APPENDIX E

GEOTECHNICAL REPORT

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CORNERSTONE
EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**215 California Drive Office Building**
Location	**215 California Drive** **Burlingame, California**
Client	**Dewey Land Company**
Client Address	**999 Baker Way, Suite 300** **San Mateo, California**
Project Number	**122-5-1**
Date	**March 25, 2015**

DRAFT

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Type of Services	**Geotechnical Investigation**
Project Name	**215 California Drive Office Building**
Location	**215 California Drive Burlingame, California**

SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Dewey Land Company for the 215 California Drive Office Building in Burlingame, California. The location of the site is shown on the Vicinity Map, Figure 1.

We reviewed the subsurface soil and ground water elevation data within a report titled, "Site Characterization Report, 215 California Drive, Burlingame, CA," prepared by Green Environment, Inc. dated December 5, 2013.

1.1 PROJECT DESCRIPTION

The planned development will be four levels of office above grade and up to three levels of below-grade parking garage. We anticipate a steel-frame office building over the below-grade concrete parking garage. The building will extend to the property limits with a footprint of about 13,350 square feet in plan. Appurtenant utility tie-ins and other improvements necessary for site development are also planned.

Grading is anticipated to include cuts up to about 36 feet if a three-level garage is constructed. Structural loading is anticipated to be similar for the anticipated construction type.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposals dated November 18, 2014 and February 9, 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, 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 Cone Penetration Tests (CPTs) advanced on December 18, 2014 and one boring drilled on February 26, 2015 with truck-mounted, hollow-stem auger drilling equipment. The boring was drilled to a depth of 100 feet; the CPTs were advanced to depths of 60 to 75 feet, where practical equipment refusal was met. Seismic shear wave velocity measurements were collected during advancement of CPT-1. Boring EB-1 was advanced adjacent to CPT-2 for direct evaluation of physical samples to correlated soil behavior.

The borings and CPTs 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, grain size analyses, washed sieve analyses, Plasticity Index tests, permeability tests, and triaxial compression tests. Details regarding our laboratory program are included in Appendix B.

1.5 CORROSION EVALUATION

Three samples from our borings from depths from 2 to 15 feet were tested for saturated resistivity, pH, and soluble sulfates and chlorides. JDH Corrosion Consultants prepared a brief corrosion evaluation based on the laboratory data, which is attached to this report in Appendix C. In general, the on-site soils can be characterized as moderately corrosive to buried metal, and non-corrosive to buried concrete.

1.6 ENVIRONMENTAL SERVICES

We understand that environmental services for the project are being provided by Green Environment, Inc. If environmental concerns are present, they 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)	2.9	4.7
San Gregorio	9.9	15.9
Monte Vista-Shannon	10.9	17.6
Hayward (Total Length)	15.5	24.9

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

SECTION 3: SITE CONDITIONS

3.1 SURFACE DESCRIPTION

The site is currently occupied by a one-story, subdivided retail/commercial building with a small parking lot at the rear. The site is bounded by contiguous commercial buildings to the northwest and southeast, Hatch Lane to the southwest, and Highland Avenue and California Drive to the northeast.

Surface pavements encountered in Boring EB-1 consisted of 1½ inches of asphalt concrete over 4½ inches of aggregate base.

3.2 SUBSURFACE CONDITIONS

Below the surface pavements, our explorations generally encountered a thin surficial layer of very stiff, low plasticity sandy clay to a depth of 5 feet overlying medium dense to dense clayey sands to a depth of about 15 feet. Interbedded very stiff to hard lean clays and dense to very dense clayey sands exist to the maximum depth explored of 100 feet.

3.2.1 Plasticity/Expansion Potential

We performed two Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of surficial soils, and the plasticity of the fines in potentially liquefiable layers. The results of the PI tests indicated a PI of 7 for the surficial clay, indicating low expansion potential to wetting and drying cycles. The PI test on a sample of clayey sand at 9½ feet resulted in a PI of 16, indicating plastic fines.

3.2.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from 2 to 4 percent over the estimated laboratory optimum moisture.

3.3 GROUND WATER

Ground water was encountered in our Boring EB-1 at a depth of about 16½ feet below current grades. All measurements were taken at the time of drilling and may not represent the stabilized levels that can be higher than the initial levels encountered. Previous monitoring well data indicates static ground water levels on the order of 11 to 12 feet below grade in August 2013.

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

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}*PGA, as allowed in the 2013 edition of the California Building Code. For our liquefaction analysis we used a PGA of 0.797g.

4.3 LIQUEFACTION POTENTIAL

The site is not currently mapped by the State of California, and is within a zone mapped as having a low liquefaction potential by the Association of Bay Area Governments (ABAG, 2007). 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, we primarily encountered stiff cohesive and medium dense to dense granular soils with clayey, plastic fines below the design ground water depth of 10 feet. Following the liquefaction analysis framework in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008), incorporating updates in *CPT and SPT Based Liquefaction Triggering Procedures* (Boulanger and Idriss, 2014), and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008) for quantitative analysis, these layers were analyzed for liquefaction triggering and potential post-liquefaction settlement. These methods compare the ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil’s estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.3 are considered to be potentially liquefiable and capable of post-liquefaction re-consolidation (i.e. settlement).

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

The soil’s CRR is estimated from the in-situ measurements from CPTs and laboratory testing on samples retrieved from our borings. SPT “N” values obtained from hollow-stem auger borings were not used in our analyses, as the “N” values obtained are less reliable in sands below ground water. The tip pressures are corrected for effective overburden stresses, taking into consideration both the ground water level at the time of exploration and the design ground water level, and stress reduction versus depth factors. The CPT method utilizes the soil behavior type index (I_C) to estimate the plasticity of the layers. Selected soil samples collected from Boring EB-1 adjacent to CPT-2 were tested to evaluate grain size, plasticity, as well as visually observed for confirmation of CPT soil behavior types.

Based on the plasticity of the fines in the sands, our screening analyses resulted in less than ¼-inch of total liquefaction; therefore, a low potential for liquefaction affecting the site, and is in general agreement with local mapping for the site by ABAG.

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 low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. As the soils encountered at the site were predominantly stiff to 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.

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
- Proximity to existing structures
- Construction dewatering induced settlements
- Soil Corrosion Potential

5.1.1 Shallow Ground Water

Shallow ground water was measured at a depth of about 16½ feet below the existing ground surface. Based on previous monitoring well measurements, ground water was present at depths of about 11 to 12 feet below grade in August 2013. Our design high ground water depth for the project analyses is 10 feet. Our experience with similar sites in the vicinity indicates that shallow ground water could significantly impact building excavation and other underground construction. These impacts typically consist of potentially wet and unstable subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering may be possible as part of the shoring of the excavations, provided the settlements discussed below are acceptable. Detailed recommendations addressing this concern are presented in the "Earthwork" section of this report.

Shallow ground water may also present several design challenges for the permanent structure. Because the planned lower levels of the garage 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 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 Proximity to Existing Structures

The planned garage excavation will go to nearly the property lines (less the space required for temporary shoring). The temporary shoring will need to accommodate loading from the adjacent buildings and provide for limited movement of these structures to reduce the potential for distress. Restrained shoring will be required to limit deflections at the top of the excavation that could result in settlement of the adjacent foundations. Underpinning of the adjacent foundations, either from a temporary basis, where the new structure garage walls will be designed to support those adjacent foundation loads, or on a permanent basis, may be considered.

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

5.1.3 Construction Dewatering Induced Settlements

We evaluated the potential settlement of the surrounding ground for a two-level and a three-level below-grade excavation with dewatering to at least 5 feet below the bottom of the mass excavation. Our analyses assumed a dewatering depth of 28 and 40 feet, resulting in just under 1 inch and 1¼ inches, respectively. If either of those settlements are considered tolerable to the adjacent structures as well as City improvements in the streets, dewatering should be feasible. We have included permeability results in Appendix B for three tests performed on clayey sands within the upper 40 feet. These permeability values are relatively low, indicating that the soils will release water slowly. If settlement due to dewatering is not desired, the shoring can be designed as undrained cut off walls, with secant soil-cement columns or similar.

5.1.4 Soil Corrosion Potential

A preliminary soil corrosion screening was performed by JDH Corrosion Consultants based on the results of analytical tests on samples of the near-surface soil. In general, the JDH report concludes that the corrosion potential for buried concrete does not warrant the use of sulfate resistant concrete. The corrosion potential for buried metallic structures, such as metal pipes, is considered moderately corrosive. JDH recommends that special requirements for corrosion control be made to protect pressurized metal pipes. A more detailed discussion of the site corrosion evaluation is presented in Appendix C.

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 development area. Demolition of existing improvements is discussed in detail below.

6.1.2 Demolition of Existing Slabs, Foundations and Pavements

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

6.1.3 Abandonment of Existing Utilities

All utilities should be completely removed from within planned building area and plugged at the back of shoring. 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.

6.2 TEMPORARY CUT AND FILL SLOPES

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

6.3 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.

Most likely temporary shoring will support the planned cuts up to 26 to 38 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 2: Suggested Temporary Shoring Design Parameters

Design Parameter	Design Value
Minimum Lateral Wall Surcharge (upper 5 feet)	120 psf
Cantilever Wall – Triangular Earth Pressure	**35 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 3,000 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, and additional 40 pcf should be added for hydrostatic pressures.

