CORNERSTONE
EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**222 East 4th Avenue**
Location	**222 East 4th Avenue** **San Mateo, California**
Client	**Lane Partners**
Client Address	**644 Menlo Avenue, Suite 204** **Menlo Park, California**
Project Number	**129-6-1**
Date	**December 20, 2019**

DRAFT

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SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Lane Partners for the 222 East 4th Avenue project located in San Mateo, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following document:

- As-built plans titled, "Draeger's Market, San Mateo, CA", prepared by Field-Paoli Architecture & Planning, dated June 24, 1996.

It is noted that building code references and recommendations provided in this report are based on the 2016 California Building Code (CBC); therefore, this recommendations and code references in this report will need to be updated based on the 2019 CBC that takes effect on January 1, 2020.

1.1 PROJECT DESCRIPTION

The project will consist of a 4-story, mixed-use (office and retail) building with 2 levels of below-grade parking on an approximately 1.1-acre site. The above-grade portion of the structure is anticipated to be of wood- and/or steel-frame construction with the below-grade parking levels of concrete-frame construction.

Structural loads were not available at the time of this report and are anticipated to be typical of this type of structure. Grading is anticipated to include cuts of up to 25 feet for excavation of the below-grade parking levels.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated October 2, 2019 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, temporary shoring, 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 November 6 and 21, 2019 with truck-mounted, hollow-stem auger drilling equipment. The borings were drilled to a depth of 60 feet below the existing grades. The borings were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

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

1.4 LABORATORY TESTING PROGRAM

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

1.5 ENVIRONMENTAL SERVICES

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

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

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 is about 40 miles wide in the Bay area and extends from the San Gregorio Fault near the coastline to the Coast Ranges-Central Valley blind thrust at the western edge of the Great Central Valley as shown on the Regional Fault Map, Figure 3. The San Andreas Fault is the dominant structure in the system, nearly spanning the length of California, and capable of producing the highest magnitude earthquakes. Many other subparallel or branch faults within the San Andreas system are equally active and nearly as capable of generating large earthquakes. Right-lateral movement dominates on these faults but an increasingly large amount of thrust faulting resulting from compression across the system is now being identified also.

The project site is located on the flatlands surrounding San Francisco Bay west of the present tidal flats. The site is mapped as Holocene age coarse-grained alluvium (Qoa: Pampeyan, 1994) and alluvial fan and fluvial deposits (Qhaf: Brabb et al., 1998), underlain by Franciscan Complex sandstone (fs) with interbedded siltstone and shale.

2.2 REGIONAL SEISMICITY

The San Francisco Bay area region is one of the most seismically active areas in the Country. While seismologists cannot predict earthquake events, the U.S. Geological Survey's Working Group on California Earthquake Probabilities 2015 revises earlier estimates from their 2008 (2008, UCERF2) publication. Compared to the previous assessment issued in 2008, the estimated rate of earthquakes around magnitude 6.7 (the size of the destructive 1994 Northridge earthquake) has gone down by about 30 percent. The expected frequency of such events statewide has dropped from an average of one per 4.8 years to about one per 6.3 years. However, in the new study, the estimate for the likelihood that California will experience a magnitude 8 or larger earthquake in the next 30 years has increased from about 4.7 percent for UCERF2 to about 7.0 percent for UCERF3.

UCERF3 estimates that each region of California will experience a magnitude 6.7 or larger earthquake in the next 30 years. Additionally, there is a 63 percent chance of at least one magnitude 6.7 or greater earthquake occurring in the Bay Area region between 2007 and 2036.

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)	3.2	5.1
Monte Vista-Shannon	9.1	14.7
San Gregorio	10.2	16.5
Hayward (Total Length)	15.1	24.3

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

SECTION 3: SITE CONDITIONS

3.1 SITE BACKGROUND

Based on aerial images provided on HistoricAerials.com website (NETROnline, 2019) and the referenced plans, the site was occupied by a few residential and/or commercial developments and several large trees in an image dated 1946. The site was occupied by a large commercial building with at-grade parking areas and a residential development in the western and eastern portions of the site, respectively, in an image dated 1956. The residential development was not observed in the eastern portion of the site in an image dated 1968; however, additional at-grade parking was observed. Significant changes to the commercial development were not observed in images dated 1980 through 1993. The current commercial development at the site was observed in an image dated 2002, and based on the referenced plans, the current development was constructed in 1996. Significant changes to the current development were not observed in images dated after 2002.

3.2 SURFACE DESCRIPTION

The site is currently occupied by a commercial development consisting of a two-story commercial building with at-grade and one level of below-grade parking. The site is relatively level and at or near the elevation of the adjacent properties and roadways. Based on the referenced plans, the elevation of the site ranges from Elevation 25 feet in the southeastern portion of the site to 29.5 feet in the northwestern portion of the site, City of San Mateo Datum (CSMD).

Surface pavements of the adjacent roadways (Ellsworth Avenue and B Street) generally consisted of 6 to 8 inches of asphalt concrete over 3 to 8 inches of aggregate base. Based on our observations, the existing pavements are in good/poor condition with significant alligator cracking.

3.3 SUBSURFACE CONDITIONS

Below the surface pavements, our explorations generally encountered undocumented fill over alluvial soil. The undocumented fill was encountered to a depth of 5 feet (corresponding to Elevations 20.5 and 24 feet CSMD for Exploratory Borings EB-1 and EB-2, respectively), and consisted of sandy lean clay and clayey sand. The underlying alluvial soil was encountered to a depth of 60 feet below the existing grades (corresponding to Elevations -34.5 and -31 feet CSMD), the maximum depth explored and consisted of very stiff to hard, sandy lean clay with gravel, sandy lean clay, and lean clay with sand. Several prominent layers of medium dense to dense, clayey sand with gravel and dense to very dense, poorly graded sand with gravel were encountered at depths of 5 to 22 feet (corresponding to Elevations 20.5 and 3.5 feet CSMD, respectively) within EB-1, and 1 to 17½ feet (corresponding to Elevations 28 and 11.5 feet CSMD, respectively) and 22½ to 26 feet (corresponding to Elevations 6.5 and 3 feet CSMD, respectively) within EB-2.

3.3.1 Plasticity/Expansion Potential

We performed one Plasticity Index (PI) test on a representative sample of the foundation bearing soil. Test results were used to evaluate expansion potential of surficial soil. The test resulted in a PI of 17, indicating low expansion potential to wetting and drying cycles.

3.3.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet are at or near the estimated laboratory optimum moisture. Moisture contents from 10 to 25 feet range from 0 to 5 percent over the estimated laboratory optimum moisture.

3.4 GROUNDWATER

Groundwater was encountered in our Exploratory Borings (EB-1 and EB-2) at depths of 31 to 32 feet below existing grades (corresponding to Elevations -2 and -6.5 feet CSMD, respectively). 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. Historic high groundwater at the site is mapped at a depth of 18 feet (CGS 2018). Additionally, groundwater monitoring wells in the vicinity of the site (e.g. within approximately 600 feet), indicate depths to groundwater to be about 16¾ to 21½ feet below the existing grades. therefore, a design groundwater depth of 18 feet is recommended.

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

3.5 CORROSION SCREENING

We tested one sample collected at a depth of 25 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	Soil Type	Depth (feet)	Soil pH[1]	Resistivity[2] (ohm-cm)	Chloride[3] (mg/kg)	Sulfate[4,5] (mg/kg)
EB-1	Lean Clay	25	6.7	3,304	5	10

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 soil may be considered moderately corrosive to buried metallic improvements (Chaker and Palmer, 1989).

