APPENDIX F
GEOTECHNICAL INVESTIGATION

CORNERSTONE
EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**1128 Douglas Avenue Apartments**
Location	**1128 and 1132 Douglas Avenue** **Burlingame, California**
Client	**Dreiling Terrones Architecture, Inc.**
Client Address	**1103 Juanita Avenue** **Burlingame, CA**
Project Number	**745-3-1**
Date	**June 19, 2015**

Prepared by

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CORNERSTONE EARTH GROUP

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SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Dreiling Terrones Architecture, Inc. for the 1128 Douglas Avenue Apartments project in Burlingame, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided a set of architectural and landscaping plans titled "New 29 Unit Apartment Building, 1128 Douglas Avenue," prepared by Dreiling Terrones Architecture Inc., dated January 21, 2015.

1.1 PROJECT DESCRIPTION

The project site is located at 1128 and 1132 Douglas Avenue in Burlingame, California, and is currently occupied by two single family homes and a small apartment complex building. The project will consist of demolishing the existing buildings and constructing a new five-story wood-framed apartment podium structure, with one-level below grade parking garage. The building will have a footprint of approximately 7,654 square feet and the parking garage floor elevation will be approximately 10 to 11 feet below ground surface. Appurtenant drive aisles and parking, walkways, utilities, landscaping, and other improvements necessary for site development are also planned.

Structural loads are not known at this time, however loads are anticipated to be typical for this type of structure, which are about 300 to 400 kips dead plus live load for isolated columns and 4 to 6 kips per lineal foot for dead plus live load at the exterior walls. We assume the below grade parking will require cuts on the order of 11 to 14 feet below the ground surface.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated June 17, 2014, which was authorized on April 7, 2015 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 May 6, 2015 with truck-mounted, hollow-stem auger drilling equipment. The borings were drilled to depths ranging from 18½ to 35 feet. The borings were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

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

1.4 LABORATORY TESTING PROGRAM

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

1.5 ENVIRONMENTAL SERVICES

Our firm is currently preparing a Phase 1 environmental site assessment, which will be presented under a separate report. If environmental concerns are determined to be present during future evaluations, the project team should review our geotechnical recommendations for compatibility with the environmental concerns.

SECTION 2: REGIONAL SETTING

2.1 REGIONAL SEISMICITY

The San Francisco Bay area 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 2007 estimates 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. As seen with damage in San Francisco and Oakland due to the 1989 Loma Prieta earthquake that was centered about 50 miles south of San Francisco, significant damage can occur at considerable distances. Higher levels of shaking and damage would be expected for earthquakes occurring at closer distances.

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)	2.7	4.3
San Gregorio	9.2	14.8
Molly Vista-Shannon	11.2	18.0

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 site is bounded by Douglas Avenue on the east and residential development on the north, south, and west. Two single-family homes and a small apartment complex occupy the relatively level site. The buildings are surrounded by various landscaping areas and asphalt-paved and gravel driveways.

Surface pavement at boring EB-1 consisted of 2 inches of asphalt concrete over 2½ inches of aggregate base. Based on visual observations, the existing pavement is in poor to moderate condition.

3.2 SUBSURFACE CONDITIONS

Below the surface pavements, our explorations encountered stiff to very stiff lean clays with variable amounts of sand to depths of 7½ and 10½ feet below the surface. Below the surficial clays alternating layers of medium dense to very dense clayey sands and very stiff clays with variable amounts of sand were encountered to the maximum depth explored of 18½ and 35 feet in borings EB-1 and EB-2, respectively.

3.2.1 Plasticity/Expansion Potential

We performed one Plasticity Index (PI) test on a representative sample to evaluate the expansion potential of the surficial soils. The result of the surficial PI test indicated a PI of 27, indicating moderate to high expansion potential to wetting and drying cycles. It is noted that the materials anticipated to be exposed at the basement level are clayey sands and are anticipated to have low expansion potential.

3.2.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 15 feet range from near optimum moisture to approximately 5 to 10 percent over the estimated laboratory optimum moisture.

3.3 GROUND WATER

Ground water was not encountered in EB-1 but was encountered in EB-2 at a depth of 15 feet below the surface. 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.

Ground water levels are not currently mapped by the State of California, however, we reviewed on-line data from the Geotracker Website regarding ground water depths in the site area. Monitoring well data by 260 El Camino Real (approximately 1250 feet southeast of the site), by 215 California Drive (approximately 1500 feet northeast of the site, and by 1480 Broadway (approximately 4900 feet northwest of the site) indicate static ground water levels ranging seasonally from approximately 8 to 16 feet below the ground surface.

It is our opinion that ground water could be encountered during construction at depths ranging up to approximately 8 feet below current grades. Therefore, for design purposes, we recommend a depth to ground water to be 7 feet below existing ground surface, which includes 1 foot of "free board" on the anticipated historical high ground water level in the site vicinity.

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

3.4 PRELIMINARY CORROSION SCREEING

We tested two samples collected at depths of 3½ and 9 feet for resistivity, pH, soluble sulfates, and chlorides. The laboratory test results are summarized in Table 2.

Table 2: Summary of Corrosion Test Results

Boring	Depth (feet)	Soil pH[1]	Resistivity[2] (ohm-cm)	Chloride[3] (mg/kg)	Sulfate[4,5] (mg/kg)
EB-1	9	7.8	5,852	6	9
EB-2	3½	7.4	1,264	4	28

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.

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

In accordance with the 2013 CBC Section 1904A.1, alternative cementitious materials for sulfate exposure shall be determined in accordance with ACI 318-11 Table 4.2.1 and Table 4.3.1. Based on the laboratory test results, no cement type restriction is required, although, in our opinion, it is generally a good practice to include some sulfate resistance and to maintain a relatively low water-cement ratio. We have summarized applicable design values and parameters from ACI 318 Table 4.3.1 below in Table 3.

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

Table 3: ACI Sulfate Soil Corrosion Design Values and Parameters

Category	Water-Soluble Sulfate (SO4) in Soil (% by weight)	Class	Severity	Cementitious Materials
S, Sulfate	< 0.10	S0	not applicable	no type restriction

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

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

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration (PGA) was estimated for analysis using a value equal to F_{PGA} x PGA, as allowed in the 2013 edition of the California Building Code. For our liquefaction analysis we used a PGA of 0.811g.

