC-13B	116	16			
	35									
				44	MC-14B	107	21			>4.5 ○
	40	▨	**Lean Clay with Sand (CL)** hard, moist, reddish brown, fine to medium sand, moderate plasticity							
		▨	**Clayey Sand with Gravel (SC)** medium dense, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	32	MC-15B	115	18		29	
	45									
			becomes very dense	50 6"	MC					
	50									
			becomes medium dense	60	MC-17B	113	17			
	55									
			becomes very dense	50 5"	MC-18B	119	15			
			Bottom of Boring at 59.4 feet.							
	60									

BORING NUMBER EB-4

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME 31-57 South B Street
PROJECT NUMBER 480-6-1
PROJECT LOCATION San Mateo, CA
DATE STARTED 2/16/22 **DATE COMPLETED** 2/16/22
GROUND ELEVATION **BORING DEPTH** 40 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.567438° **LONGITUDE** -122.323816°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY EA
▽ **AT TIME OF DRILLING** 24 ft.
NOTES
▼ **AT END OF DRILLING** 33 ft.

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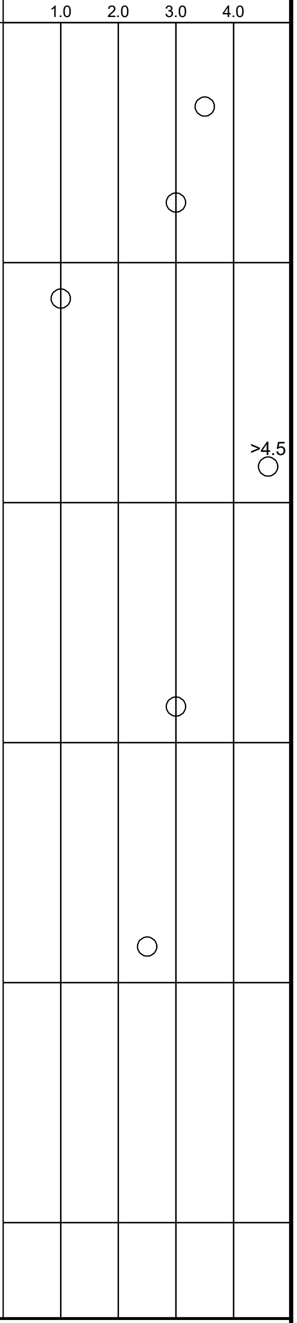
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
	0	6 inches Portland cement concrete and 6 inches aggregate base						
		Sandy Lean Clay (CL) [Fill] very stiff, moist, dark brown, fine to coarse sand, low plasticity Liquid Limit = 28, Plastic Limit = 19	24	MC-1B	105	19	9	
		Lean Clay with Sand (CL) [Fill] very stiff, moist, brown, fine sand, low plasticity	9	MC-2B	94	18		
	5	**Lean Clay with Sand (CL)** stiff, moist, reddish brown, fine sand, low plasticity	20	MC-3B	104	20		
	10	**Sandy Lean Clay (CL)** hard, moist, reddish brown, fine to medium sand, some fine subangular to subrounded gravel, low plasticity	52	MC-4B	111	17		
	15	**Lean Clay with Sand (CL)** very stiff, moist, reddish brown, fine to medium sand, moderate plasticity	34	MC				
			73	MC-6B	125	12		
	20	**Clayey Sand with Gravel (SC)** very dense, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	55	SPT				
▽		medium dense	38	MC				
	25		39	SPT-9		16		16

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



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PROJECT NAME 31-57 South B Street

PROJECT NUMBER 480-6-1

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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
			Clayey Sand with Gravel (SC) very dense, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel		☒					
				46	MC-10B	115	17			
	30		**Sandy Lean Clay (CL)** very stiff, moist, reddish brown, fine to medium sand, moderate plasticity	35	SPT					○
	▼		**Clayey Sand with Gravel (SC)** medium dense, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel							
				48	MC-12B	113	16			
	35		becomes dense	31	SPT					
				67	MC-14B	124	13			
	40		Bottom of Boring at 40.0 feet.							
	45									
	50									
	55									

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 51 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