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

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

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

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

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

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

6.4.2 Construction Dewatering and Potential Ground Settlements

Ground water levels are expected to be significantly above the planned excavation bottom for both the two-level and three-level excavation alternatives; therefore, either temporary dewatering will be necessary during construction or the shoring will need to be designed as undrained cut off walls.

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. We have performed three permeability tests on samples of the clayey sand zones within the upper 40 feet to aid in the design.

The dewatering design should maintain ground water at least 5 feet below the bottom of the mass excavation. 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. As a draw down depth of 28 or 40 feet is planned, we evaluated the potential settlement due to draw down. We estimate that there could be up to about 1 inch of settlement at the ground surface for a draw down to 28 feet and 1¼ inches for a draw down to 40 feet. If this settlement is deemed excessive, we recommend alternative shoring methods such as tied back slurry walls or soil mixed curtain walls be considered.

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

As the structure will be supported on a mat foundation, the excavation should be cut neat without allowing construction equipment to access the exposed subgrade. If construction equipment will need to operate near subgrade, the recommendations in the “Below-Grade Excavation Stabilization” section below should be followed.

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.

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 6 to 9 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 to 10,000 square feet and/or site winterization is desired, chemical treatment with quicklime (CaO), kiln-dust, or cement may be more cost-effective than removal and replacement. Recommended chemical treatment depths will typically range from 12 to 18 inches depending on the magnitude of the instability.

6.6.4 Below-Grade Excavation Stabilization

As the planned basement excavation will extend significantly below the current ground water level, we recommend that the contractor plan to excavate an additional 12 to 18 inches below subgrade, place a layer of stabilization fabric (Mirafi 500X, or equivalent) at the bottom, and backfill with clean, crushed rock. The crushed rock should be consolidated in place with light vibratory equipment. Rubber-tire equipment should not be allowed to operate on the exposed subgrade; the crushed rock should be stockpiled and pushed out over the stabilization fabric.

6.7 MATERIAL FOR FILL

6.7.1 Re-Use of On-site Soils

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

6.7.2 Potential Import Sources

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

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

6.8 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 consolidated in place with vibratory equipment. Each lift of fill and all subgrade should be firm and unyielding under construction equipment loading in addition to meeting the compaction requirements to be approved. The contractor (with input from a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions. General recommendations for soil stabilization are provided in the “Subgrade Stabilization Measures” section of this report. Where the soil’s PI is 20 or greater, the expansive soil criteria should be used.

Table 3: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Soils	95	>1
Basement Wall Backfill	Without Surface Improvements	90	>1
Basement Wall Backfill	With Surface Improvements	95[4]	>1
Trench Backfill	On-Site Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum
Flatwork Subgrade	On-Site Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95	NA

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

6.9 TRENCH BACKFILL

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

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

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

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

6.10 SITE DRAINAGE

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

6.11 LOW-IMPACT DEVELOPMENT (LID) IMPROVEMENTS

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

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

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

- Locally, seasonal high ground water is expected at a depth of 10 feet.

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 clay.

- Bioswales constructed within 3 feet of existing 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.

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

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

7.2 SEISMIC DESIGN CRITERIA

We understand that the project structural design will 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 borings and review of local geology, the site is underlain by deep alluvial soils with typical SPT “N” values between 15 and 50 blows per foot. Therefore, we have classified the site as Soil Classification D. The mapped spectral acceleration parameters S_S and S_1 were calculated using the USGS computer program *Earthquake Ground Motion Parameters*, Version 5.1.0, revision date February 10, 2011, 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 4: CBC Site Categorization and Site Coefficients

Classification/Coefficient	**Design Value**
Site Class	D
Site Latitude	37.57889°
Site Longitude	-122.34463°
0.2-second Period Mapped Spectral Acceleration[1], S_S	2.039g
1-second Period Mapped Spectral Acceleration[1], S_1	0.962g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv	1.5
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	2.039g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.443g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.359g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.962g

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

7.3 SHALLOW FOUNDATIONS

7.3.4 Reinforced Concrete Mat Foundations

Due to the magnitude of the hydrostatic uplift for both the two-level and three-level below-grade garage alternatives, the structures 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.

To reduce potential differential movement, all mats should be designed for a maximum average areal bearing pressure of 1,500 psf for dead plus live loads; at column or wall loading, the maximum localized bearing pressure should be limited to 4,500 psf. When evaluating wind and seismic conditions, allowable bearing pressures may be increased by one-third. These pressures are net values; the weight of the mat may be neglected for the portion of the mat extending below grade. Top and bottom mats of reinforcing steel should be included as required to help span irregularities and differential settlement.

7.3.5 Mat Foundation Settlement

We estimate differential static settlements on the order of ½- to 1 inch for recompression of the subgrade soils. If modulus of soil subgrade reaction is desired for a structural analysis such as SAFE, we should be provided with the initial output of contact pressures. An initial modulus of 10 pci should be used.

7.3.6 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 mat edges. An ultimate frictional resistance of 0.45 applied to the mat dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above.

7.4 HYDROSTATIC UPLIFT AND WATERPROOFING

As previously discussed, we recommend a design ground water level of 10 feet below grade. This will result in significant uplift pressures at the bottom of the mat foundation. Likely the design will require uplift anchors for permanent resistance of the pressures. A buoyancy evaluation should be performed by the structural engineer to evaluate the number of anchors that would be needed. Uplift anchor design parameters are provided in the section below. The bottom of the mat foundation and the full height of the garage walls should be fully waterproofed, with the design performed by a waterproofing consultant.

7.5 UPLIFT GROUND ANCHORS

Ground anchors may be used to resist seismic uplift loads as well as the long term hydrostatic uplift forces. For both two-level and three-level excavations, we estimate ultimate uplift capacities for 8-inch-diameter, high pressure grouted ground anchors of 250 kips for a 35-foot bonded length and 280 kips for a 40-foot bonded length. The structural engineer should apply an appropriate factor of safety to the ultimate values. All anchors should be load tested to confirm design capacity in accordance with FHWA recommendations. Ground anchors should be spaced at a minimum of 3 feet on center. Construction tolerances for vertical alignment should be specified such that there will not be overlap at the anchor tips.

SECTION 8: VEHICULAR AND PEDESTRIAN PAVEMENTS

8.1 ASPHALT CONCRETE

Patching of existing asphalt concrete pavements in the public right-of-way should match in kind the existing structural section, or conform to a minimum section provided by the City.

8.2 PORTLAND CEMENT CONCRETE

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

8.3 EXTERIOR FLATWORK

Exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading should be at least 4 inches thick and supported on at least 4 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the “Earthwork” recommendations of this report. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

SECTION 9: RETAINING WALLS

9.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall. If a drainage system is constructed behind the wall above the design ground water level of 10 feet that will prevent the build-up of hydrostatic pressures as discussed in the section below, we recommend that the walls with level backfill be designed for the following pressures:

Table 5: Recommended Drained 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. Where 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. Waterproofing of the walls tied into the waterproofing of the mat foundation should be designed by a waterproofing consultant.

9.2 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 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, 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.

9.3 WALL DRAINAGE

If drainage above the design ground water level of 10 feet (or to some shallower depth) will be included to reduce the lateral earth pressures, this system should consist of Miradrain, AmerDrain or other equivalent drainage matting 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 system by a horizontal drainage strip and closed or through-wall system such as the TotalDrain system from AmerDrain. Water will be diverted inside the building at several locations, tied to solid piping system that carries the water to a sump location.

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

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

9.4 BACKFILL

Where surface improvements will be located over any 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.

9.5 FOUNDATIONS

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

SECTION 10: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Dewey Land Company specifically to support the design of the 215 California Drive Office Building 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.