4.3 LIQUEFACTION POTENTIAL

The site is not currently mapped by the State of California, but is within a zone mapped as having a moderate liquefaction potential by the Association of Bay Area Governments. However, we screened the site for liquefaction during our site exploration by retrieving samples from the site, performing visual classification on sampled materials, and performing various tests to further classify the soil properties.

4.3.1 Background

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

4.3.2 Analysis

As discussed in the “Subsurface” section above, several sand layers were encountered below the design ground water depth of 7 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 laboratory testing on samples retrieved from our borings. SPT “N” values obtained from hollow-stem auger borings were used in our analyses and corrected for effective overburden stresses. Soils that have corrected SPT blow counts greater than 30 blows per foot are considered too dense to liquefy and have been screened out of our analysis.

4.3.3 Summary

Our analysis indicates that the sand layers would not be expected to experience liquefaction. Therefore, we conclude the liquefaction potential is very low at this site based on our explorations.

4.3.4 Ground Rupture Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground rupture or sand boils. For ground rupture 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. Because there is not a potential for liquefaction to occur at the site, ground rupture is not anticipated to be an issue.

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.

There are no open faces within a distance considered susceptible to lateral spreading; therefore, in our opinion, the potential for lateral spreading to affect the site is not a concern for this project.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

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

4.6 TSUNAMI/SEICHE

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

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

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the 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 ¾ miles inland from the San Francisco Bay shoreline, and is approximately 24 feet above mean sea level. Additionally, the site is mapped by the State of California Tsunami Inundation Map as not being within an inundation area. 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 with a 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood." 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.

- Shallow ground water
- Presence of moderately to highly expansive soils
- Differential movement at on-grade to on-structure transitions
- Soil Corrosion Potential

5.1.1 Shallow Ground Water

Ground water was measured at a depth of 15 feet below the existing ground surface in one of our borings. Additionally, monitoring well data within the site vicinity indicate static water levels seasonally ranging from 8 to 16 feet below the ground surface at the well locations as discussed in Section 3.3.

The proposed below-grade garage floor level for the podium structure appears to extend to approximately 10 to 11 feet below grade and the mat foundation will likely extend to a depth of approximately 12 to 14 feet below grade. As monitoring wells within the site vicinity indicate seasonal ground water fluctuating between 8 and 16 feet below grade, there is a potential that ground water will be encountered during underground grading and construction. Impacts associated with the high ground water typically consist of potentially wet and unstable subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering and shoring of the proposed below-grade parking level and utility trenches would likely be required on the site for below-grade excavations extending below about 8 to 10 feet. The contractor should include provisions for controlling ground water and the temporary shoring design will need to include surcharge pressures for ground water. Detailed recommendations addressing this concern are presented in the "Earthwork" section of this report.

Shallow ground water will also present several design challenges for the permanent structure. We recommend that a design ground water level of 7 feet below the existing ground surface be used to design the structure. Because the planned garage level will likely be below seasonal ground water levels, draining the garage walls and lower level slab would require an expensive full-time dewatering system. Therefore, we recommend waterproofing the below-grade walls, and designing the mat foundation and garage walls, including construction joints, to resist hydrostatic pressure. In our opinion, it may make sense to drain the garage walls above or slightly below the design ground water level for a more efficient wall design above that elevation, and as a precaution against higher than expected uplift forces for the structure.

5.1.2 Presence of Moderately to Highly Expansive Soils

As discussed, moderately to highly expansive surficial soils generally blanket the site. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. To reduce the potential for damage to the planned structures at the ground surface, slabs-on-grade should have sufficient reinforcement and be supported on a layer of non-expansive fill; footings should extend below the zone of seasonal moisture fluctuation. At-grade flatwork should also be supported on a layer of non-expansive fill. In addition, it is important to limit moisture changes in the surficial soils by using positive drainage away from buildings as well as limiting landscaping watering. Detailed grading and foundation recommendations addressing this concern are presented in the following sections.

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

Some of the surficial improvements will transition from on-grade support to overlying the basement. 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 cantilever 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 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, we conclude that the use of sulfate resistant concrete is not required for buried concrete; however, the corrosion potential for buried metallic structures, such as metal pipes, is considered mildly to severely corrosive. As the preliminary soil corrosion screening was based on the results of limited sampling, consideration may be given to collecting and testing additional samples from the upper 10 feet and to consult with a corrosion engineer to confirm the classification.

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, CLEARING AND PREPARATION

6.1.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements within the proposed at-grade development area. Demolition of existing improvements is discussed in detail below. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Based on our site observations, surficial stripping should extend about 3 to 6 inches below existing grade in vegetated areas.

6.1.2 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. Near surface at-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. It is noted that the tree roots within the limits of the basement should be removed as part of the basement excavation.

6.1.3 Demolition of Existing Slabs, Foundations and Pavements

All slabs, foundations, and pavements should be completely removed from within planned building areas, which is anticipated due to the basement excavations for this project. 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 provide subsurface drainage. A discussion of recycling existing improvements is provided later in this report.

6.1.4 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 risks 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. In general, the risk is relatively low for single utility lines less than 4 inches in diameter, and increases with increasing pipe diameter.

6.2 REMOVAL OF EXISTING FILLS

While fills were not encountered in our borings, any fills that are not removed by the basement excavation 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 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.

6.3 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 B materials. Below 10 feet, the soils should be considered 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 slope at a 1:1 inclination unless the OSHA soil classification indicates otherwise.

6.4 BELOW-GRADE EXCAVATIONS

The bottom of the garage excavation will likely consist of saturated native soils, and a stable working surface will most likely be required, consisting of at least 12 to 18 inches of clean crushed rock. The final thickness of crushed rock needed should be based on the judgment of the contractor and the type of equipment and material loading that is likely to occur. As an alternative, chemical treatment may be feasible to stabilize the bottom of the excavation. Heavy rubber-tired vehicles, such as concrete trucks, are unlikely to be able to access the bottom of the excavation without stabilized access. Destabilized or disturbed areas will require repair using methods approved by the geotechnical engineer.

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 about 14 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.