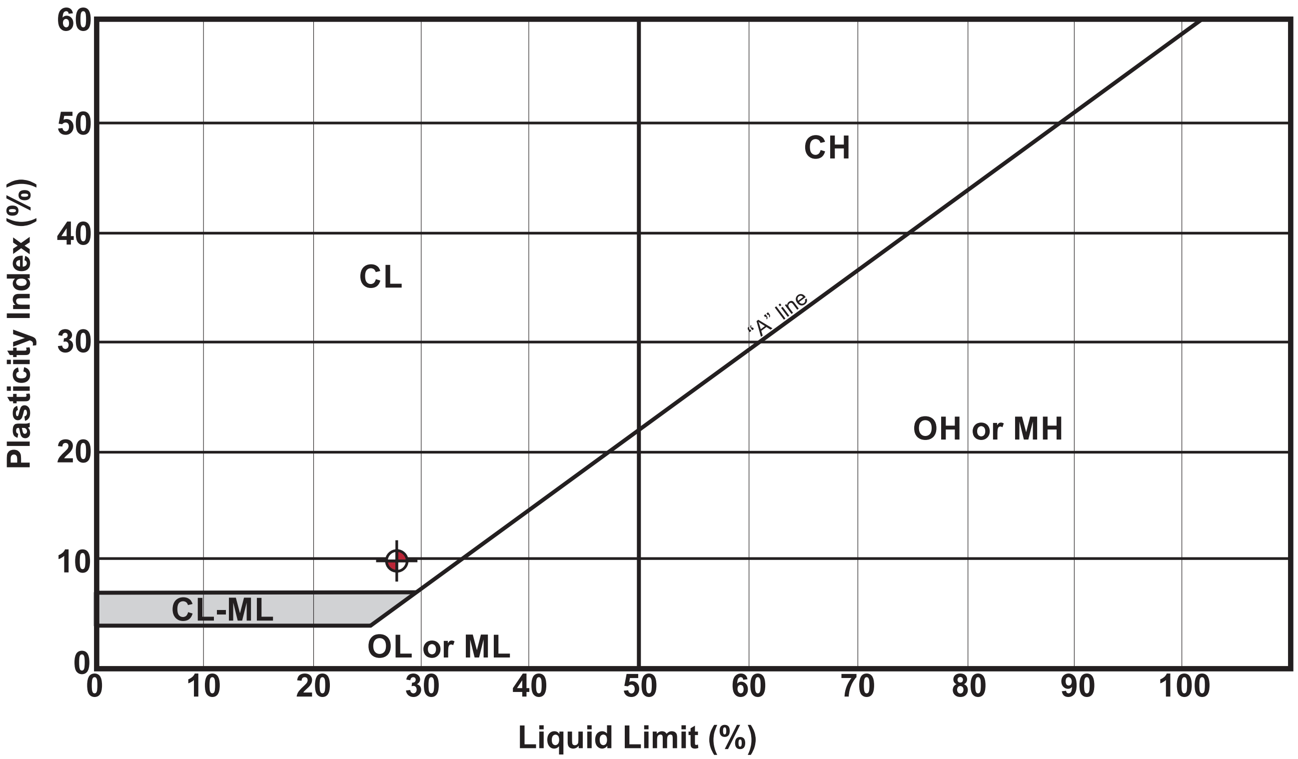
Dry Densities: In place dry density determinations (ASTM D2937) were performed on 35 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on six 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: Three 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.

Corrosion: – A suite of corrosion tests were performed on two samples of the subsurface soils including saturated resistivity, pH, and soluble sulfates and chlorides. Results of these tests are attached in this appendix.

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-4	2.0	19	28	19	9	—	Sandy Lean Clay (CL) [Fill]