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

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

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

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

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

SECTION 11: REFERENCES

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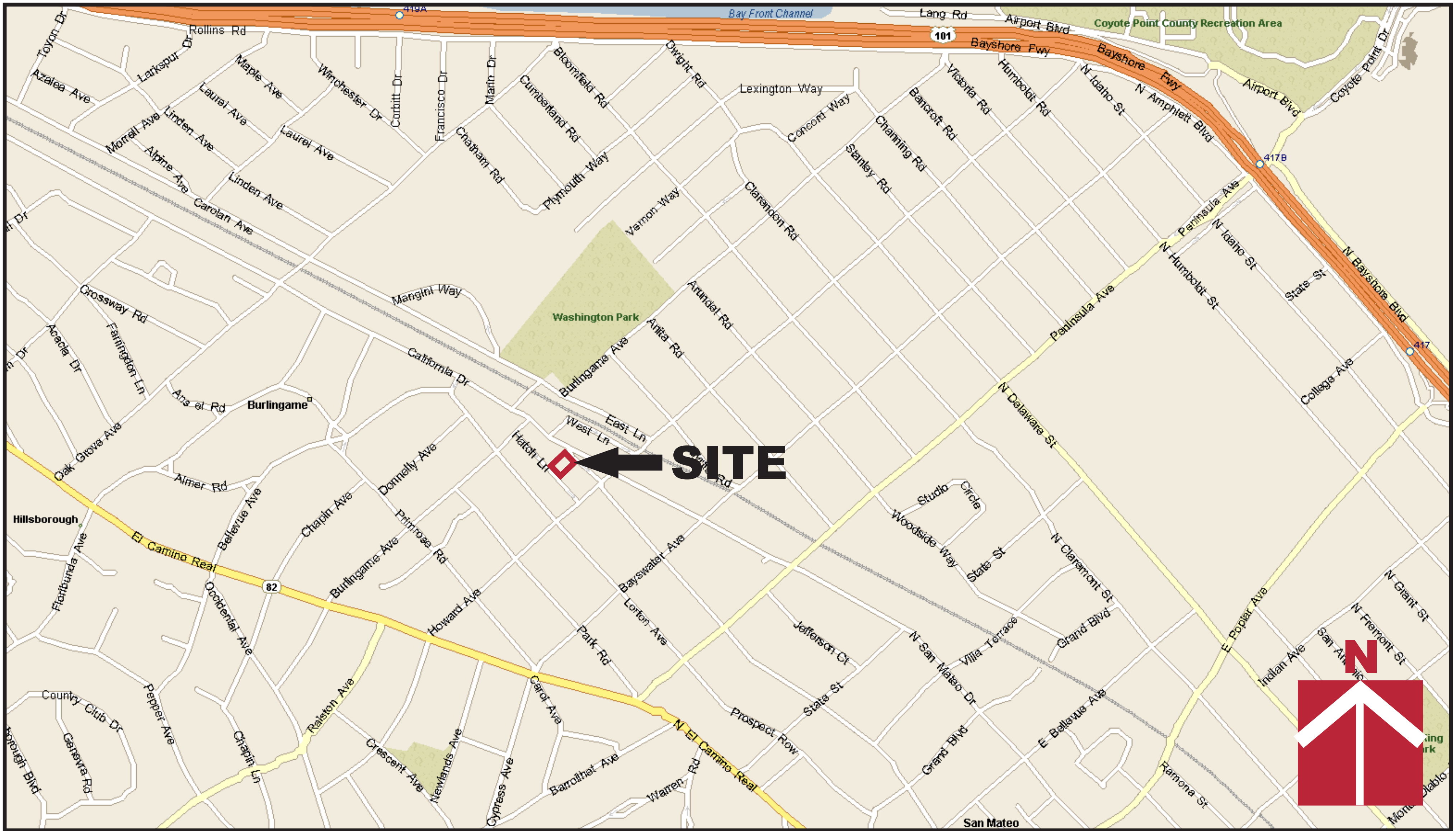


CORNERSTONE EARTH GROUP

Vicinity Map

215 California Drive
Burlingame, CA

Project Number
122-5-1

Figure Number
Figure 1

Date
March 2015

Drawn By
RRN

Base by Google Earth, dated 2/23/2014

Project Number 122-5-1

Figure Number Figure 2

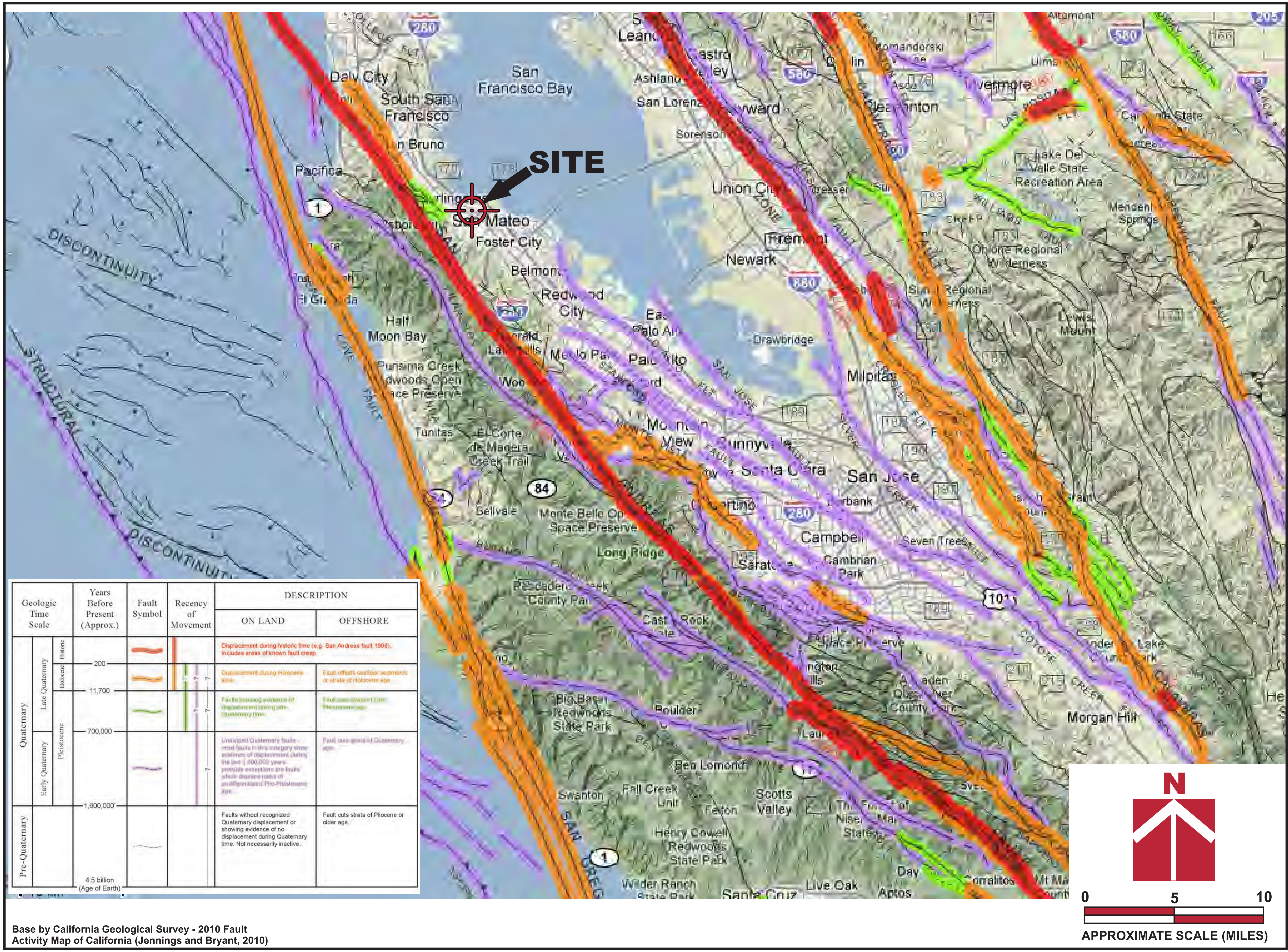
Drawn By RRN

Date March 2015

Site Plan

215 California Drive
Burlingame, CA

CORNERSTONE
EARTH GROUP



<table>
<tr><th rowspan="2">Geologic Time Scale</th><th></th><th></th><th rowspan="2">Years Before Present (Approx.)</th><th rowspan="2">Fault Symbol</th><th rowspan="2">Recency of Movement</th><th colspan="2">DESCRIPTION</th></tr>
<tr><th></th><th></th><th>ON LAND</th><th>OFFSHORE</th></tr>
<tr><td>Quaternary</td><td>Late Quaternary</td><td>Historic</td><td>200</td><td>🟥</td><td></td><td>Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.</td><td></td></tr>
<tr><td></td><td></td><td>Holocene</td><td>11,700</td><td>🟧</td><td>?</td><td>Displacement during Holocene time.</td><td>Fault offsets seafloor sediments or strata of Holocene age.</td></tr>
<tr><td></td><td></td><td>Pleistocene</td><td>700,000</td><td>🟩</td><td>?</td><td>Faults showing evidence of displacement during late Quaternary time.</td><td>Fault cuts strata of Late Pleistocene age.</td></tr>
<tr><td></td><td>Early Quaternary</td><td></td><td>1,600,000*</td><td>🟪</td><td>?</td><td>Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.</td><td>Fault cuts strata of Quaternary age.</td></tr>
<tr><td>Pre-Quaternary</td><td></td><td></td><td>4.5 billion (Age of Earth)</td><td>—</td><td></td><td>Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.</td><td>Fault cuts strata of Pliocene or older age.</td></tr>
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Base by California Geological Survey - 2010 Fault Activity Map of California (Jennings and Bryant, 2010)

Project Number 122-5-1

Figure Number Figure 3

Date March 2015

Drawn By RRN

Regional Fault Map

215 California Drive
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 and 20-ton truck-mounted Cone Penetration Test equipment. One 8-inch-diameter exploratory boring was drilled on February 26, 2015 to a depth of 100 feet. Two CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on December 18, 2014, to depths of 60 to 75 feet, where each met with practical equipment refusal. The approximate locations of exploratory borings and CPTs are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

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

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

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

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

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

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

OTHER MATERIAL SYMBOLS

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

SAMPLER TYPES

☒ SPT

⧗ Modified California (2.5" I.D.)