6.4.1 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. We recommend that heavy construction loads (cranes, etc.) and material stockpiles be kept at least 15 feet behind the shoring. Where this loading cannot be set back, the shoring will need to be designed to support the loading. The shoring designer should provide for timely and uniform mobilization of soil pressures that will not result in excessive lateral deflections. Minimum suggested geotechnical parameters for shoring design are provided in the table below.

Table 4: 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 – Trapezoidal Earth Pressure	**Increase from 0 to 25H* psf
Passive Pressure – Starting at 2 feet below the bottom of the excavation	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
** The cantilever and restrained pressures are for drained designs with dewatering. If undrained shoring is designed, an additional 40 pcf should be added for hydrostatic pressures. For the purposes of design of the temporary shoring system, we would advise a design ground water level of 10 feet below the ground surface. If a period of more than two years transpires from the date of this report and the date of shoring system construction, this should be reviewed and revised, if necessary.

The restrained earth pressure may also be distributed as described in Figure 23 of the *FHWA Circular No. 4 – Ground Anchors and Anchored Systems* (with the hinge points at ¼H and ¾H) provided the total pressure is established from the uniform pressure above.

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

We performed our borings with hollow-stem auger drilling equipment and as such were not able to evaluate the potential for caving soils, which can create difficult conditions during soldier beam, 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.

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. The monitoring frequency should be established and agreed 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.

6.4.2 Construction Dewatering

Ground water levels are expected to potentially be about 4 to 7 feet above the planned excavation bottom depending on the time of year of construction; therefore temporary dewatering may be necessary during construction. Prior to the start of construction, we would recommend excavation of potholes to evaluate the depth of ground water. Design, selection of the equipment and dewatering method, and construction of temporary dewatering should be the responsibility of the contractor. Modifications to the dewatering system are often required in layered alluvial soils and should be anticipated by the contractor. The dewatering plan, including planned dewatering well filter pack materials, should be forwarded to our office for review prior to implementation.

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

Temporary draw down of the ground water table can cause the subsidence outside the excavation area, causing settlement of adjacent improvements. We should be retained to evaluate the potential settlements of the dewatering system. If settlements are deemed excessive for adjacent improvements, we recommend revising the dewatering plan including but not limited to alternative shoring methods such as tied back slurry walls or soil mixed curtain walls.

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

6.5 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.

The subgrade for any mat foundation/thickened slab extending to or below ground water (i.e. the basement level) should generally be cut to the desired grades, including the thickness for any subgrade stabilization, as discussed below.

6.6 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 near optimum moisture to about 10 percent over the estimated laboratory optimum in the upper 15 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.

Even presuming that temporary dewatering will be included for the below-grade garage excavation, the soils above the depressed water table will be nearly saturated and will be wet and difficult to work with.

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 particular site conditions.

6.6.1 Scarification and Drying

The subgrade may be scarified to a depth of 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 geosynthethic (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 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.

6.6.4 Below-Grade Excavation Stabilization

As the planned basement excavation will extend below the design ground water level, we recommend that the contractor plan for stabilization of the excavation bottom where construction traffic is planned. This may include chemical treatment of 12 to 18 inches of the subgrade, depending on the method of mass excavation and disturbance to the excavation bottom, or alternatively excavating an additional 12 to 18 inches below subgrade, placing a layer of stabilization fabric (Mirafi 500X or approved equivalent) at the bottom, and backfilling with clean, crushed rock. The crushed rock should be consolidated in place with light vibratory equipment. Rubber-tired equipment should not be allowed to operate on the exposed subgrade; the crushed rock should be stockpiled and pushed out over the stabilization fabric.

A Cornerstone representative should observe the excavation bottom, including potentially observing proof rolling, prior to placing a rat slab and waterproofing to provide more detailed stabilization and subgrade repair recommendations.

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 oversized 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 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 structures footprint 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.

6.7.3 Non-Expansive Fill Using Lime Treatment

As discussed above, non-expansive fill should have a Plasticity Index (PI) of 15 or less. As an alternative to importing non-expansive fill, chemical treatment can be considered to create non-expansive fill. It has been our experience that high PI clayey soil materials will likely need to be mixed with at least 3 to 4 percent quicklime (CaO) or approved equivalent to adequately reduce the PI of the on-site soils to 15 or less. In our opinion, this site is not big enough to make chemical treatment an economical option for the at-grade soils. If this option is considered, additional laboratory tests should be performed during initial site grading to further evaluate the optimum percentage of quicklime required.

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

Table 5: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Expansive Soils	87 – 92	>3
	Low Expansion Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Expansive Soils	95	>3
	Low Expansion Soils	95	>1
Basement Wall Backfill	Without Surface Improvements	90	>1
Basement Wall Backfill	With Surface Improvements	95[4]	>1
Trench Backfill	On-Site Expansive Soils	87 – 92	>3
Trench Backfill	Low Expansion Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Low Expansion Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum
Flatwork Subgrade	On-Site Expansive Soils	87 - 92	>3
Flatwork Subgrade	Low Expansion Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Expansive Soils	87 - 92	>3
Pavement Subgrade	Low Expansion 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)
4 – Using light-weight compaction or walls should be braced

6.8.1 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).

Excavations extending to or below ground water will also be likely to destabilize under construction equipment loading. Long exposure to summer months may allow the subgrade to significantly dry out. If expansive soils are allowed to dry out significantly, re-moisture conditioning may require several days of gradual re-wetting (flooding is not recommended), or deep scarification, moisture conditioning, and re-compaction. Consideration may be given to preparing subgrade just prior to crushed rock placement if significant drying and re-moisture conditioning is anticipated.

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.

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.

6.10 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 on splash blocks or 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. We should review their locations to evaluate and provide recommendations to mitigate the impacts on the proposed structure.

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 ground water is not mapped in the area, but was encountered as high as 15 feet below grade in one of our borings. Additionally, monitoring well information within the site vicinity indicate seasonal ground water levels at about 8 to 16 feet below existing grades at the well locations.

- In our opinion, infiltration locations within 10 feet of the buildings would create a geotechnical hazard.

- Infiltration measures, devices, or facilities may conflict with the location of existing or proposed underground utilities or easements. Infiltration measures, devices, or facilities should not be placed on top of or very near to underground utilities such that they discharge to the utility trench, restrict access, or cause stability concerns.