In accordance with the 2016 CBC Section 1904A.1, alternative cementitious materials for different exposure categories and classes shall be determined in accordance with ACI 318-14 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 318-14, Table 19.3.1.1 below in Table 2B.

Table 2B: ACI 318-14 Table 19.3.1.1 Exposure Categories and Classes

Boring No. / Soil Type	Freezing and Thawing (F)	Sulfate (S, soil)	In Contact with Water (W)	Corrosion Protection of Reinforcement (C)
EB-1 / Lean Clay	F0[1]	S0[2]	W0[3]	C1[4]

1 (F0) "Concrete not exposed to freezing-and-thawing cycles" (ACI 318-14)
2 (S0) "Water soluble sulfate in soil, percent by mass" is less than 0.10 (ACI 318-14)
3 (W0) "Concrete in contact with water and low permeability is not required" (ACI 318-14)
4 (C1) "Concrete exposed to moisture but not to an external source of chlorides" (ACI 318-14)

In addition, ACI 318-14, Table 19.3.2.1 provides requirements for concrete by exposure class. Table 2C below indicates different requirements that we recommend be followed for the concrete design.

Table 2C: ACI 318-14 Table 19.3.2.1 Requirements for Concrete by Exposure Class

Exposure Class	Maximum water:cement ratio	Minimum Compressive Strength (psi)	Cementitious materials – Types (ASTM C150)	Maximum Water-Soluble Chloride Ion Content (% wt)
F0	N/A	2,500	N/A	N/A
S0	N/A	2,500	No type restriction	N/A
W0	N/A	2,500	N/A	N/A
C1	N/A	2,500	N/A	0.30 (0.06)[1]

1 Maximum water-soluble chloride ion content for non-pre-stressed concrete, (value for pre-stressed concrete).

We recommend the structural engineer and a corrosion engineer be retained to confirm the information provided and for additional recommendations, as required.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. The site is not located within a State-designated Alquist Priolo Earthquake Fault Zone. As shown in

Figure 3, no known surface expression of fault traces is thought to cross the site; therefore, fault rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. Peak ground accelerations (PGA) of 0.764g was estimated for analysis using a value equal to $PGA_M = F_{PGA} \times PGA_G$ (Equation 11.8-1) as allowed in the 2016 California Building Code (CBC).

4.3 LIQUEFACTION POTENTIAL

The site is not currently mapped within a liquefaction hazard zone (CGS, 2018) and is within a zone mapped as having a very low to low susceptibility to liquefaction by the Association of Bay Area Governments (ABAG, 2007). We screened the site for liquefaction during our site exploration by retrieving samples from the site, performing visual classification on sampled materials, and performing various tests to further classify the soil properties.

4.3.1 Background

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

4.3.2 Analysis

As discussed in the “Subsurface” section above, our borings predominately encountered dense sand below the design groundwater depth of 18 feet; however, we performed a liquefaction analysis to evaluate the potential for liquefaction induced settlement. Following the procedures in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008) and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008) for quantitative analysis, this layer was 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.

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 density and strength obtained from field SPT blow counts (“N” value). The “N” values are corrected for effective overburden stresses, taking into consideration both the ground water level at the time of exploration and the design ground water level, and stress reduction versus depth factors. The “N” values are also corrected for fines content, hammer efficiency, boring diameter, rod length, and sampler type (with or without liners).

4.3.3 Summary of Liquefaction Potential

Our analyses indicate the dense to very dense sandy layers encountered in our borings below the design groundwater depth of 18 feet are not susceptible to liquefaction based on the Yoshimine (2006) method.

4.3.4 Ground Rupture Potential

The methods used to estimate liquefaction settlement assume there is a sufficient cap of non-liquefiable material to prevent ground rupture or sand boils. Based on the results of our liquefaction analysis discussed above, the potential for ground rupture to occur at the site is negligible.

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.

San Mateo Creek is located approximately 980 to 1,450 feet northwest and northeast of the site, respectively. Additionally, the potential for liquefaction at the site is negligible. Therefore, the potential for lateral spreading to occur and/or impact the proposed improvements at the site is also considered to be negligible.

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 medium dense sandy soil encountered in our borings above a design groundwater depth of 18 based on the work by Pradell (1998). Our analyses indicate that seismic compaction of the sandy soil could result in up to ¾ inch of settlement at the ground surface after strong seismic shaking; however, based on the upper 25 feet of soil being removed for excavation of the basement levels, the remaining seismic sand settlement that could occur would be negligible.

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 maps of tsunami inundation zones (CGS, 2009) and the study of tsunami inundation potential for the San Francisco Bay Area (Ritter and Dupre, 1972), 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 1.2 miles inland from the San Francisco Bay shoreline and is approximately 25.5 to 29 feet above mean sea level. 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 determined to be outside the 0.2% annual chance floodplain. We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

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.

- Proximity of basement excavation to existing improvements
- Presence of cohesionless soil at basement level
- Differential movement at on-grade to on-structure transitions

- Potential for seismic and static settlement
- Soil corrosion potential

5.1.1 Proximity of Basement Excavation to Existing Improvements

We understand the basement will likely extend to the existing property lines. Temporary shoring to support the approximately 25-foot deep excavation adjacent to East 4th and 5th Avenues, Ellsworth Avenue, and B Street will likely be necessary. Recommendations for temporary shoring are provided in this report.

5.1.2 Presence of Cohesionless Soil at Basement Level

As discussed, cohesionless (sandy) soils with variable amounts of fines were encountered within portions of the upper 25 feet of the soil profile that may be susceptible to localized sloughing or caving. Contractors should plan on forming footings where sand with low fines contents are encountered, as well as preparation of slab-on-grade subgrade just prior to concrete placement. Other similar construction issues as relates to temporary shoring, utility excavations, and granular material at the base of the basement excavation. These considerations are discussed further within the “Earthwork” and “Foundations” sections of this report.

5.1.3 Differential Movement At On-grade to On-Structure Transitions

Some of the at-grade sidewalks and other improvements will transition from on-grade support to overlying the basement. Where the depth of soil cover overlying the basement roof and at-grade improvements is thin or where basement walls extend to within inches of finished grade, these transition areas typically experience increased differential movement due to a variety of causes, including difficulty in achieving compaction of retaining wall backfill closest to the wall. We recommend consideration be given to where engineered fill is placed behind retaining walls extending to near finished grade, and that subslabs be included beneath flatwork or pavers that can span at least 3 feet beyond the wall. If surface improvements are included that are highly sensitive to differential movement, additional measures may be necessary. We also recommend that retaining wall backfill be compacted to 95 percent where surface improvements are planned (see “Retaining Wall” section).

5.1.4 Potential For Seismic and Static Settlement

5.1.4.1 Seismic Sand Settlement

Additionally, our seismic sand settlement analysis indicates that there is a potential for seismic compaction of localized sand layers located above the design groundwater depth of 18 feet during a significant seismic event. Our analysis indicates that seismic sand settlement of up to ¾ inch could occur at the ground surface; however, based on the upper 25 feet of soil being removed for excavation of the basement levels, our analysis indicates the remaining total seismic sand settlement will be negligible.

5.1.4.2 Static Settlement

We performed static settlement analyses, based on assumed foundation loads, to estimate the settlement that will occur due to static loading conditions for the proposed building. Our analysis indicates that approximately 1¼ inches of total static settlement will occur for the proposed four-story office/retail building under the anticipated static structural loads, with about ¾ inch of differential static settlement in 30 feet across a mat foundation.