CORNERSTONE
EARTH GROUP

Plasticity Index Testing Summary

31 to 57 South B Street
San Mateo, CA

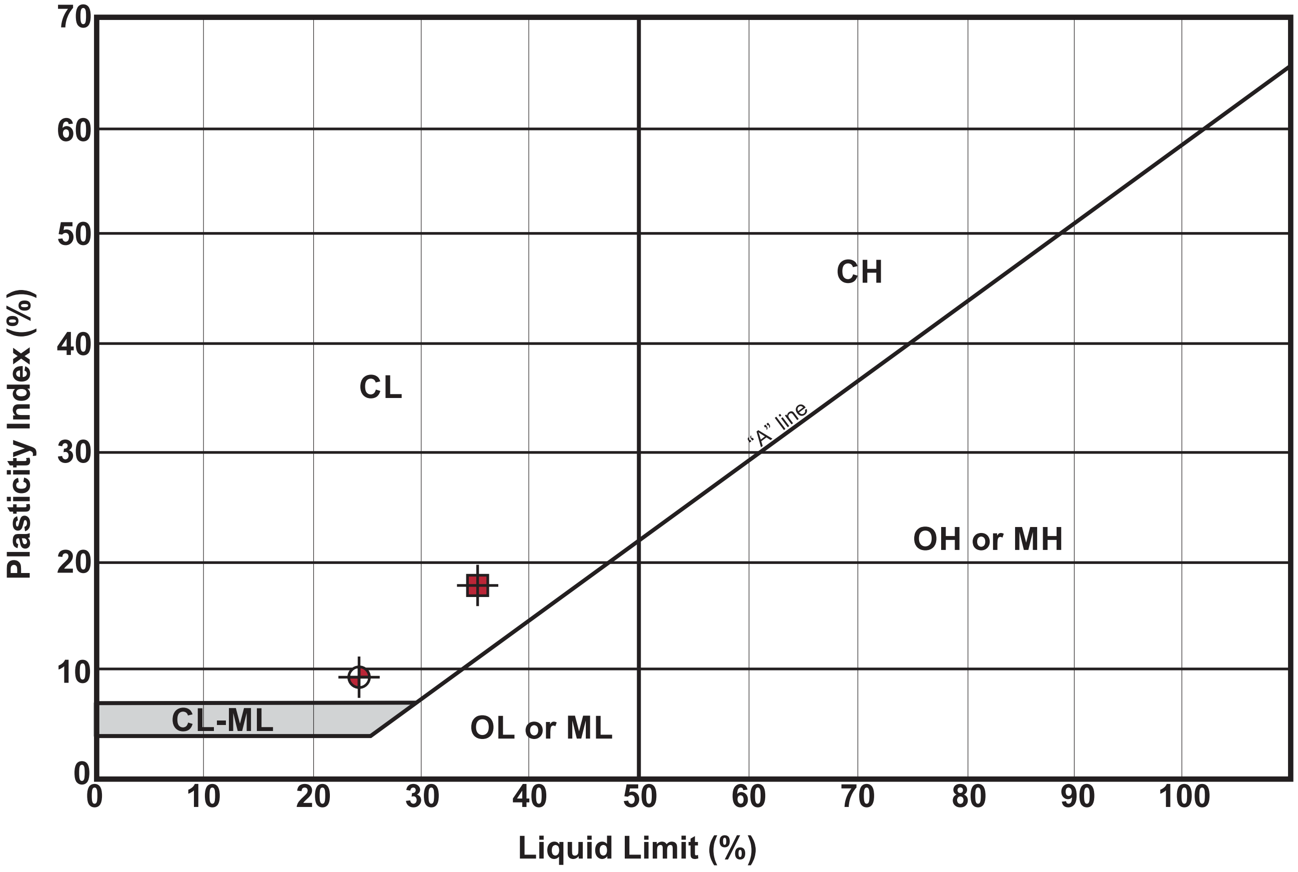
Project Number
480-6-1

Figure Number
Figure B1

Date
February 2022

Drawn By
FLL

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	2.0	18	24	16	9	—	Lean Clay with Sand (CL)
⊞	EB-2	20.5	18	35	17	18	20	Clayey Sand (SC) (CL fines)

CORNERSTONE
EARTH GROUP

Plasticity Index Testing Summary

31-57 South B Street
San Mateo, CA

Project Number
307-25-1

Figure Number
Figure B1

Date
June 2019

Drawn By
FLL

COOPER
TESTING LABORATORY

Corrosivity Tests Summary

CTL # 640-1320 **Date:** 6/17/2019 **Tested By:** PJ **Checked:** PJ
Client: Cornerstone Earth Group **Project:** South B Street **Proj. No:** 307-25-1
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
EB-1	3A	5.5	-	-	1,106	5	119	0.0119	6.8	-	-	-	18.9	Yellowish Brown Sandy CLAY
EB-2	6A	19.0	-	-	4,812	7	17	0.0017	7.5	-	-	-	15.4	Yellowish Brown Clayey SAND w/ Gravel