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 required, infiltration (percolation) testing should be performed on representative samples of potential bioswale materials prior to construction to check for general conformance with the specified infiltration rates.

- It should be noted that multiple laboratory tests may be required to evaluate the properties of the bioswale materials, including percolation, landscape suitability and possibly environmental analytical testing depending on the source of the material. We recommend that the landscape architect provide input on the required landscape suitability tests if bioswales are to be planted.

- 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.

- If required by governing agencies, field infiltration testing should be specified on the grading and improvement plans. The appropriate infiltration test method, duration and frequency of testing should be specified in accordance with local requirements.

- 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.

6.12 LANDSCAPE CONSIDERATIONS

Since the near-surface soils are moderately to highly expansive, we recommend greatly reducing the amount of surface water infiltrating the 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 slopes,
- Regulating the amount of water distributed to lawns or planter areas by using irrigation timers, and
- 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, as discussed in Section 5, the proposed structure may be supported on a mat foundation system designed for hydrostatic pressures provided the recommendations in the "Earthwork" section and the sections below are followed.

7.2 SEISMIC DESIGN CRITERIA

The project structural design should be based on the 2013 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The "Seismic Coefficients" used to design buildings are established based on a series of tables and figures addressing different site factors, including the 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 boring and review of local geology, the site is underlain by 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 USGS computer program *Design Maps*, located at http://geohazards.usgs.gov/designmaps/us/application.php, based on the site coordinates presented below and the site classification. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 6: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.58055°
Site Longitude	-122.34988°
0.2-second Period Mapped Spectral Acceleration[1], S_S	2.077g
1-second Period Mapped Spectral Acceleration[1], S_1	0.982g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv	1.5

[1]For Site Class B, 5 percent damped.

Table 6 continues

Table 6 Continued

Classification/Coefficient	Design Value
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	2.077g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.474g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.384g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.982g
Mapped MCE Geometric Mean Peak Ground Acceleration - PGA	0.811g
Site Coefficient Based on PGA and Site Class - F_{PGA}	1.0

[1]For Site Class B, 5 percent damped.

7.3 REINFORCED CONCRETE MAT FOUNDATIONS

7.3.1 Mat Foundation Bearing Pressures

Due to the magnitude of potential hydrostatic uplift, the structure should 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.

All mats may be designed for an average allowable bearing pressure of 1000 pounds per square foot (psf) for dead plus live loads, with maximum localized allowable bearing pressures of 2,500 psf at column or wall loads. Allowable bearing pressures may be increased by one-third for all loads including wind or seismic. These allowable bearing pressures are net values; the weight of the mat can be neglected for design purposes. All mats should be reinforced with top and bottom steel, or as determined appropriate by the structural engineer, to provide structural continuity and to help span local irregularities. These recommendations may be revised depending on the particular design method selected by the structural engineer.

The bottom of mats will extend below the design ground water level. Therefore, as presented in the “Hydrostatic Uplift and Waterproofing” section of this report, the mats should be waterproofed.

7.3.2 Mat Foundation Settlement

Based on the above loading and the allowable bearing pressures presented above, we estimate total static mat settlement of up to ½ inch will occur, with post-construction static differential settlement of about ¼ inch over a distance of about 30 feet.

As the structural engineer performs design of the mat foundations, we should be retained to review the analysis, confirm our settlement estimates, and provide a modulus of subgrade reaction based on actual mat contact pressures, if needed.

7.3.3 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 0.40 applied to the mat dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 400 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.

7.3.4 Mat Foundation Construction Considerations

The mat will be constructed near the current ground water level and even if temporary dewatering is included, the soil above the water table will be at near saturated conditions. Subgrade stabilization may be required as discussed in the "Earthwork" section above.

7.4 HYDROSTATIC UPLIFT AND WATERPROOFING

As previously discussed, it is our opinion that ground water could be encountered during construction at depths ranging from approximately 8 to 16 feet below current grades. However, for design purposes including hydrostatic uplift and waterproofing, we recommend a design depth to ground water to be 7 feet based on available data.

Where portions of the structures are constructed near the design ground water level, including bottoms of slabs-on-grade and mat foundations, they should be designed to resist potential hydrostatic uplift pressures. Retaining walls extending below design ground water should be waterproofed and designed to resist hydrostatic pressure for the full wall height. Where portions of the walls extend above the design ground water level, a drainage system may be added as discussed in the "Retaining Wall" section, if desired; otherwise the walls should be designed as undrained for the full height. It may be necessary to construct a "Rat Slab" as part of the water proofing system.

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

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 INTERIOR SLABS-ON-GRADE AT EXISTING GRADE LEVELS

As the Plasticity Index (PI) of the surficial soils ranges up to 27, any proposed on-site at-grade slabs-on-grade should be supported on at least 12 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. Improvements in the City of Burlingame right of way should be design in accordance with the City of Burlingame standard plans. The NEF layer should be constructed over 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 (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 in accordance with the recommendations in the "Earthwork" section of this report.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying soils. 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.

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-thick 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 ½- to ¾-inch crushed rock with less than 5 percent passing the No. 200 sieve, should be placed below the vapor retarder and consolidated in place with vibratory equipment. The capillary break rock may be considered as the upper 4 inches of the non-expansive fill previously recommended.

- 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.

8.3 EXTERIOR FLATWORK

Exterior slabs-on-grade, such as pedestrian walkways, patios, driveways, and sidewalks, may experience seasonal movement due to the native expansive soils; therefore, some cracking or vertical movement of conventional slabs should be anticipated where imported fill is not planned in flatwork areas. There are several alternatives for mitigating the impacts of expansive soils beneath concrete flatwork. We are providing recommendations to reduce distress to concrete flatwork that includes moisture conditioning the subgrade soils, using non-expansive fill, and providing adequate construction and control joints to control cracks that do occur. It should be noted that minor slab movement or localized cracking and/or distress could still occur.