Foundations should be designed to tolerate the anticipated total and differential settlement. Based on our analysis with assumed foundation loads, it should be feasible to support the proposed building 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.

5.1.5 Soil Corrosion Potential

Our testing indicates sulfate exposure at the site is low and therefore no cement-type restrictions to buried concrete. The corrosion potential for buried metallic structures, such as metal pipes, is considered moderately corrosive. Based on the results of the preliminary soil corrosion screening, special requirements for corrosion control will likely be required to protect metal pipes and fittings.

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.

We anticipate the majority of the existing improvements at the site will be removed during excavation for the basement; however, as an owner value-engineered option, existing slabs and foundations that extend into areas of planned at-grade improvements such as flatwork 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 drilled piers are encountered within areas of planned at-grade improvements, they should be cut off at an elevation at least 60 inches below proposed improvements or the final subgrade elevation, whichever is deeper. The remainder of the drilled pier could remain in place. As discussed, improvements encountered within the planned building footprint are anticipated to be removed during excavation for the below-grade parking levels. 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 existing utilities within the building footprint are anticipated to be removed during excavations for the basement levels. 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 and subsurface improvements to be removed within the proposed development area. Demolition of existing improvements is discussed in the prior paragraphs.

6.3 REMOVAL OF EXISTING FILLS

Although up to 5 feet of undocumented fill was encountered within our borings, fills extending into planned at-grade flatwork areas will likely be removed during excavation for the basement levels.

If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should be screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below.

6.4 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. On a preliminary basis, the upper 15 feet at the site may be classified as OSHA Soil Type C materials. A Cornerstone representative should be retained to confirm the preliminary site classification. Recommended soil parameters for temporary shoring are provided in the “Temporary Shoring” section of this report.

Excavations performed during site demolition and fill removal should be sloped at 3:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Excavations extending more than 5 feet below building subgrade and excavations in pavement and flatwork areas should be sloped in accordance with the OSHA soil classification.

6.5 BELOW-GRADE EXCAVATIONS

Below-grade excavations may be constructed with temporary slopes in accordance with the “Temporary Cut and Fill Slopes” section above if space allows. Alternatively, temporary shoring may support the planned cuts up to 25 feet. We have provided geotechnical parameters for shoring design in the section below. The choice of shoring method should be left to the contractor’s judgment based on experience, economic considerations and adjacent improvements such as utilities, pavements, and foundation loads. Temporary shoring should support adjacent improvements without distress and should be the contractor’s responsibility. A pre-condition survey including photographs and installation of monitoring points for existing site improvements should be included in the contractor’s scope. We should be provided the opportunity to review the geotechnical parameters of the shoring design prior to implementation; the project structural engineer should be consulted regarding support of adjacent structures.

6.5.1 Temporary Shoring

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

Table 3: Suggested Temporary Shoring Design Parameters

Design Parameter	Design Value
Minimum Lateral Wall Surcharge (upper 5 feet)	120 psf
Cantilever Wall – Triangular Earth Pressure	40 pcf
Restrained Wall – Uniform Earth Pressure	25H*
Passive Pressure – Starting at 2 feet below the bottom of the excavation or bottom of foundation excavations, whichever is deeper	400 pcf up to 2,000 psf maximum uniform pressure

* H equals the height of the excavation; passive pressures are assumed to act over twice the soldier pile diameter

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

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

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

The above recommendations are for the use of the design team; the contractor in conjunction with input from the shoring designer should perform additional subsurface exploration they deem necessary to design the chosen shoring system. A California-licensed civil or structural engineer must design and be in responsible charge of the temporary shoring design. The contractor is responsible for means and methods of construction, as well as site safety.

6.6 AT-GRADE SUBGRADE PREPARATION

After site clearing and demolition is complete, and prior to backfilling any excavations resulting from fill removal or demolition, at-grade excavation subgrade and subgrade within areas to receive additional site fills and/or slabs-on-grade should be scarified to a depth of at least 6 inches, moisture conditioned, and compacted in accordance with the “Compaction” section below.

6.7 SUBGRADE PREPARATION

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.

6.8 SUBGRADE STABILIZATION MEASURES

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

As discussed in the "Subsurface" section in this report, the in-situ moisture contents are about 0 to 5 percent over the estimated laboratory optimum in the upper 25 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 soil.

6.8.1 Below-Grade Excavation Stabilization

As the planned basement excavation will extend near the current ground water level, we recommend that the contractor plan to excavate an additional 12 to 18 inches below subgrade, place a layer of stabilization fabric (Mirafi RS380i, or equivalent) at the bottom, and backfill with clean, crushed rock. The crushed rock should be consolidated in place with light vibratory equipment. Rubber-tire equipment should not be allowed to operate on the exposed subgrade; the crushed rock should be stockpiled and pushed out over the stabilization fabric.

6.9 MATERIAL FOR FILL

6.9.1 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.

6.9.2 Potential Import Sources

Imported and non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less, and not contain recycled asphalt concrete where it will be used within the building area. 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.

6.10 COMPACTION REQUIREMENTS

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

Table 4: 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
General Fill (below a depth of 5 feet)	On-Site Soils	95	>1
Basement Wall Backfill	Without Surface Improvements	90	>1
Basement Wall Backfill	With Surface Improvements	95[4]	>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	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)
4 – Using light-weight compaction or walls should be braced

6.10.1 Construction Moisture Conditioning

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

6.11 TRENCH BACKFILL

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

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

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

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

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

6.12 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.

6.13 LANDSCAPE CONSIDERATIONS

Since the near-surface soils are moderately to highly expansive, we recommend greatly reducing the amount of surface water infiltrating these soils near foundations and exterior slabs-on-grade. This can typically be achieved by:

- Using drip irrigation
- Avoiding open planting within 3 feet of the building perimeter or near the top of existing slopes
- Regulating the amount of water distributed to lawns or planter areas by using irrigation timers
- Selecting landscaping that requires little or no watering, especially near foundations.

We recommend that the landscape architect consider these items when developing landscaping plans.

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed structure may be supported on shallow foundations consisting of a mat foundation provided the recommendations in the "Earthwork" section and the sections below are followed.

7.2 SEISMIC DESIGN CRITERIA

As discussed, the 2019 CBC will take effect starting January 1, 2020; therefore, the project structural design will be based on the 2019 CBC, which provides criteria for the seismic design of buildings in Chapter 16. The "Seismic Coefficients" used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Based on our borings and review of local geology, the site is underlain by deep alluvial soils with typical SPT "N" values between 15 and 50 blows per foot. Therefore, we have classified the site as Soil Classification D. The mapped spectral acceleration parameters S_S and S_1 were calculated using the online web-based program *ATC Hazards by Location* (https://hazards.atcouncil.org), based on the site coordinates presented below and the site classification. **The values in Table 4 should not be used for design unless in the judgement of the project structural engineer Exception 2 under Section 11.4.8, of ASCE 7-16, can be used for project design.**

Table 4: 2019 CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.564246°
Site Longitude	-122.321656°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.88g
1-second Period Mapped Spectral Acceleration[1], S_1	0.771g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv	*null
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.88g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	*null
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.254g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	*null

*null – Site-specific ground motion hazard analysis required.

In our opinion, the proposed structure may be supported on shallow foundations consisting of a mat foundation provided the recommendations in the “Earthwork” section and the sections below are followed.