▯ Rock Core

■ Shelby Tube

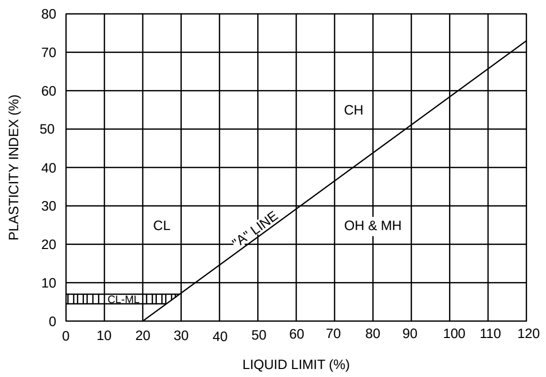
○ No Recovery

✋ Grab Sample

ADDITIONAL TESTS

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

PLASTICITY CHART



PENETRATION RESISTANCE
(RECORDED AS BLOWS / FOOT)

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

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

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

CORNERSTONE
EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number
A-1

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME 215 California Drive
PROJECT NUMBER 122-5-1
PROJECT LOCATION Burlingame, CA
DATE STARTED 2/26/15 **DATE COMPLETED** 2/26/15
GROUND ELEVATION **BORING DEPTH** 100 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY PKM
▽ **AT TIME OF DRILLING** 16.7 ft.
NOTES
▼ **AT END OF DRILLING** 16.7 ft.

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

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0	1½ inches ashalt concrete over 4½ inches aggregate base							
		Sandy Silty Clay (CL-ML) stiff to very stiff, moist, dark brown to brown, fine to medium sand, low plasticity Liquid Limit = 20, Plastic Limit = 13	25	MC-1B	113	15	7		○
			23	MC-2B	113	15			○
	5	**Clayey Sand with Gravel (SC)** medium dense to dense, moist, brown with reddish brown mottles, fine to coarse sand, fine subangular to subrounded gravel	50	MC-3B	120	14			
			61	4A MC 4B	101 111	14 12	 16	 24	
	10	Liquid Limit = 32, Plastic Limit = 16 Perm 1 x 10^{-5}cm/s							
		color change to gray	45	MC-5B	119	13			
	15	Perm 1 x 10^{-6}cm/s	64	6A MC 6B	112 113	18 15			>4.5 ○
▼		**Sandy Lean Clay (CL)** hard, moist, brown, fine to medium sand, low plasticity							
		Lean Clay with Sand (CL) very stiff to hard, moist, brown with reddish brown mottles, fine to medium sand, moderate plasticity	48	MC-7B	104	22			○
	20								
			60	MC					>4.5 ○
	25	**Clayey Sand with Gravel (SC)** dense, moist, brown with reddish brown mottles, fine to coarse sand, fine subangular to subrounded gravel	80	9A MC 9B	109 119	21 17		29	
			63	MC-10B	119	15			
	30								

Continued Next Page

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME 215 California Drive

PROJECT NUMBER 122-5-1

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.

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
	35	**Clayey Sand with Gravel (SC)** dense, moist, brown with reddish brown mottles, fine to coarse sand, fine subangular to subrounded gravel Perm 1 x 10^{-7}cm/s	62	MC-11B	112	19		
	40	**Lean Clay with Sand (CL)** hard, moist, brown, fine to medium sand, moderate plasticity	45	MC-12A	98	27		
			50 6"	MC-13	110	20		
	45		60	SPT				
	50	very stiff	72	MC-15B	107	21		
	55		43	MC-16A	93	31		
		Silty Sand (SM) medium dense, moist, brown, fine to medium sand						
	60	**Lean Clay with Sand (CL)** stiff, moist, brown, fine to medium sand, moderate plasticity	52	MC-17A	109	19		
		Clayey Sand with Gravel (SC) dense to very dense, moist, brown with reddish brown mottles, fine to coarse sand, fine subangular to subrounded gravel	50 6"	18A MC	111	17		23
				18B	111	19		
	65	**Lean Clay with Sand (CL)** very stiff to hard, moist, brown, fine to medium sand, moderate plasticity						

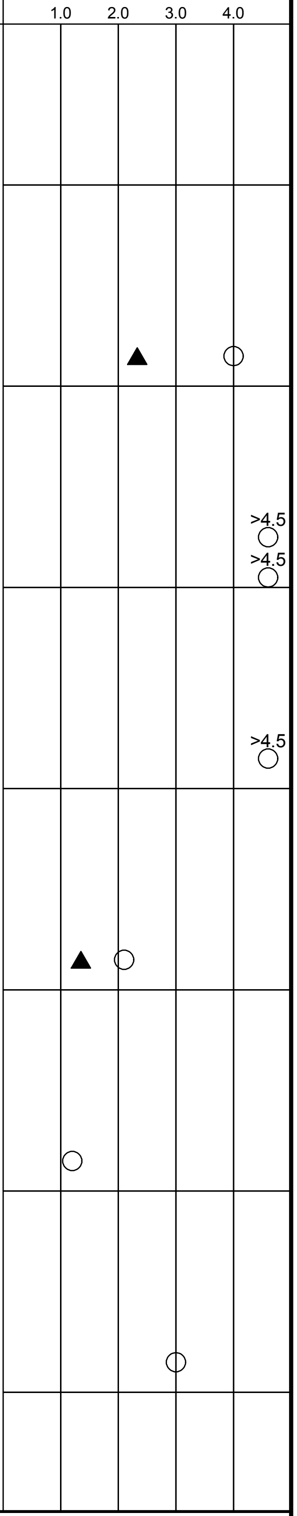
UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL



Continued Next Page

BORING NUMBER EB-1

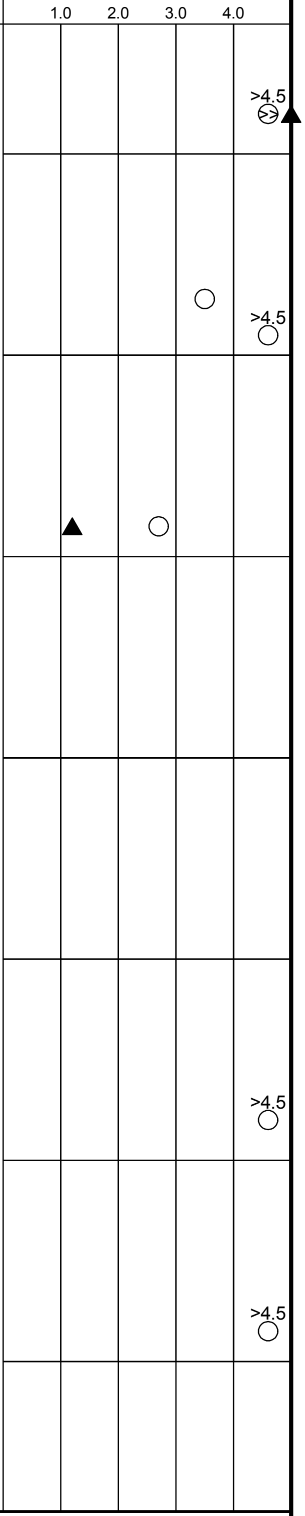
CORNERSTONE
EARTH GROUP

PROJECT NAME 215 California Drive
PROJECT NUMBER 122-5-1
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.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	70	▨	**Lean Clay with Sand (CL)** very stiff to hard, moist, brown, fine to medium sand, moderate plasticity	50 5"	MC-19B	108	22			>4.5
		▨	**Sandy Lean Clay (CL)** very stiff, moist, brown, fine to medium sand, low plasticity							
	75	▨	**Lean Clay with Sand (CL)** hard, moist, brown, fine to medium sand, moderate plasticity	50 6"	MC-20C	103	23			>4.5
	80	▨	**Sandy Lean Clay (CL)** very stiff, moist, brown, fine to medium sand, low plasticity	60	MC-21B	108	22			
	85	▨	**Clayey Sand with Gravel (SC)** very dense, moist, brown with reddish brown mottles, fine to coarse sand, fine subangular to subrounded gravel	50 6"	MC-22B	105	20			
	90			50 6"	MC					
				50 6"	SPT					
	95	▨	**Sandy Lean Clay (CL)** hard, moist, gray and brown with reddish brown mottles, fine to coarse sand, low to moderate plasticity	50 6"	SPT					>4.5
	100	▨	**Lean Clay with Sand (CL)** hard, moist, brown with gary mottles, fine to medium sand, moderate plasticity	76	SPT					>4.5
			Bottom of Boring at 100.0 feet.							