- The minimum recommendation for concrete flatwork constructed on moderately to highly expansive soils is to properly prepare the clayey soils prior to placing concrete. This is typically achieved by scarifying, moisture conditioning, and re-compacting the subgrade soil. Subgrade soil should be moisture conditioned to at least 3 percent over the laboratory optimum and compacted using moderate compaction effort to a relative compaction of 87 to 92 percent (ASTM Test Method D1557). Since the near surface soils may have been previously compacted and tested, the subgrade soils could possibly be moisture conditioned by gradually wetting the soil, depending on the time of year slab construction occurs. This should not include flooding or excessively watering the soil, which would likely result in a soft, unstable subgrade condition, and possible delays in the construction while waiting for the soil to dry out. In general, the subgrade should be relatively firm and non-yielding prior to construction.

- Concrete flatwork, excluding pavements that would be subject to wheel loads, should be at least 4 inches thick and underlain by at least 6 inches of non-expansive fill (NEF). Non-expansive fill may include aggregate base, crushed rock, or imported soil with a PI of 15 or less. In addition, the upper 4 inches of NEF should also meet Class 2 aggregate base requirements. As an alternative, the Class 2 aggregate base can also be extended the full depth of NEF as recommended above. Non-expansive fill should be compacted to at least 90 percent relative compaction. 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.

- We recommend a maximum control joint spacing of about 2 feet in each direction for each inch of concrete thickness and a construction joint spacing of 10 to 12 feet. Construction joints that abut the foundations and retaining walls should include a felt strip, or approved equivalent, that extends the full depth of the exterior slab. This will help to reduce the potential for permanent vertical offset between the slabs due to friction between the concrete edges. We recommend that exterior slabs be isolated from adjacent foundations.

At the owner’s option, if desired to reduce the potential for vertical offset or widening of concrete cracks, consideration should be given to using reinforcing steel, such as No. 3 rebar spaced at 18 inches on center each direction.

SECTION 9: VEHICULAR PAVEMENTS

9.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 5. The design R-value was chosen based on engineering judgment considering the variable surface conditions.

Table 7: Asphalt Concrete Pavement Recommendations, Design R-value = 5

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.5	10.0
4.5	2.5	9.0	11.5
5.0	3.0	10.0	13.0
5.5	3.0	11.5	14.5
6.0	3.5	12.0	15.5
6.5	4.0	12.0	17.0

*Caltrans Class 2 aggregate base; minimum R-value of 78
*Caltrans Class 2 aggregate base; minimum R-value of 78

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 be using the pavements.

Asphalt concrete pavements constructed on expansive subgrade where the adjacent areas will not be irrigated for several months after the pavements are constructed may experience longitudinal cracking parallel to the pavement edge. These cracks typically form within a few feet of the pavement edge and are due to seasonal wetting and drying of the adjacent soil. The cracking may also occur during construction where the adjacent grade is allowed to significantly dry during the summer, pulling moisture out of the pavement subgrade. Any cracks that form should be sealed with bituminous sealant prior to the start of winter rains. One alternative to reduce the potential for this type of cracking is to install a moisture barrier at least 24 inches deep behind the pavement curb.

9.2 PORTLAND CEMENT CONCRETE

The exterior Portland Cement Concrete (PCC) pavement recommendations tabulated below are based on methods presented in the Portland Cement Association (PCA) design manual (PCA, 1984). We have provided a few pavement alternatives as an anticipated Average Daily Truck Traffic (ADTT) was not provided. An allowable ADTT should be chosen that is greater than what is expected for the development.

Table 8: PCC Pavement Recommendations, Design R-value = 5

Allowable ADTT	Minimum PCC Thickness (inches)
13	5.5
130	6.0

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi, supporting the PCC on at least 6 inches of Class 2 aggregate base compacted as recommended in the "Earthwork" section, 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. Due to the expansive surficial soils present, we recommend that the construction and expansion joints be dowelled.

9.3 PAVEMENT CUTOFF

Surface water penetration into the pavement section can significantly reduce the pavement life, due to the expansive clays. While quantifying the life reduction is difficult, a normal 20-year pavement design could be reduce 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 10: RETAINING WALLS

10.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall. 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 9: 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.

10.2 SEISMIC LATERAL EARTH PRESSURES

The 2013 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. We reviewed the seismic earth pressures for the proposed basement using procedures generally based on the Mononobe-Okabe method. Because the walls are likely greater than 10 to 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. Because the wall is restrained, or will act as a restrained wall, and will be designed for 45 pcf (equivalent fluid pressure) plus a uniform earth pressure of 8H psf, based on current recommendations for seismic earth pressures (Lew et al., SEAOC 2010), it appears that active earth pressures plus a seismic increment do not exceed the fixed wall earth pressures. Therefore, in our opinion, an additional seismic increment above the design earth pressures is not required as long as the walls are designed for the restrained wall earth pressures recommended above.

10.3 PERMANENT POST-GROUTED TIEBACK ANCHORS

We are providing permanent tieback anchor design parameters if they are desired. The restrained lateral earth pressures provided above should be used to develop anchor design loads. The structural engineer should apply an appropriate factor of safety for static and seismic loading conditions. We suggest a minimum of 2.0 for static conditions and 1.5 for seismic conditions.

Based on information in FHWA's Geotechnical Engineering Circular No. 4 (1999), "Ground Anchors and Anchored Systems", the unbonded length should extend a minimum distance of H/5 (H is the height of the retaining wall) past a failure plan of 30° (degrees). For anchor bond capacity, an ultimate bond stress of 1,450 psf for post-grouted anchors, corresponding to the lower end of FHWA-IF-99-015, Table 7, indicating estimated bond strength for medium dense to dense sands of 0.08 to 0.38 MPa (approximately 1,650 to 7,900 psf) and for very stiff clays of 0.07 to 0.17 MPa (approximately 1,450 to 3,550 psf). All anchors should be load tested to confirm design capacity in accordance with FHWA recommendations or testing criteria provided by shoring designer.

Consideration should be given to future improvements, including utility excavations and the temporary removal of overburden. If new utility corridors are planned, the temporary excavations should be taken into account when designing the tieback lengths.

10.4 WALL DRAINAGE

10.4.1 At-Grade Site Walls

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

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

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

10.4.2 Below-Grade Walls

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

Sections of horizontal drainage strips should be connected with either the manufacturer’s connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path.

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 the permanent walls, the drainage systems discussed in the “At-Grade Site Walls” section may also be used.