7.3 SHALLOW FOUNDATIONS

As discussed, groundwater was encountered within our borings at depths of 31 to 32 feet below the existing grade. Additionally, a design groundwater depth of 18 feet is recommended. The bottom of the basement foundations is assumed to be at a depth of 25 feet; therefore, the design groundwater is about 7 feet above the assumed bottom of the basement foundation. We recommend that waterproofing and hydrostatic pressure due perched water (above the basement slab and behind the walls) be considered in the foundation design. Detailed recommendations are provided below.

7.3.1 Reinforced Concrete Mat Foundation

As discussed, the four-story office/retail building will include a two-level basement anticipated to be up to 25 feet below the existing grades. Based on the assumed structural loads and the depth to historic high groundwater, the structure may be supported on a mat foundation.

7.3.2 Allowable Mat Bearing Pressure

We have estimated areal loading for our analysis based on a building load of 125 psf per floor for the above-grade steel office/retail structure and 150 psf per floor for the basement. Based on the assumed structural loads and provided total footprint, we have estimated an average areal pressure of about 800 pounds per square foot (psf) for the structure. We recommend the allowable bearing pressure at heavier loaded portions of the mat slab be limited to an allowable bearing pressure of 2,500 psf for dead plus live loads. The maximum bearing pressure may be increased by one-third for all loads, including wind or seismic. This pressure is a net value; the mat weight may be neglected for the portion of the mat extending below grade. Top and bottom reinforcing steel should be included as required to help span irregularities and differential settlement. It is essential that we observe the mat foundation pad prior to placement of reinforcing steel.

7.3.3 Mat Foundation Settlement

We estimate the total settlement due to static loading would be about 1¼ inches with differential movement of about ¾ inch generally from the center of the mat to the mat edges. If foundations designed in accordance with the above recommendations are not capable of resisting such differential movement, additional reinforcement or increased mat thickness may be required.

7.3.4 Mat Foundation 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 0.4 (0.27 allowable) applied to the mat dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 500 pcf (330 pcf allowable) may be used in design.

7.3.5 Mat Modulus of Soil Subgrade Reaction

We recommend using a variable modulus of subgrade reaction to provide a more accurate soil response and prediction of shears and moments in the mat. This will require at least one iteration between our soil model and the structural SAFE (or equivalent) analysis for the mat. A preliminary modulus of subgrade reaction for the initial analysis is provided below.

As discussed above, we estimated an average areal pressure of 800 psf within the structure. Based on this pressure, we calculated a preliminary modulus of soil subgrade reaction for the mat foundation. Based on the anticipated loads for the mat slabs and soil conditions, we recommend an initial modulus of soil subgrade reaction of 10 pci be used for preliminary SAFE runs. As discussed above, this modulus of soil subgrade reaction is intended for use in the first iteration of the structural SAFE analysis for the mat design. We will provide a revised plan with contours of equal modulus of subgrade reaction values following our receipt of output from initial SAFE runs indicating bearing pressures due to dead plus live loading across the mat.

7.4 HYDROSTATIC UPLIFT AND WATERPROOFING

As discussed, perched groundwater conditions could potentially develop in the sand layers during the life of the structure. To mitigate potential impacts to the structure due to perched groundwater buildup, we recommend that basement walls be designed with full drainage behind the walls or be designed for hydrostatic pressure (an additional 40 pcf of fluid pressure) and waterproofed. However, waterproofing should be considered for drained wall also.

If a perimeter foundation drain system to on-site detention is not planned, we recommend designing the slab for some amount of hydrostatic uplift pressure. As the issue is the potential for short duration perched water events and not long-term water table rises, the choice of the amount of uplift pressure is difficult to predict, if it will even occur over the life of the structure. Therefore, the design could proceed based on a risk versus cost basis. For a lower risk, we judge 7 feet of hydrostatic pressure to be reasonable, as measured from the basement finished floor. To lower slab cost, at somewhat higher risk, 5 feet can be used.

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 BELOW-GRADE INTERIOR SLAB-ON-GRADE

As the Plasticity Index (PI) of the surficial soils ranges up to 17, the proposed slabs-on-grade may be directly on subgrade prepared in accordance with the recommendations in the "Earthwork" section of this report. However, if unstable subgrade conditions are encountered, subgrade stability recommendations provided in Section 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 NEF 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 at least 3 percent over the 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.

8.2 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 10-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

- 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.

- 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.

8.3 EXTERIOR FLATWORK (AT-GRADE)

Exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading may be supported directly on subgrade prepared in accordance with the "Earthwork" recommendations of this report. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

SECTION 9: RETAINING WALLS

9.1 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 5: Recommended Lateral Earth Pressures

Wall Condition	Lateral Earth Pressure*	Additional Surcharge Loads
Unrestrained – Cantilever Wall	45 pcf	⅓ of vertical loads at top of wall
Restrained – Braced Wall	45 pcf + 8H** psf	½ of vertical loads at top of wall

* Lateral earth pressures are based on an equivalent fluid pressure for level backfill conditions
** H is the distance in feet between the bottom of footing and top of retained soil

Basement walls should be designed as restrained walls. 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.

9.2 SEISMIC LATERAL EARTH PRESSURES

The 2016 California Building Code (CBC) states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. We developed seismic earth pressures for the proposed basement using interim recommendations generally based on refinement of the Mononobe-Okabe method (Lew et al., SEAOC 2010). Because the walls are greater than 12 feet in height, and peak ground accelerations are greater than 0.40g, we checked the result of the seismic increment when added to the recommended active earth pressure against the recommended fixed wall earth pressures. Basement walls are not free to deflect and should therefore be designed for static conditions as a restrained wall, which is also a CBC requirement. Based on current recommendations for seismic earth pressures, it appears that active earth pressures plus a seismic increment exceed the restrained (i.e. at-rest), static wall earth pressures. Therefore, we recommend checking the walls for the seismic condition in accordance with the interim recommendations of the above referenced paper and the 2013 CBC.

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

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

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

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

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

E - should represent the seismic increment component in Eq. 16-7, a triangular load with a resultant force of $18.5H^2$, which should be applied one third of the height up from the base of the wall (and which can also be expressed as an equivalent fluid pressure equal to 37 pcf).

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

9.3 WALL DRAINAGE

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

Sections of horizontal drainage strips should be connected with either the manufacturer's connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path. In addition, where drainage panels will connect from a horizontal application for plaza areas to vertical basement wall drainage panels, the drainage path must be maintained. We are not aware of manufactured corner protection suitable for this situation; therefore, we recommend that a section of crushed rock be placed at the transitions. The crushed rock should be at least 3 inches thick, extend at least 12 inches horizontally over the top of the basement roof and 12 inches down from the top of the basement wall, and have a layer of filter fabric covering the crushed rock.

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

9.4 VEHICLE SURCHARGE LOADS

We understand the basement walls along East 4th and 5th Avenues, B Street, and South Ellsworth Avenue will possibly be extended under the existing sidewalk. Therefore, the basement walls will likely be subject to traffic loads. We recommend the basement walls along these streets be designed to resist a traffic surcharge load of 250 psf located at the top of the wall.

9.5 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 surface improvements are not planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

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

9.6 FOUNDATIONS (AT-GRADE)

Site retaining walls may be supported on a continuous spread footing on natural, undisturbed soil or engineered fill, be at least 15 inches wide, and extend at least 16 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. The deeper footing embedment is due to the presence of highly expansive soils and is intended to embed the footing below the zone of significant seasonal moisture fluctuation, reducing the potential for differential movement.

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.

SECTION 10: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Lane Partners specifically to support the design of the commercial development located at 222 East 4th Avenue 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.