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



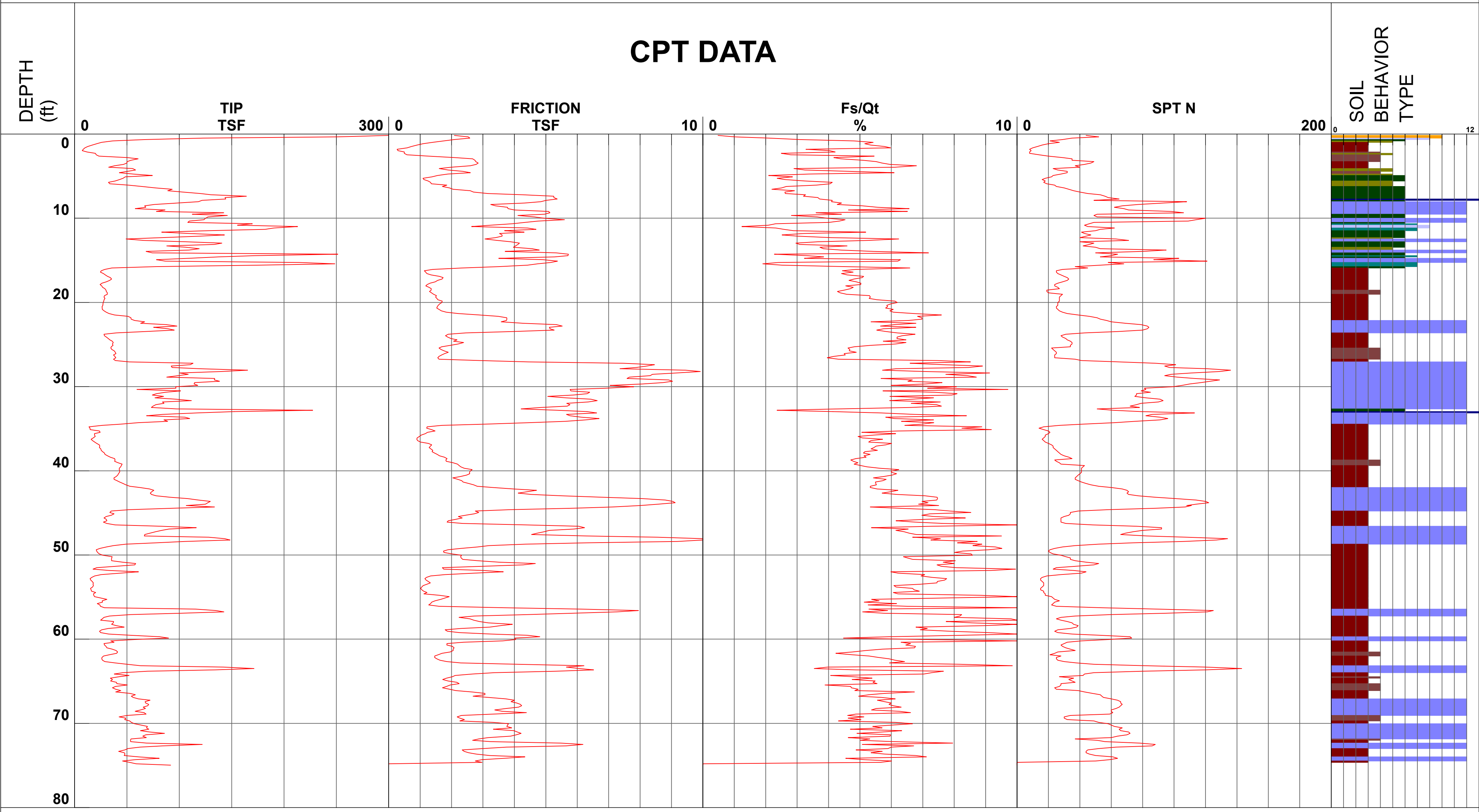
Cornerstone Earth Group

Project 215 California Drive Office
Job Number 122-5-1
Hole Number CPT-01
EST GW Depth During Test 5.00 ft

Operator CB-BH
Cone Number DDG1298
Date and Time 12/18/2014 10:18:12 AM

Filename SDF(014).cpt
GPS
Maximum Depth 74.97 ft

Net Area Ratio .8



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

Cone Size 10cm squared

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

Cornerstone Earth Group

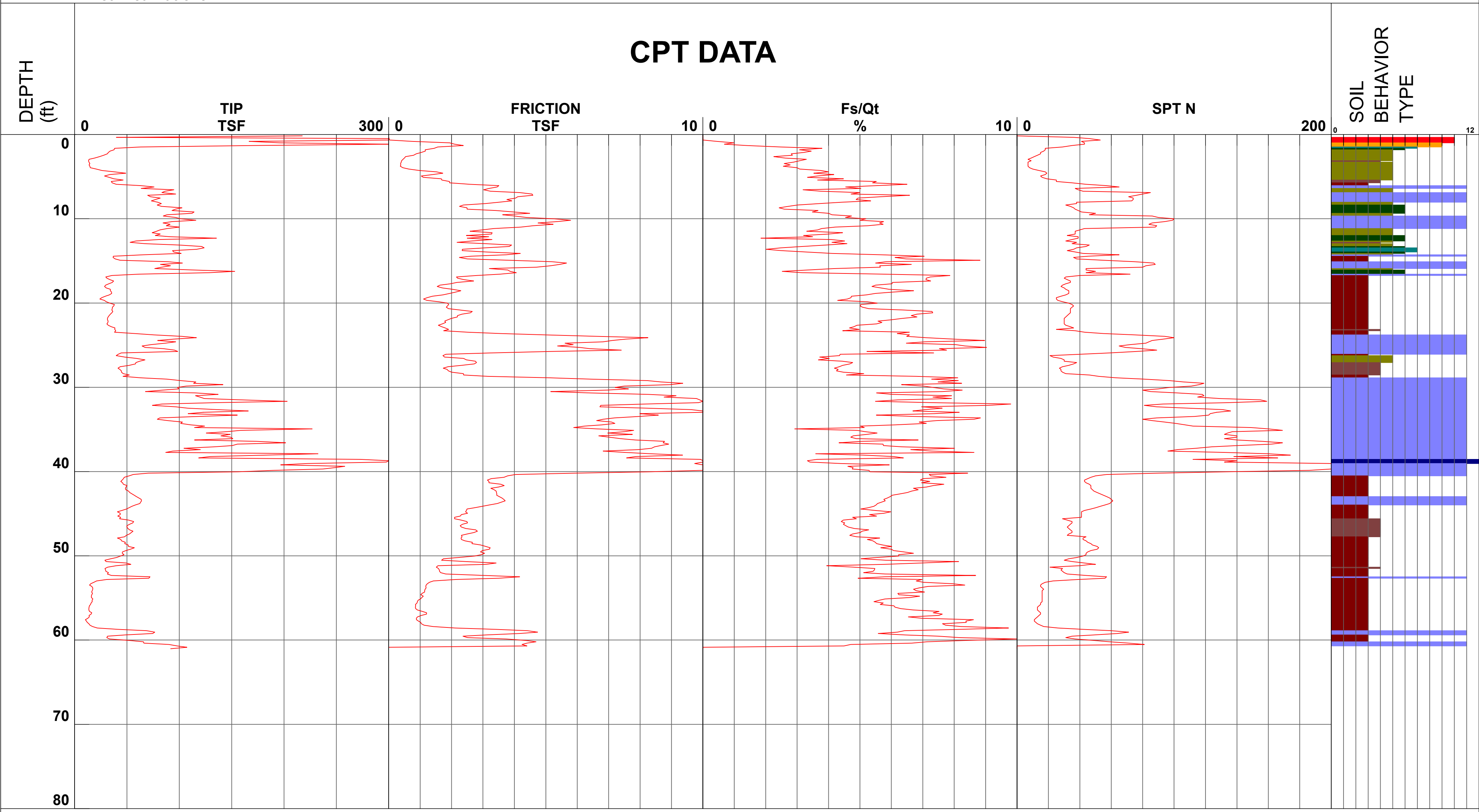
Project 215 California Drive Office
Job Number 122-5-1
Hole Number CPT-02
EST GW Depth During Test 5.00 ft

Operator CB-BH
Cone Number DDG1298
Date and Time 12/18/2014 8:59:48 AM

Filename SDF(013).cpt
GPS
Maximum Depth 61.02 ft

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay

■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt

■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand

■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

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

APPENDIX B: LABORATORY TEST PROGRAM

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

Moisture Content: The natural water content was determined (ASTM D2216) on 24 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 24 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.

Grain Size Analyses: The particle size distribution (ASTM D422) was determined on one sample of the subsurface soils to aid in the classification of these soils. Results of these tests are attached in this appendix.

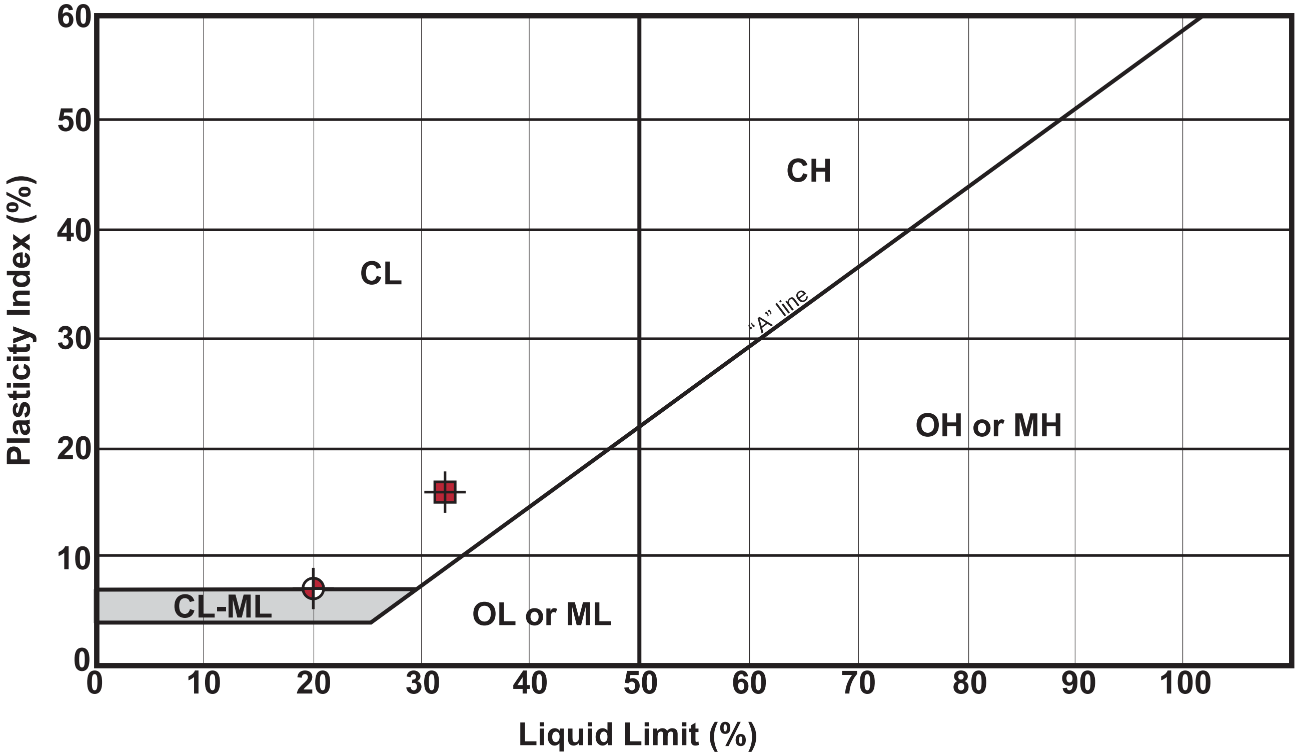
Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on three samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