10.5 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 no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

10.6 FOUNDATIONS

The basement retaining walls may be supported on the mat foundation designed in accordance with the recommendations presented in the “Foundations” section of this report.

SECTION 11: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Dreiling Terrones Architecture, Inc. specifically to support the design of the 1128 Douglas Avenue Apartments project in Burlingame, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

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

Dreiling Terrones Architecture, Inc. may have provided Cornerstone with plans, reports and other documents prepared by others. Dreiling Terrones Architecture, 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 12: REFERENCES

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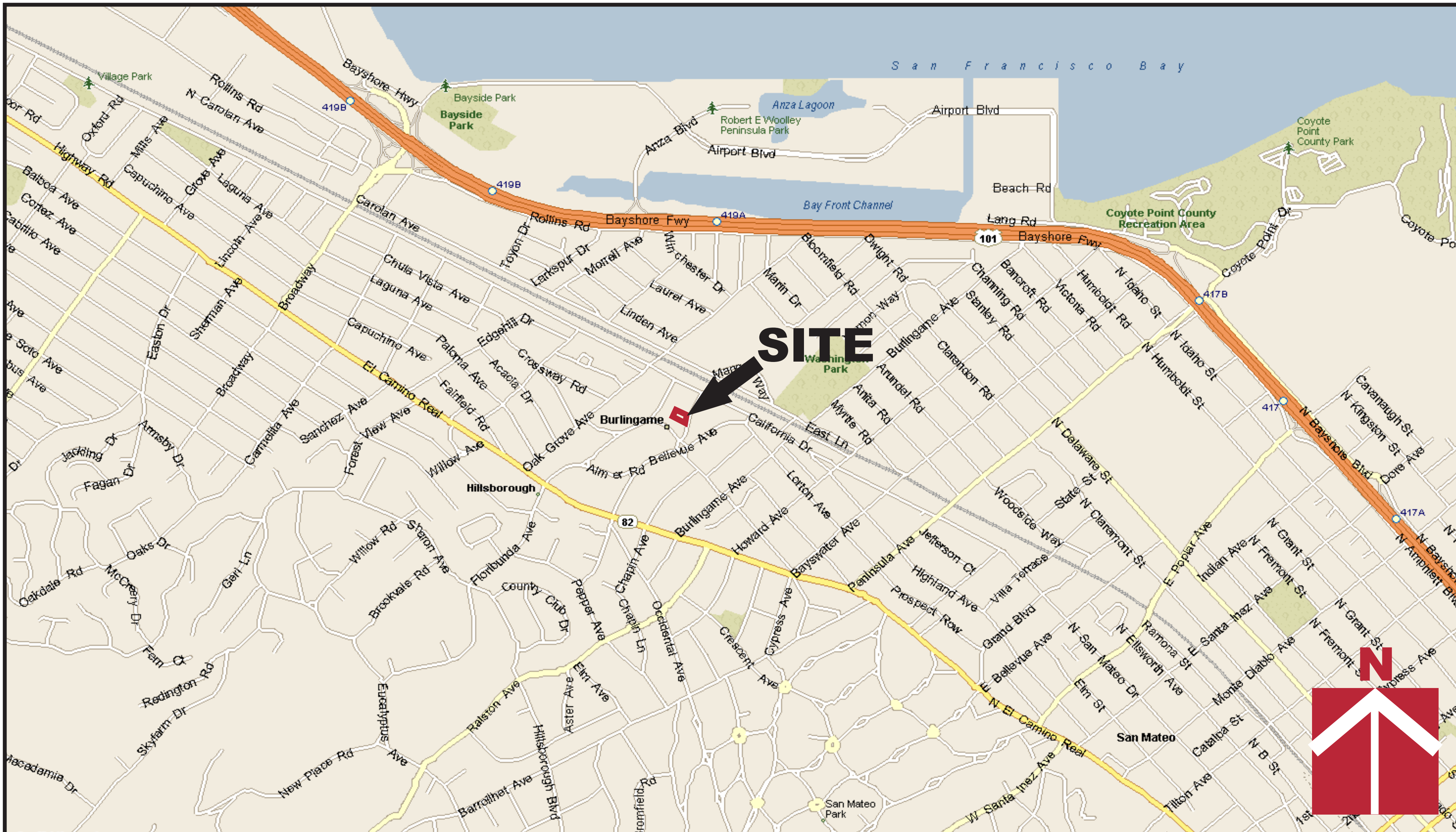


CORNERSTONE EARTH GROUP	**Vicinity Map**	Project Number 345-3-1
	1128 Douglas Avenue Apartements **Burlingame, CA**	Figure Number Figure 1
		Date May 2015 · Drawn By RRN

Legend

Approximate location of exploratory boring (EB)