Lane Partners, LLC may have provided Cornerstone with plans, reports and other documents prepared by others. Lane Partners, LLC 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 11: REFERENCES

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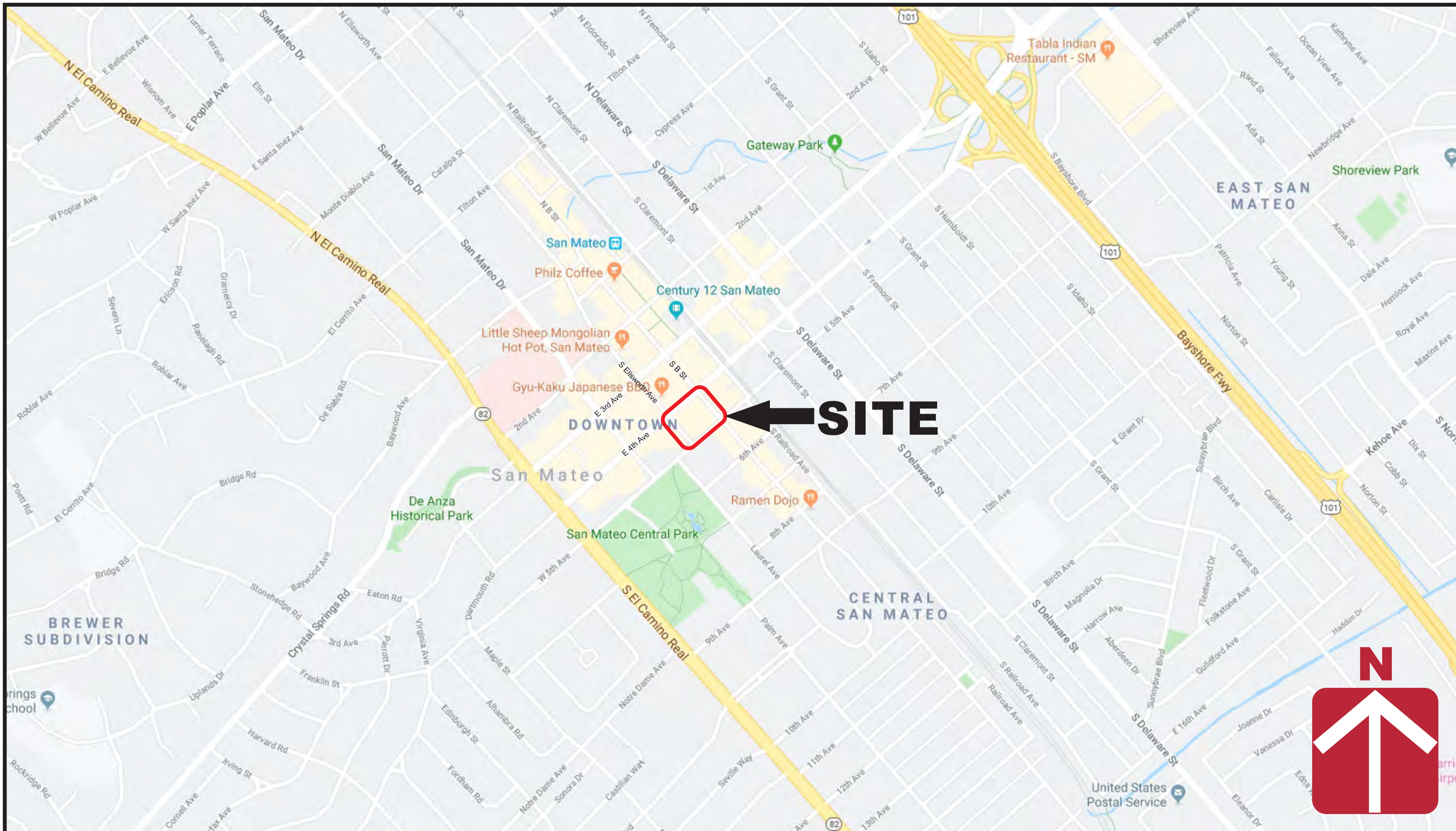
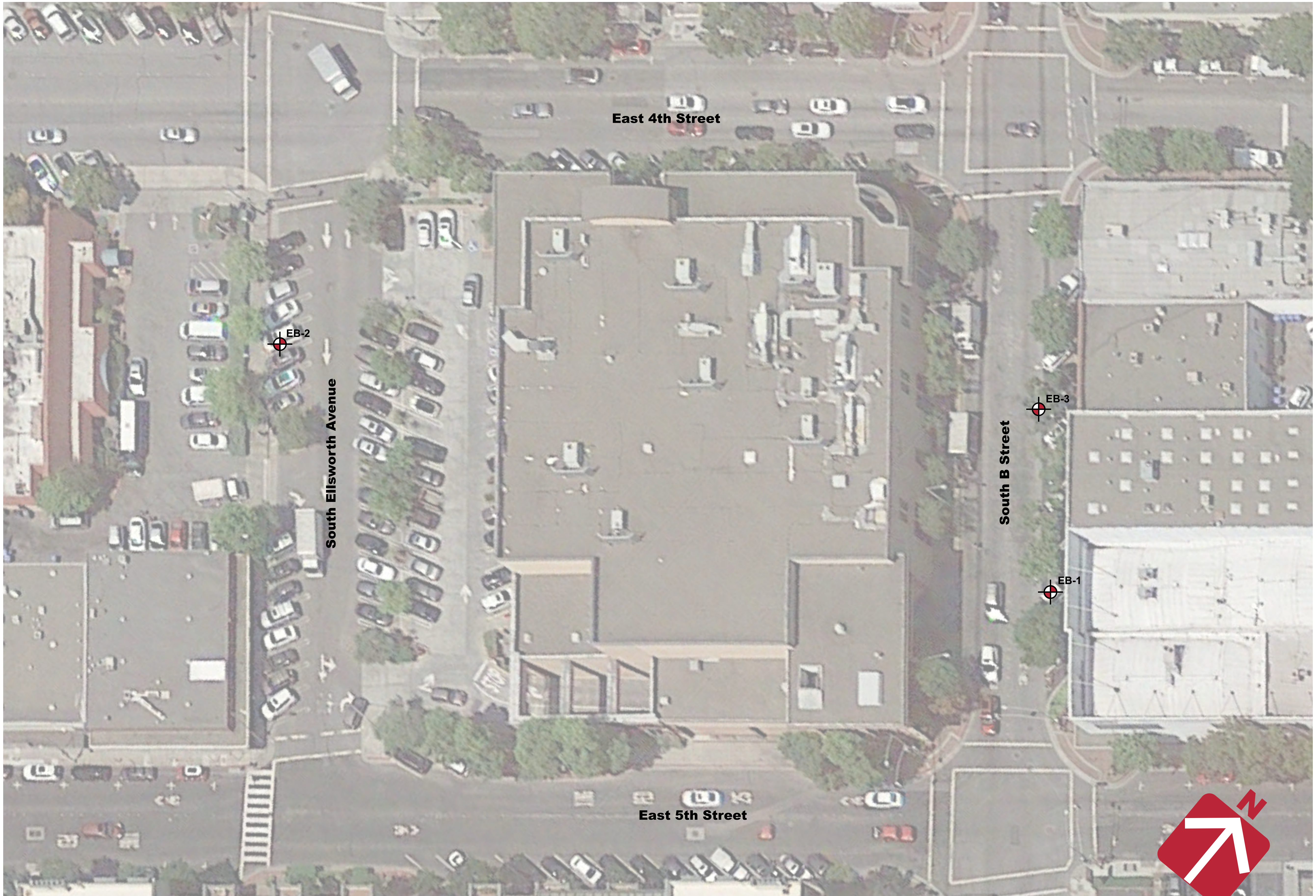