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

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

Permeability: Three falling head permeability tests (ASTM D5084) were performed on samples of the subsurface soil to measure the hydraulic conductivity of those materials. Results of the tests are included 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-1	2.0	15	20	13	7	—	Sandy Silty Clay (CL-ML)
▣	EB-1	9.5	12	32	16	16	24	Clayey Sand (SC) (CL fines)

CORNERSTONE
EARTH GROUP

Plasticity Index Testing Summary

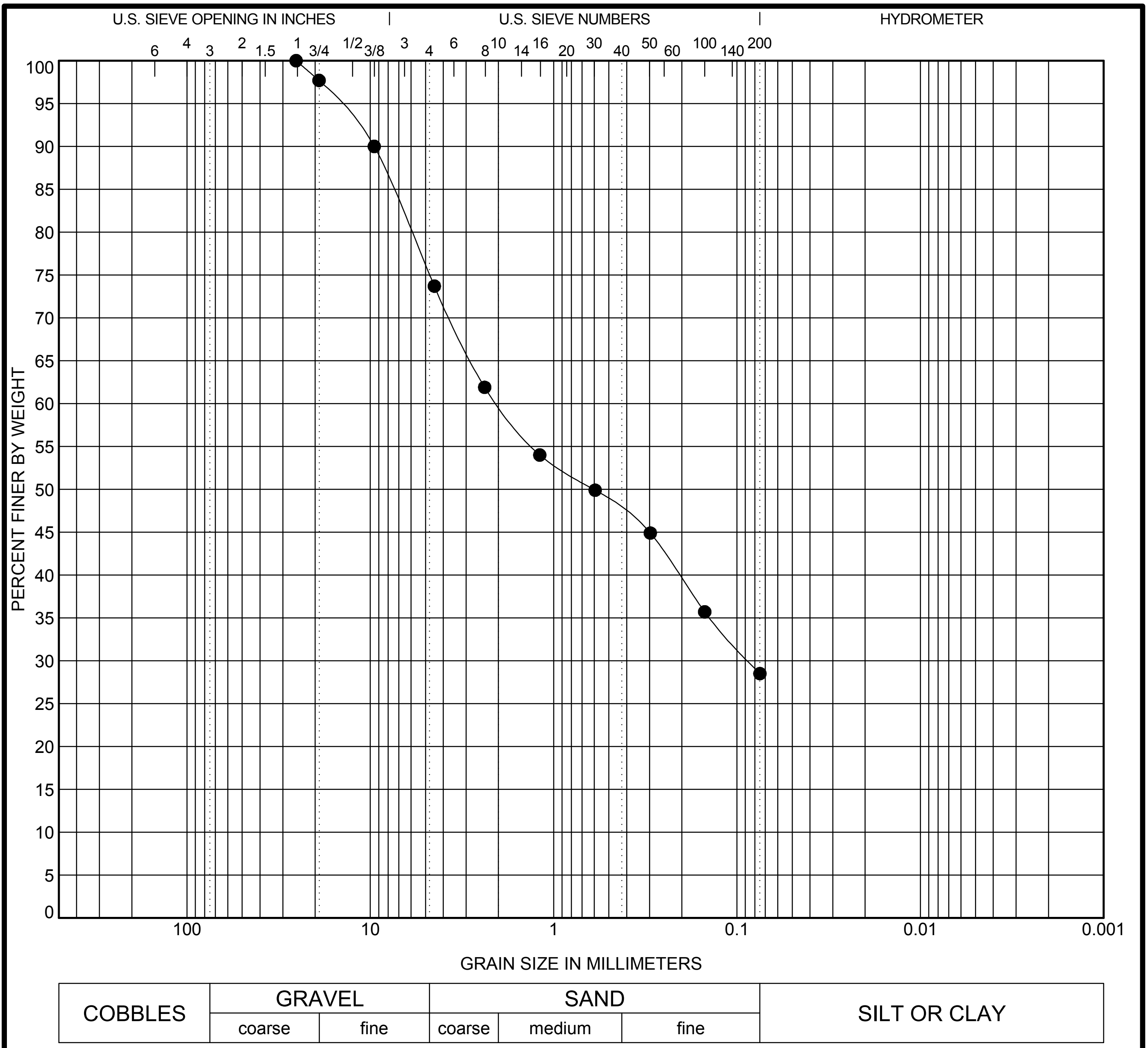
215 California Drive
Burlingame, CA

Project Number
122-5-1

Figure Number
Figure B1

Date
March 2015

Drawn By
FLL



Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● EB-1	**24.3**	**Clayey Sand with Gravel (SC)**					

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● EB-1	**24.3**	**25.4**	**2.015**	**0.087**		**25.0**	**46.5**	**28.5**	