0 25 50

APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 2/23/2014

Site Plan

1128 Douglas Avenue Apartements
Burlingame, CA

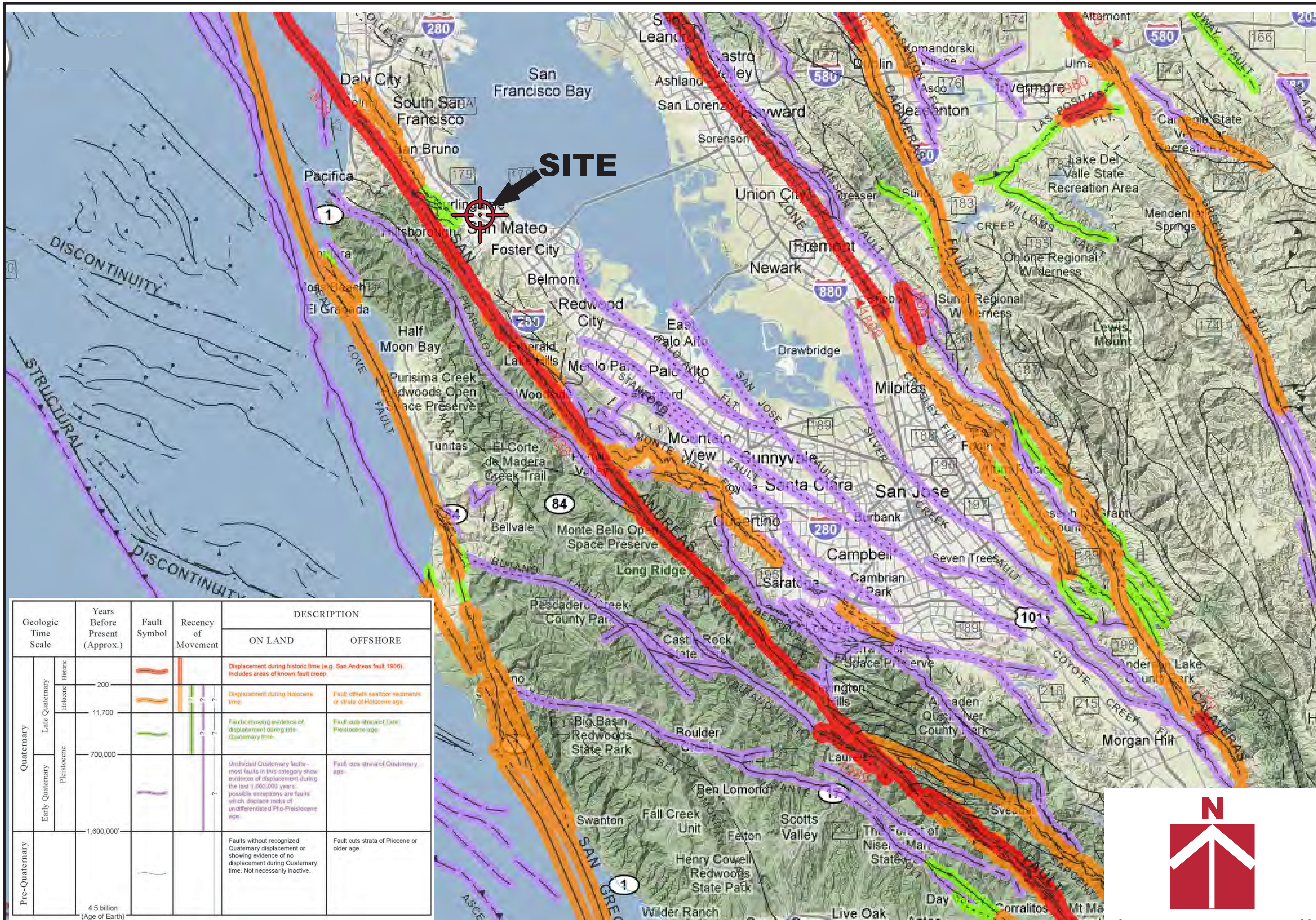
Project Number 745-3-1

Figure Number Figure 2

Date May 2015

Drawn By RRN

CORNERSTONE EARTH GROUP



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

Project Number 745-3-1

Figure Number Figure 3

Drawn By RRN

Date May 2015

Regional Fault Map

1128 Douglas Avenue Apartements
Burlingame, 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 May 6, 2015 to depths of 18½ to 35 feet. The approximate locations of the exploratory borings are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil, are included as part of this appendix.

Boring locations were approximated using existing site boundaries and other site features as references. Boring elevations were not determined. The locations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. 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

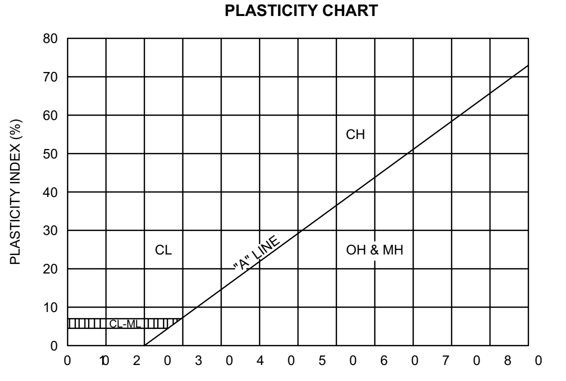
SAMPLER TYPES

- 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

CORNERSTONE EARTH GROUP

PROJECT NAME 1128 Douglas Avenue Apartments
PROJECT NUMBER 745-3-1
PROJECT LOCATION Burlingame, CA

DATE STARTED 5/6/05 **DATE COMPLETED** 5/6/05
GROUND ELEVATION 24 FT +/- **BORING DEPTH** 18.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.580673° **LONGITUDE** -122.349932°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY PKM
AT TIME OF DRILLING Not Encountered
NOTES
AT END OF DRILLING Not Encountered

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

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (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
24.0 / 23.8 / 23.6	0	2 inches asphalt concrete over 2½ inches aggregate base							
		Lean Clay (CL) very stiff, moist, dark brown, fine sand, moderate plasticity		GB-1		26			
				GB-2		23			
	5		38	MC-3B	112	18			○
18.0		**Sandy Lean Clay (CL)** very stiff, moist, brown, fine to coarse sand, low to moderate plasticity							
16.5		**Clayey Sand (SC)** dense, moist, brown with reddish brown mottles, fine to coarse sand, some fine subangular gravel	67	MC-4B	119	11			
	10		50 / 6"	MC-5B	119	15			
			51	MC-6B	115	16			
	15		64	MC-7A	114	15		24	
			54	MC-8B	113	18			○
6.0 / 5.5		**Lean Clay with Sand (CL)** very stiff, moist, brown, fine to medium sand, moderate plasticity							
		Bottom of Boring at 18.5 feet.							
	20								

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME 1128 Douglas Avenue Apartments
PROJECT NUMBER 745-3-1
PROJECT LOCATION Burlingame, CA

DATE STARTED 5/6/05 **DATE COMPLETED** 5/6/05
DRILLING CONTRACTOR Exploration Geoservices, Inc.
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
LOGGED BY PKM
NOTES

GROUND ELEVATION 24 FT +/- **BORING DEPTH** 35 ft.
LATITUDE 37.580409° **LONGITUDE** -122.349719°
GROUND WATER LEVELS:
▽ **AT TIME OF DRILLING** 15 ft.
▼ **AT END OF DRILLING** 15 ft.