CORNERSTONE EARTH GROUP

Vicinity Map

222 East 4th Avenue
San Mateo, CA

Project Number	129-6-1
Figure Number	Figure 1
Date	December 2019
Drawn By	RRN



Legend

Approximate location of exploratory boring (EB)

0 40 80

APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 8/9/2018

Project Number 129-6-1

Figure Number Figure 2

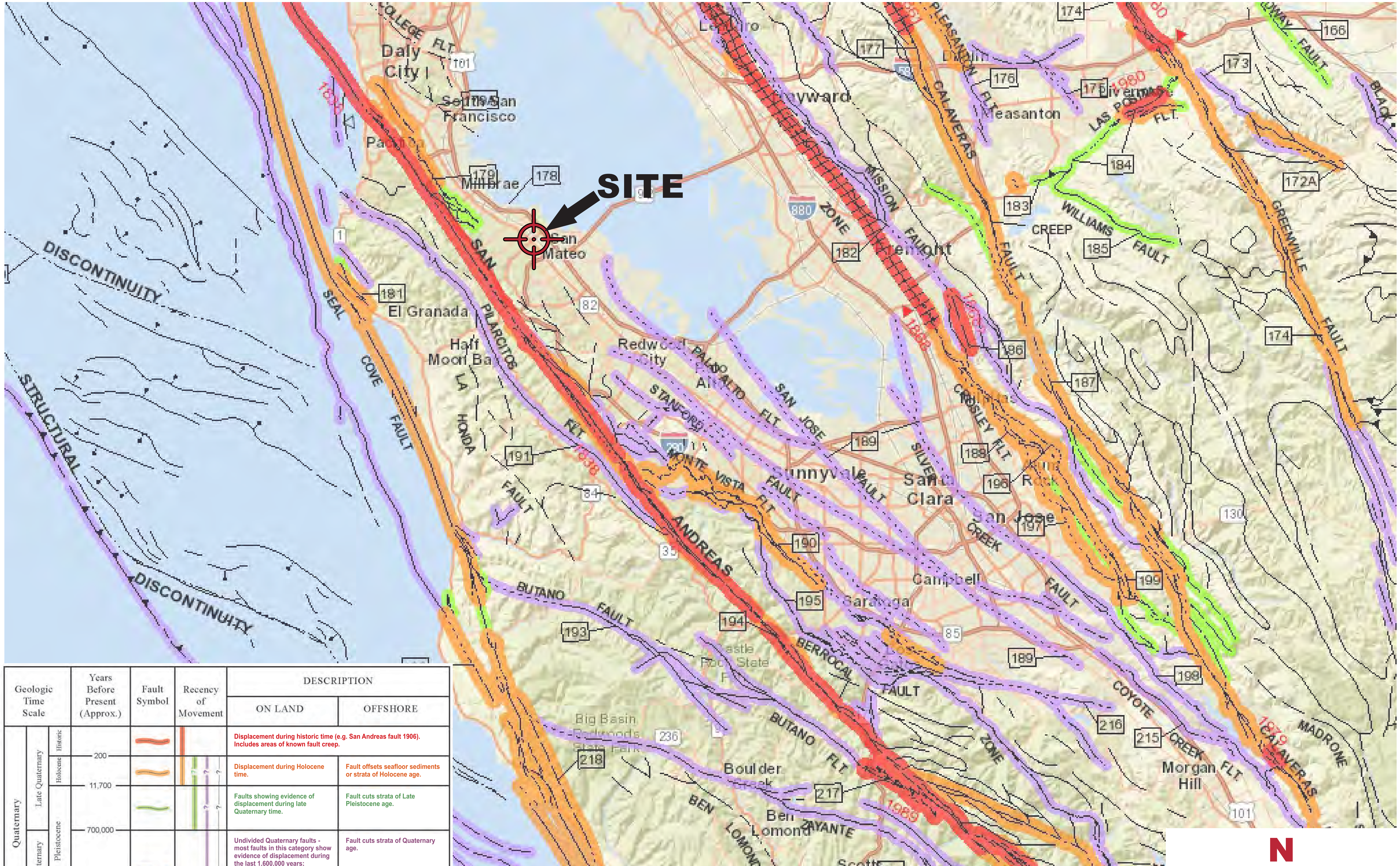
Date December 2019

Drawn By RRN

Site Plan

222 East 4th Avenue
San Mateo, CA

CORNERSTONE EARTH GROUP



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	11,700		? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	700,000		? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		1,600,000		?	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			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 129-6-1

Figure Number Figure 3

Date December 2019

Drawn By RRN

Regional Fault Map

222 East 4th Avenue
San Mateo, CA

CORNERSTONE EARTH GROUP

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem, auger drilling equipment. Two 8-inch-diameter exploratory borings were drilled on November 6 and 21, 2019 to a depth of 60 feet. The approximate locations of exploratory borings are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil, are included as part of this appendix.

Boring locations were approximated using existing site boundaries and other site features as references. Boring elevations were based on interpolation of plan contours. The locations and elevations 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.

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME 222 East 4th Avenue
PROJECT NUMBER 129-6-1
PROJECT LOCATION San Mateo, CA

DATE STARTED 11/21/19 **DATE COMPLETED** 11/21/19
DRILLING CONTRACTOR Exploration Geoservices, Inc.
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
LOGGED BY BCG
NOTES B Street

GROUND ELEVATION 25.5 FT +/- **BORING DEPTH** 60 ft.
LATITUDE 37.564387° **LONGITUDE** -122.321193°
GROUNDWATER LEVELS:
∇ **AT TIME OF DRILLING** 30 ft.
▼ **AT END OF DRILLING** 32 ft.

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

Undrained shear strength, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
25.5	0	8 inches asphalt concrete over 3 inches aggregate base							
24.8 / 24.6		**Sandy Lean Clay (CL) [Fill]** dry, reddish brown, fine to medium sand, low plasticity		GB					
				GB-2		7			
				GB					
20.5	5	**Clayey Sand with Gravel (SC)** very dense, moist, brown to reddish brown, fine to coarse sand, fine to coarse subrounded to subangular gravel	50/5"	MC-4B	115	11			
17.5		**Poorly Graded Sand with Clay and Gravel (SP-SC)** dense, moist, brown, fine to coarse sand, fine to coarse subrounded to angular gravel	66	MC-5B	128	5			
	10								
		becomes very dense	50/6"	MC-6B	111	8			
	15								
		becomes dense	73	MC-7B	125	8			
	20								
3.5		**Sandy Lean Clay with Gravel (CL)** very stiff to hard, moist, brown, fine to coarse sand, fine to coarse subrounded to subangular gravel, low plasticity	51	SPT					○
	25		50/6"	MC-9B	118	16			○ >4.5
-1.5									
-2.5									

Continued Next Page

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME 222 East 4th Avenue

PROJECT NUMBER 129-6-1

PROJECT LOCATION San Mateo, CA

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
-2.5		**Lean Clay with Sand (CL)** very stiff, moist, gray with brown mottles, fine to medium sand, trace gravel, low to moderate plasticity	75	MC-10B	108	21			
	30								
-5.5		**Sandy Lean Clay with Gravel (CL)** hard, moist, brown, fine to coarse sand, fine to coarse subangular to angular gravel, low to moderate plasticity	50/6"	MC-11B	129	16			>4.5
	35								
-11.5		**Sandy Lean Clay (CL)** very stiff, moist, brown and gray mottled, fine to medium sand, some gravel, low plasticity	52	MC-12B	112	19			
	40								
			50/6"	MC-13B	105	23			
	45								
-21.5		**Sandy Lean Clay with Gravel (CL)** very stiff, moist, brown, fine to coarse sand, fine to coarse subangular to angular gravel, low plasticity	54	SPT-14		19			
	50								
-26.5		**Clayey Sand with Gravel (SC)** very dense, moist, brown, fine to coarse sand, fine to coarse subangular to angular gravel	74	SPT-15		15			
	55								
		becomes dense	46	SPT-16		14			
-34.5	60	Bottom of Boring at 60.0 feet.							

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME 222 East 4th Avenue	
PROJECT NUMBER 129-6-1	
PROJECT LOCATION San Mateo, CA	
DATE STARTED 11/6/19	**DATE COMPLETED** 11/6/19
GROUND ELEVATION 29 FT +/-	**BORING DEPTH** 60 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.	
LATITUDE 37.564053°	**LONGITUDE** -122.322136°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger	
GROUNDWATER LEVELS:	
LOGGED BY BCG	▽ **AT TIME OF DRILLING** 29 ft.
NOTES Ellsworth Avenue	▼ **AT END OF DRILLING** 31 ft.

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

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
29.0	0	6 inches asphalt concrete over 8 inches aggregate base							
28.5									
27.8		**Clayey Sand (SC) [Fill]** moist, brown to reddish brown, fine to medium sand, some fine subangular to subrounded gravel		GB-G1		11			
24.0	5	**Clayey Sand with Gravel (SC)** medium dense, moist, brown to reddish brown, fine to coarse sand, fine to coarse subrounded to subangular gravel	44	MC-1B	128	12			
			47	MC-2B	124	9			
	10								
		becomes dense	69	MC-3B	125	11			
	15								
11.5		**Sandy Lean Clay (CL)** hard, moist, brown, fine to coarse sand, some fine gravel, low to moderate plasticity	36	SPT-4		16			>4.5 ○
	20								
6.5		**Clayey Sand with Gravel (SC)** very dense, moist, brown, fine to coarse sand, fine to coarse subangular to angular gravel	50/5"	MC-5B	134	12		17	
	25								
3.0									
1.0			44	MC-6A	113	19			○

Continued Next Page

CORNERSTONE EARTH GROUP

PROJECT NAME 222 East 4th Avenue

PROJECT NUMBER 129-6-1

PROJECT LOCATION San Mateo, CA

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

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)
1.0	30		**Sandy Lean Clay (CL)** very stiff, moist, reddish brown, fine to medium sand, fine subangular gravel, low to moderate plasticity. Liquid Limit = 32, Plastic Limit = 15	36	SPT-7		16	17		
-3.0	35		**Lean Clay with Sand (CL)** hard, moist, reddish brown, fine to medium sand, some fine subangular gravel, moderate plasticity	50/6"	MC-8B	115	20			>4.5
-8.0	40		**Sandy Lean Clay (CL)** hard, moist, reddish brown, fine to medium sand, some fine subangular gravel, low plasticity	72	MC-9B	120	17			>4.5
-13.0	45		**Clayey Sand with Gravel (SC)** very dense, moist to wet, orange-brown, fine to coarse sand, fine to coarse subangular to angular gravel	50/5"	MC-10	122	15			
-18.0	50		**Poorly Graded Sand with Clay and Gravel (SP-SC)** very dense, moist, brown, fine to coarse sand, fine to coarse subrounded to angular gravel	50/6"	SPT-11		10		12	
-22.0	55		**Sandy Lean Clay (CL)** very stiff, moist, brown, fine to medium sand, some subangular gravel, low to moderate plasticity	46	SPT-12		19			
			becomes stiff, increased sand content	65	SPT-13		20			
-31.0	60		Bottom of Boring at 60.0 feet.							

APPENDIX B: LABORATORY TEST PROGRAM

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

Moisture Content: The natural water content was determined (ASTM D2216) on 27 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 17 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.

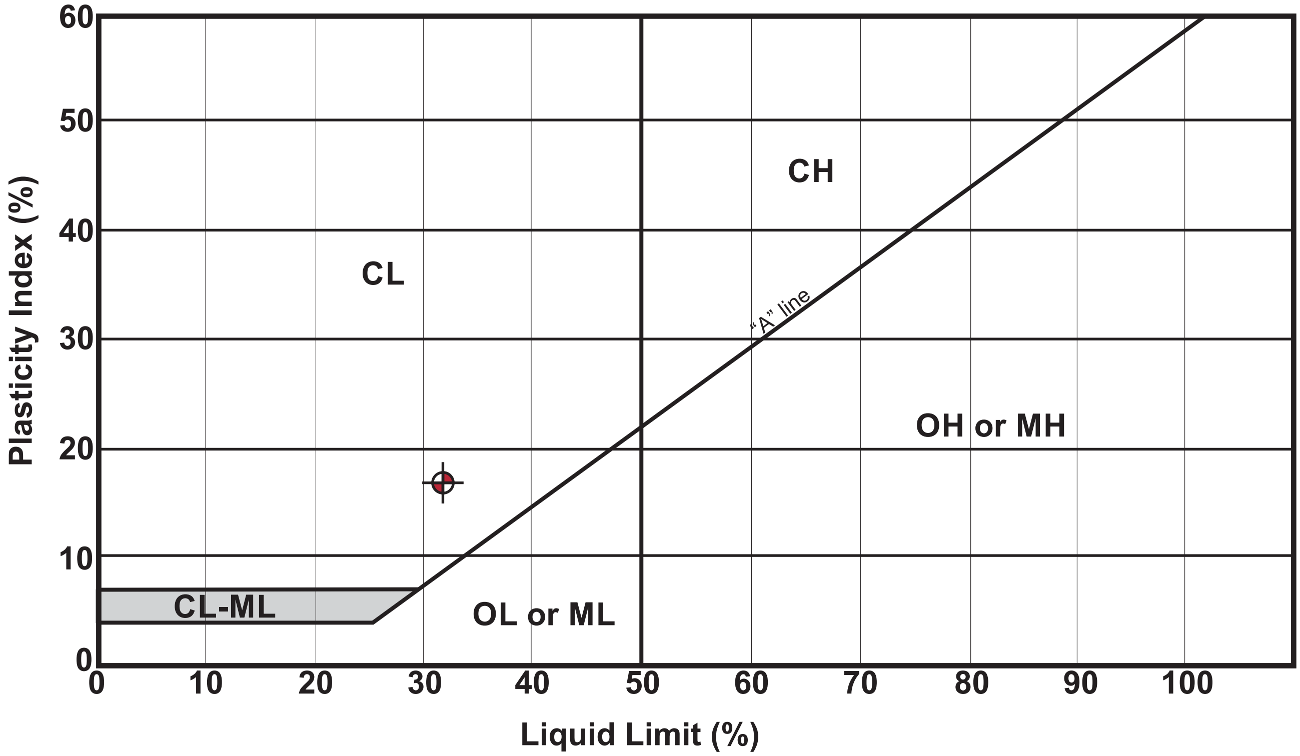
Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on two samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: One Plasticity Index determination (ASTM D4318) was performed on a sample of the subsurface soil 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 this test are shown on the boring log at the appropriate sample depth.