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf (○ HAND PENETROMETER, △ TORVANE, ● UNCONFINED COMPRESSION, ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL) 1.0 2.0 3.0 4.0
24.0	0	**Lean Clay (CL)** very stiff, moist, dark brown, fine sand, moderate plasticity Liquid Limit = 46, Plastic Limit = 19	33	MC-1B	103	19	27		○
			37	MC-2B	99	24			○
	5		30	MC-3B	100	23			○
17.0		**Sandy Lean Clay (CL)** stiff, moist, brown with gray mottles, fine to coarse sand, moderate plasticity	24	MC-4B	109	20			○
13.5	10	**Clayey Sand (SC)** medium dense, moist, light reddish brown with gray mottles, fine to medium sand	44	MC-5B	113	18			
11.0		**Clayey Sand (SC)** dense, moist, brown with reddish brown mottles, fine to coarse sand, some fine subangular gravel	50/6"	MC-6B	119	13			
9.0 ▼	15	**Lean Clay with Sand (CL)** very stiff, moist, brown, fine sand, moderate plasticity	33	SPT-7		29			○
7.0		**Clayey Sand (SC)** dense, moist, brown with reddish brown mottles, fine to medium sand, some fine subangular gravel	71	MC-8B	111	19			>4.5 ○
			50/6"	MC-9B	110	17			
	20								
1.0									

Continued Next Page

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 6/12/15 08:40 - P:\DRAFTING\GINT FILES\745-3-1 DOUGLAS AVE BURLINGAME.GPJ

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME 1128 Douglas Avenue Apartments

PROJECT NUMBER 745-3-1

PROJECT LOCATION Burlingame, CA

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

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (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
1.0		**Clayey Sand (SC)** very dense, moist, brown with reddish brown mottles, fine to medium sand, some fine subangular gravel	64	SPT-10		17		16	
	25								
-3.0		**Lean Clay with Sand (CL)** very stiff, moist, brown, fine sand, moderate plasticity	50/5"	MC-11B	106	21			○ (~2.3)
	30								
			67	SPT-12		24			○ (~2.7)
-11.0	35	Bottom of Boring at 35.0 feet.							
	40								
	45								

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

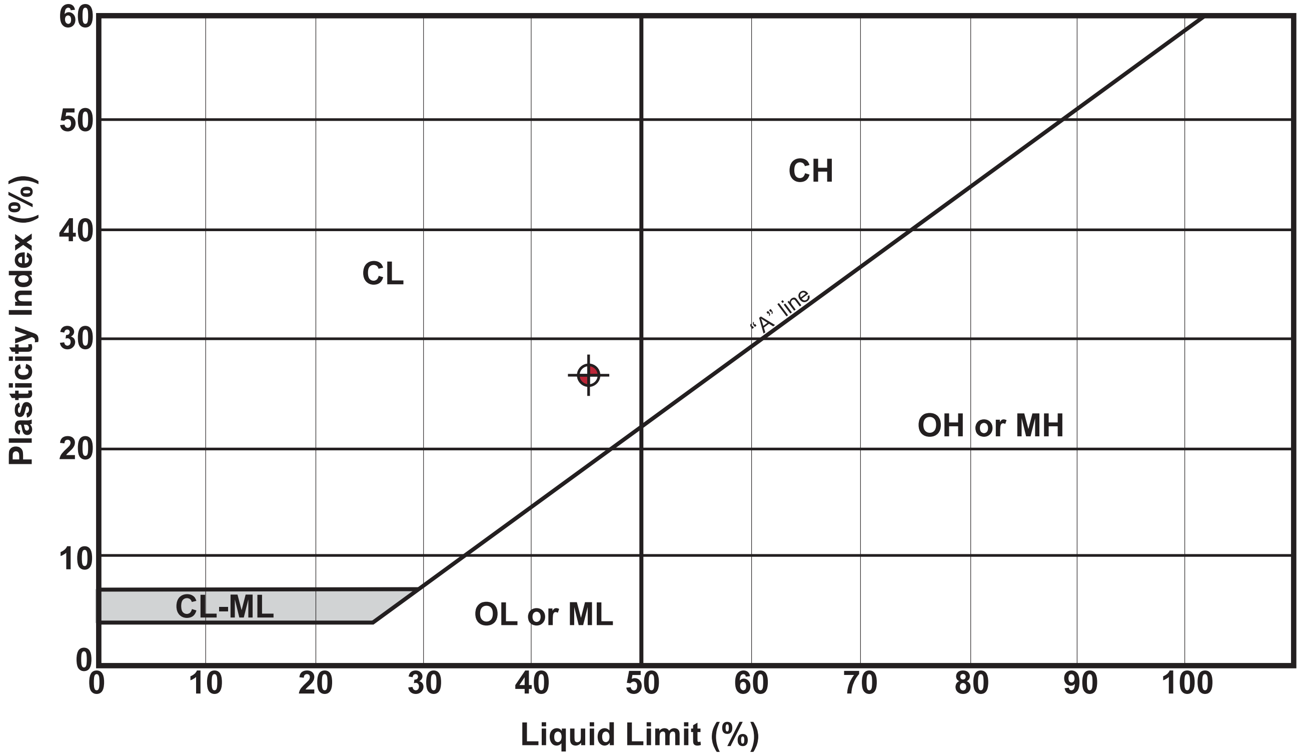
Dry Densities: In place dry density determinations (ASTM D2937) were performed on 15 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

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.

Corrosion: Two samples were tested for pH (ASTM G51), resistivity (ASTM G57), chloride (ASTM D4327), and sulfate (ASTM D4327). 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	2.0	19	46	19	27	—	Lean Clay (CL)

CORNERSTONE EARTH GROUP

Plasticity Index Testing Summary

1128 Douglas Avenue
Burlingame, CA

Project Number 745-3-1

Figure Number Figure B1

Date May 2015

Drawn By FLL

COOPER TESTING LABORATORY

Corrosivity Tests Summary

CTL # 640-822 **Date:** 6/2/2015 **Tested By:** PJ **Checked:** PJ
Client: Cornerstone Earth Group **Project:** Douglas Ave Apts **Proj. No:** 745-3-1
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	4A	9.0	-	-	5,852	6	9	0.0009	7.8	-	-	-	10.5	Dark Yellowish Brown Sandy CLAY w/ Gravel
EB-2	2A	3.5	-	-	1,264	4	28	0.0028	7.4	-	-	-	24.5	Very Dark Grayish Brown CLAY w/ Sand