Undrained-Unconsolidated Triaxial Shear Strength: The undrained shear strength was determined on two relatively undisturbed samples by unconsolidated-undrained triaxial shear strength testing (ASTM D2850). The results of this test are included as part of this appendix.

Soil Corrosion: One soluble sulfate determination (ASTM D4327), one resistivity test (ASTM G57), one chloride determination (ASTM D4327), and one pH determination (ASTM G51) were performed on a sample of the subsurface soil. 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-2	28.5	16	32	15	17	—	Sandy Lean Clay (CL)

CORNERSTONE EARTH GROUP

Plasticity Index Testing Summary

222 East 4th Avenue
San Mateo, CA

Project Number 129-6-1

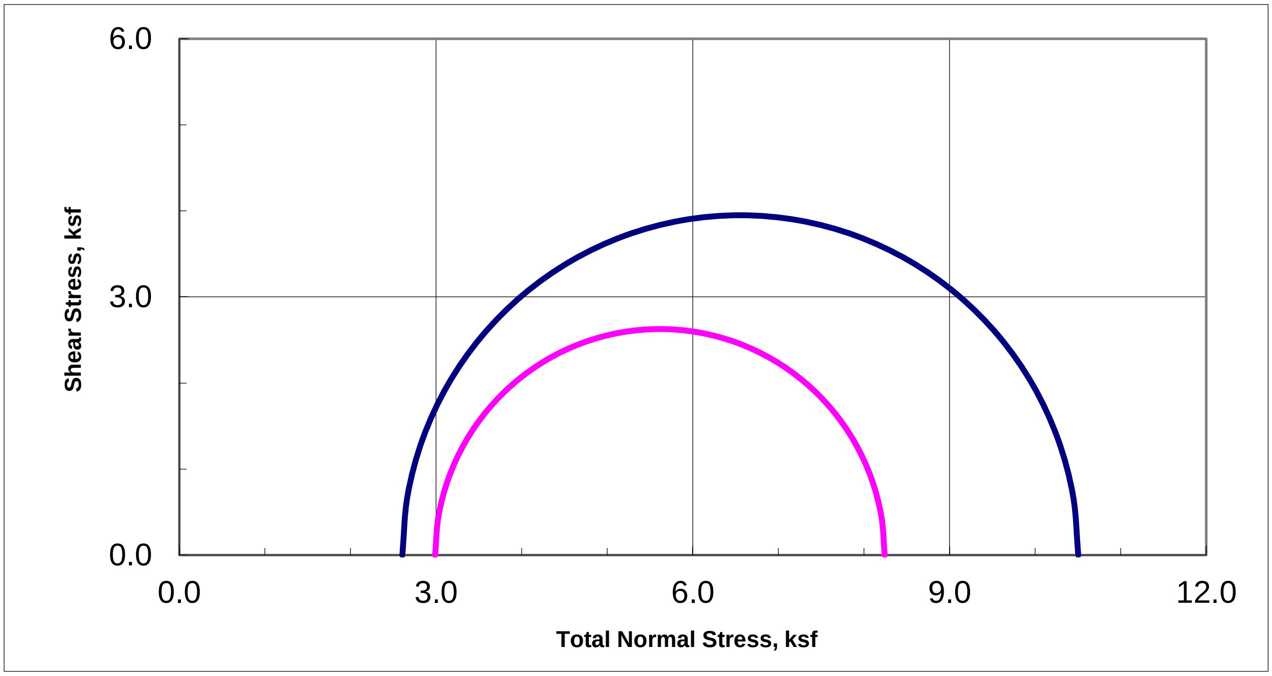
Figure Number Figure B1

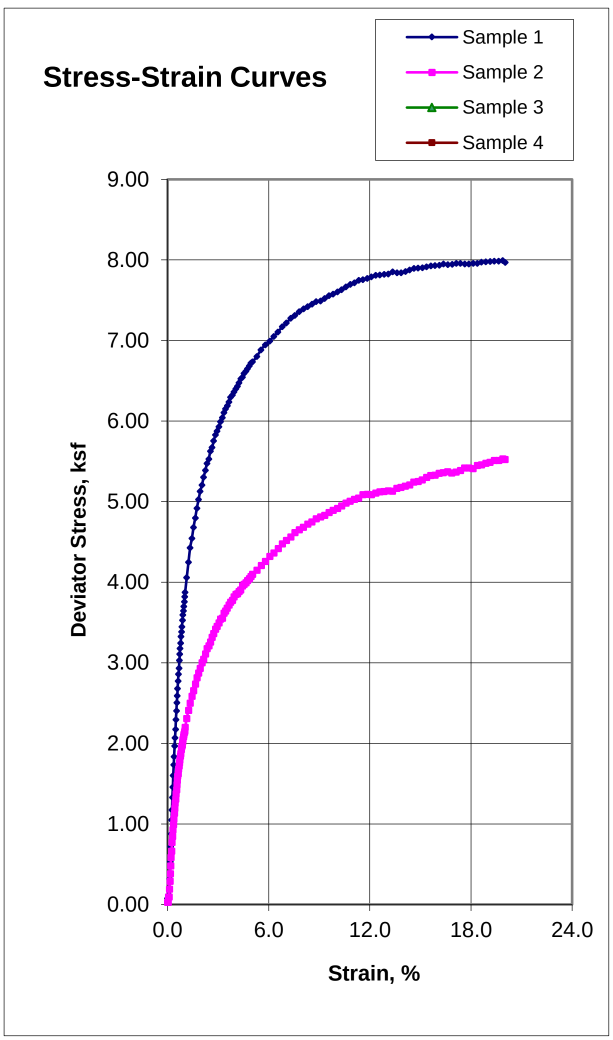
Date November 2019

Drawn By FLL

Cooper Testing Labs, Inc.
937 Commercial Street
Palo Alto, CA 94303

Unconsolidated-Undrained Triaxial Test
ASTM D2850





Sample Data	1	2	3	4
Moisture %	21.1	19.2		
Dry Den,pcf	108.3	111.9		
Void Ratio	0.585	0.534		
Saturation %	99.2	99.0		
Height in	4.99	4.98		
Diameter in	2.42	2.41		
Cell psi	18.1	20.8		
Strain %	15.00	15.00		
Deviator, ksf	7.899	5.253		
Rate %/min	1.00	1.00		
in/min	0.050	0.050		
Job No.:	640-1369			
Client:	Cornerstone Earth Group			
Project:	129-6-1			
Boring:	EB-1	EB-1		
Sample:	10B	12B		
Depth ft:	29.5	39.0		

Visual Soil Description

Sample #	
1	Yellowish Brown Sandy CLAY w/ Gravel
2	Yellowish Brown Sandy CLAY
3	
4	

Remarks:

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

Corrosivity Tests Summary

CORNERSTONE EARTH GROUP

Job Number	129-6-1	Date Tested	11/26/2019
Job Name	222 East 4th Avenue	Tested By	PWG
Location	San Mateo, CA		

Sample I.D.			Soil Visual Description	Moisture Content %	pH	Temp. at Testing C°	Resistivity (Ohm-cm) Corrected to 15.5 C°		Chloride Dry Wt. mg/kg	Sulfate Dry Wt. mg/kg
Boring	Sample No.	Depth, ft.					As Received	Saturated		
				ASTM D2216	ASTM G51		G57	ASTM G57	ASTM D4327	ASTM D4327
EB-1	9A	25.0	Brown Sandy Lean Clay (CL)	16.4	6.7	19.7	15,139	3,304	5	10