CORNERSTONE
EARTH GROUP

GRAIN SIZE DISTRIBUTION

Project: 215 California Drive

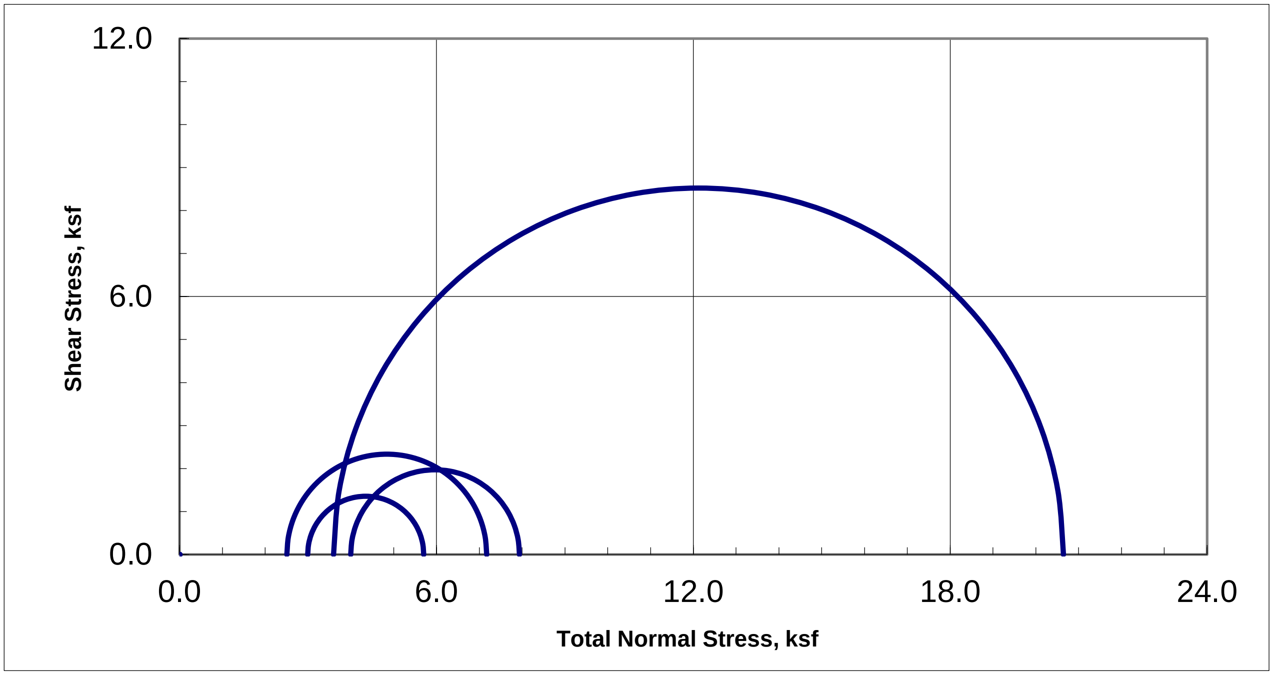
Location: Burlingame, CA

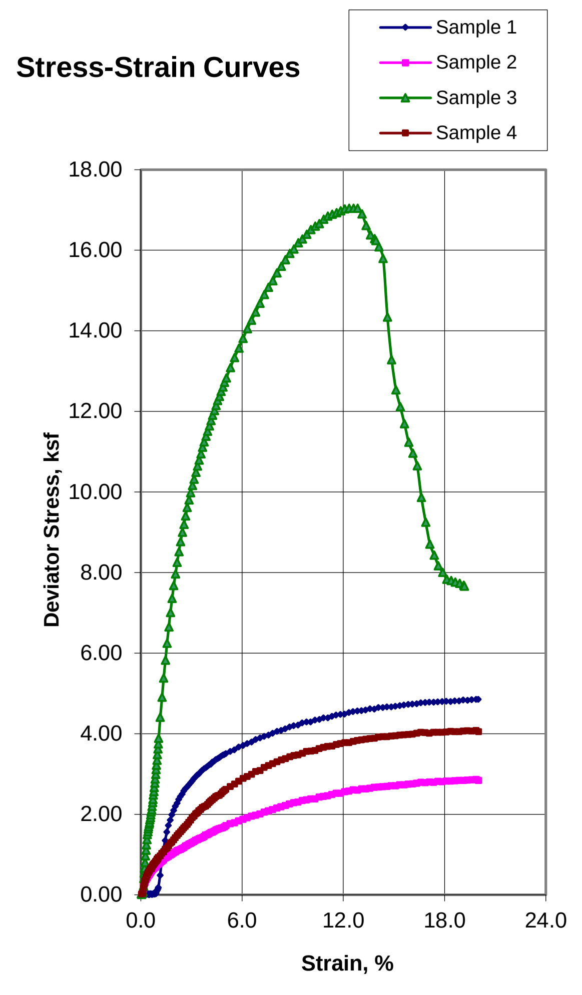
Number: 122-5-1

US GRAIN SIZE - CORNERSTONE 0812.GDT - 3/6/15 09:13 - P:\DRAFTING\GINT FILES\122-5-1 215 CALIFORNIA DR.GPJ

Unconsolidated-Undrained Triaxial Test

ASTM D2850





Sample Data				
	1	2	3	4
Moisture %	27.3	31.1	21.6	21.5
Dry Den,pcf	98.0	93.3	107.7	108.3
Void Ratio	0.784	0.873	0.622	0.614
Saturation %	97.6	99.8	97.1	98.1
Height in	5.00	4.99	5.03	5.00
Diameter in	2.41	2.40	2.41	2.40
Cell psi	17.4	20.8	25.0	27.8
Strain %	15.00	15.00	12.35	15.00
Deviator, ksf	4.668	2.710	17.047	3.941
Rate %/min	1.00	1.00	1.00	1.00
in/min	0.050	0.050	0.050	0.050
Job No.:	**640-777**			
Client:	**Cornerstone Earth Group**			
Project:	**215 California Dr - 122-5-1**			
Boring:	EB-1	EB-1	EB-1	EB-1
Sample:	12A	16A	19B	21B
Depth ft:	39	54	69	79

Visual Soil Description

Sample #	
1	Yellowish Brown CLAY, trace Sand
2	Yellowish Brown Sandy CLAY
3	Light Olive Brown CLAY, trace Sand
4	Yellowish Brown CLAY w/ Sand & Gravel

Remarks:

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

COOPER
TESTING LABORATORY

Hydraulic Conductivity
ASTM D 5084
Method C: Falling Head Rising Tailwater

Job No: 640-777 **Boring:** EB-1 **Date:** 03/10/15
Client: Cornerstone Earth Group **Sample:** 4A **By:** MD/PJ
Project: 215 California Dr - 122-5-1 **Depth, ft.:** 9.0 **Remolded:**
Visual Classification: Reddish Brown Clayey GRAVEL w/ Sand

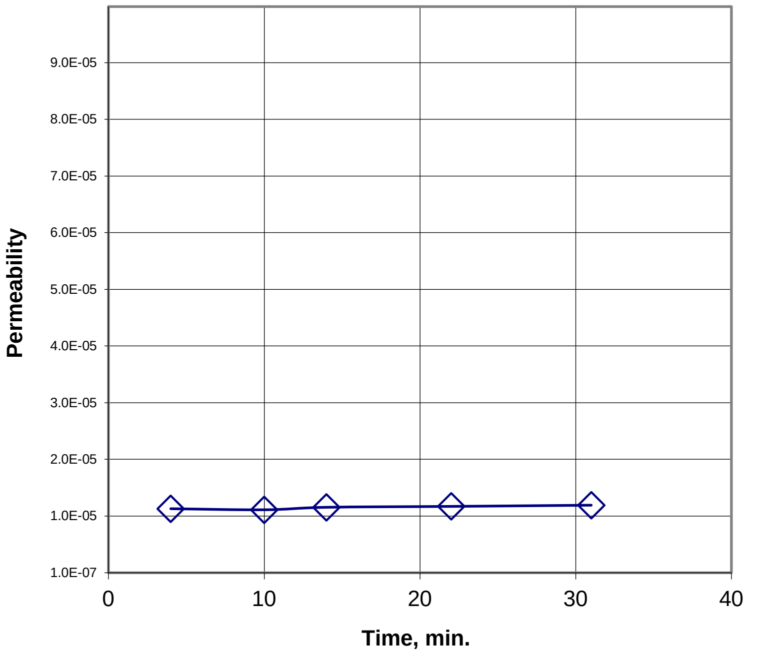
Max Sample Pressures, psi:

Cell:	Bottom	Top	Avg. Sigma3
64	59	59	5

B: = >0.95 **("B" is an indication of saturation)**

Max Hydraulic Gradient: = 11

Date	Minutes	Head, (in)	K,cm/sec
3/5/2015	0.00	24.00	**Start of Test**
3/5/2015	4.00	22.95	1.1E-05
3/5/2015	10.00	21.50	1.1E-05
3/5/2015	14.00	20.45	1.2E-05
3/5/2015	22.00	18.60	1.2E-05
3/5/2015	31.00	16.65	1.2E-05



Average Hydraulic Conductivity: 1.E-05 cm/sec

Sample Data:	**Initial (As-Received)**		**Final (At-Test)**
Height, in	2.52		2.19
Diameter, in	2.41		2.46
Area, in2	4.55		4.74
Volume in3	11.44		10.38
Total Volume, cc	187.5		170.2
Volume Solids, cc	112.8		112.8
Volume Voids, cc	74.7		57.3
Void Ratio	0.7		0.5
Total Porosity, %	39.8		33.7
Air-Filled Porosity (θa),%	17.5		0.8
Water-Filled Porosity (θw),%	22.3		32.9
Saturation, %	56.1		97.6
Specific Gravity	2.70	Assumed	2.70
Wet Weight, gm	346.6		360.6
Dry Weight, gm	304.7		304.7
Tare, gm	0.00		0.00
Moisture, %	13.8		18.4
Wet Bulk Density, pcf	115.3		132.2
Dry Bulk Density, pcf	101.4		111.7
Wet Bulk Dens.ρb, (g/cm^3)	1.85		2.12
Dry Bulk Dens.ρb, (g/cm^3)	1.62		1.79

Remarks: +3/4" Gravel noted after test. The sample slumped after the test. Therefore the post-test dimensions, and all associated values, are approximate.

Hydraulic Conductivity
ASTM D 5084
Method C: Falling Head Rising Tailwater

Job No:	640-777	**Boring:**	EB-1	**Date:**	03/12/15
Client:	Cornerstone Earth Group	**Sample:**	6A	**By:**	MD/PJ
Project:	215 California Dr - 122-5-1	**Depth, ft.:**	15	**Remolded:**	

Visual Classification: Yellowish Brown Silty SAND (slightly plastic)

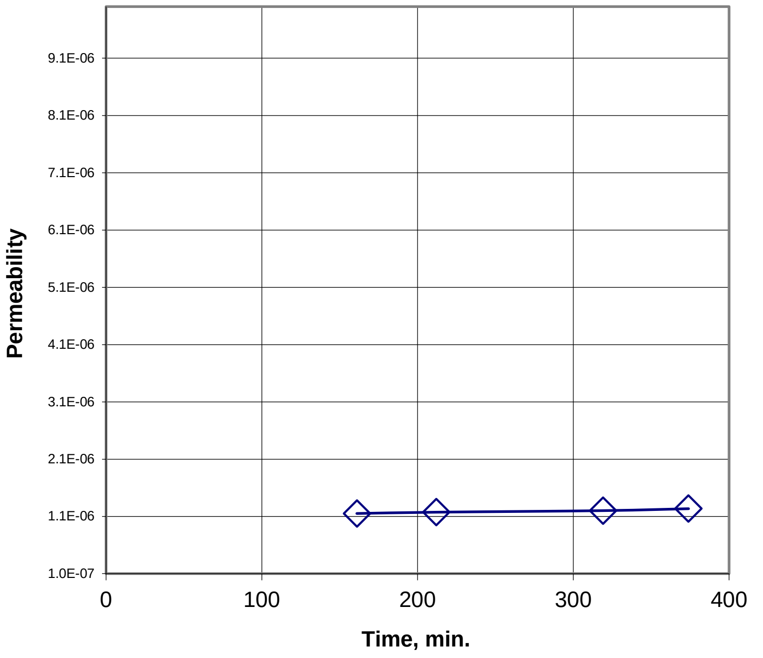
Max Sample Pressures, psi:

Cell:	Bottom	Top	Avg. Sigma3
53.5	49	48	5

B: = >0.95 **("B" is an indication of saturation)**

Max Hydraulic Gradient: = 17

Date	Minutes	Head, (in)	K,cm/sec
3/10/2015	0.00	42.69	Start of Test
3/10/2015	161.00	36.59	1.2E-06
3/10/2015	212.00	34.69	1.2E-06
3/10/2015	319.00	30.99	1.2E-06
3/10/2015	374.00	28.99	1.2E-06



Average Hydraulic Conductivity: **1.E-06** **cm/sec**

Sample Data:	Initial (As-Received)		Final (At-Test)
Height, in	2.51		2.48
Diameter, in	2.42		2.42
Area, in2	4.58		4.58
Volume in3	11.50		11.34
Total Volume, cc	188.4		185.8
Volume Solids, cc	125.5		125.5
Volume Voids, cc	62.9		60.3
Void Ratio	0.5		0.5
Total Porosity, %	33.4		32.5
Air-Filled Porosity (θa),%	0.8		0.7
Water-Filled Porosity (θw),%	32.7		31.8
Saturation, %	97.8		97.9
Specific Gravity	2.70	Assumed	2.70
Wet Weight, gm	400.3		397.8
Dry Weight, gm	338.8		338.8
Tare, gm	0.00		0.00
Moisture, %	18.2		17.4
Wet Bulk Density, pcf	132.6		133.6
Dry Bulk Density, pcf	112.2		113.8
Wet Bulk Dens.ρb, (g/cm^3)	2.12		2.14
Dry Bulk Dens.ρb, (g/cm^3)	1.80		1.82

Remarks:

COOPER TESTING LABORATORY

Hydraulic Conductivity
ASTM D 5084
Method C: Falling Head Rising Tailwater

Job No:	640-777	**Boring:**	EB-1	**Date:**	03/12/15
Client:	Cornerstone Earth Group	**Sample:**	11B	**By:**	MD/PJ
Project:	215 California Dr - 122-5-1	**Depth, ft.:**	34.5	**Remolded:**	

Visual Classification: Reddish Brown Clayey GRAVEL w/ Sand

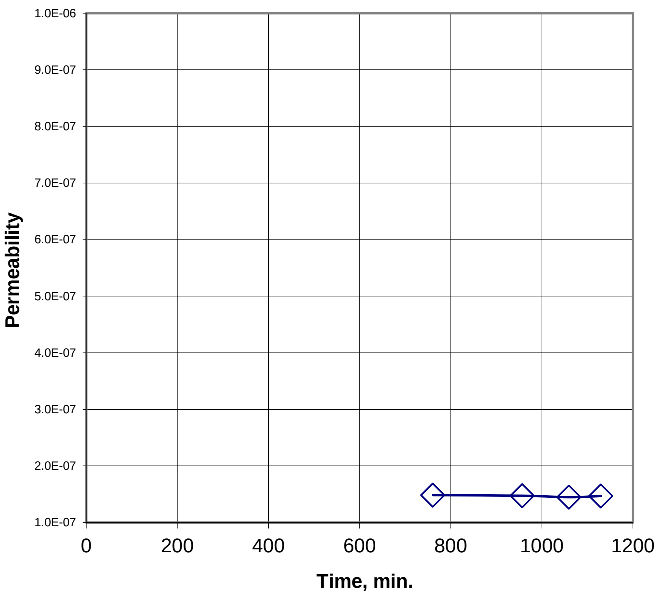
Max Sample Pressures, psi:

Cell:	Bottom	Top	Avg. Sigma3
74	69.5	68.5	5

B: = >0.95 **("B" is an indication of saturation)**

Max Hydraulic Gradient: = 17

Date	Minutes	Head, (in)	K,cm/sec
3/10/2015	0.00	42.69	**Start of Test**
3/11/2015	761.00	38.84	1.5E-07
3/11/2015	957.00	37.99	1.5E-07
3/11/2015	1060.00	37.59	1.4E-07
3/11/2015	1130.00	37.19	1.5E-07



Average Hydraulic Conductivity: **1.E-07** **cm/sec**

Sample Data:	**Initial (As-Received)**		**Final (At-Test)**
Height, in	2.50		2.50
Diameter, in	2.42		2.42
Area, in2	4.59		4.59
Volume in3	11.48		11.46
Total Volume, cc	188.0		187.7
Volume Solids, cc	122.5		122.5
Volume Voids, cc	65.5		65.2
Void Ratio	0.5		0.5
Total Porosity, %	34.8		34.7
Air-Filled Porosity (θa),%	0.8		0.1
Water-Filled Porosity (θw),%	34.0		34.7
Saturation, %	97.7		99.8
Specific Gravity	2.75	Assumed	2.75
Wet Weight, gm	400.9		402.0
Dry Weight, gm	336.9		336.9
Tare, gm	0.00		0.00
Moisture, %	19.0		19.3
Wet Bulk Density, pcf	133.0		133.6
Dry Bulk Density, pcf	111.8		112.0
Wet Bulk Dens.ρb, (g/cm^3)	2.13		2.14
Dry Bulk Dens.ρb, (g/cm^3)	1.79		1.79

Remarks:

APPENDIX C: SITE CORROSIVITY EVALUATION

JDH CORROSION CONSULTANTS REPORT DATED __________

Corrosivity Tests Summary

CTL # 640-780
Date: 3/24/2015
Tested By: PJ
Checked: PJ
Client: Cornerstone Earth Group
Project: 215 California Dr
Proj. No: 122-5-1
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	2A	4.0	-	-	2,150	14	70	0.0070	7.3	-	-	-	15.7	Dark Brown Sandy CLAY
EB-1	3A	6.0	-	-	2,267	16	44	0.0044	7.1	-	-	-	19.2	Dark Yellowish Brown Sandy CLAY
EB-1	6B	15.5	-	-	2,335	9	3	0.0003	6.8	-	-	-	19.1	Yellowish Brown CLAY w/ Sand

jdh corrosion consultants, inc.

Protecting the infrastructure
through innovative
Corrosion Engineering Solutions

March 27, 2015

Cornerstone Earth Group
1259 Oakmead Parkway
Sunnyvale, California 94085

Attention: **Laura Knutson, P.E., G.E.**
Principal Engineer

Subject: **Site Corrosivity Evaluation**
215 California Drive
Burlingame, CA
Job #: 122-5-1

Dear Laura,

In accordance with your request, we have reviewed the laboratory soils data for the above referenced project site. Our evaluation of these results and our corresponding recommendations for corrosion control for the above referenced project foundations and buried site utilities are presented herein for your consideration.

Soil Testing & Analysis

Soil Chemical Analysis

Three (3) soil samples from the project site were chemically analyzed for corrosivity by ***Cooper Testing Laboratories***. Each sample was analyzed for chloride and sulfate concentration, pH, resistivity at 100% saturation and moisture percentage. The test results are presented in Cooper Testing Laboratories *Corrosivity Test Summary* dated 3/24/2015. The results of the chemical analysis were as follows:

Soil Laboratory Analysis

Chemical Analysis	Range of Results	Corrosion Classification*
Chlorides	9 - 16 mg/kg	Non-corrosive*
Sulfates	3 – 70 mg/kg	Non-corrosive**
pH	6.8 – 7.3	Non-corrosive *
Moisture (%)	15.7 – 19.2 %	Not-applicable
Resistivity at 100% Saturation	2,150 – 2,335 ohm-cm	**Moderately Corrosive***

* With respect to bare steel or ductile iron.
** With respect to mortar coated steel

Discussion

Reinforced Concrete Foundations

Due to the low levels of water-soluble sulfates found in these soils, there is no special requirement for sulfate resistant concrete to be used at this site. The type of cement used should be in accordance with California Building Code (CBC) for soils which have less than 0.10 percent by weight of water soluble sulfate (SO_4) in soil and the minimum depth of cover for the reinforcing steel should be as specified in CBC as well.

Underground Metallic Pipelines

The soils at the project site are generally considered to be “moderately corrosive” to ductile/cast iron, steel and dielectric coated steel based on the saturated resistivity measurements. Therefore, special requirements for corrosion control are required for buried metallic utilities at this site depending upon the critical nature of the piping. Pressure piping systems such as domestic and fire water should be provided with appropriate coating systems and cathodic protection, where warranted. In addition, all underground pipelines should be electrically isolated from above grade structures, reinforced concrete structures and copper lines in order to avoid potential galvanic corrosion problems.

LIMITATIONS

The conclusions and recommendations contained in this report are based on the information and assumptions referenced herein. All services provided herein were performed by persons who are experienced and skilled in providing these types of services and in accordance with the standards of workmanship in this profession. No other warrantees or guarantees, expressed or implied, is provided.

We thank you for the opportunity to be of service to ***Cornerstone Earth Group*** on this project and trust that you find the enclosed information satisfactory. If you have any questions, or if we can be of any additional assistance, please feel free to contact us at (925) 927-6630.

Respectfully submitted,

Mohammed Ali

Mohammed Ali, P.E.
JDH Corrosion Consultants, Inc.
Principal

REGISTERED PROFESSIONAL ENGINEER
MOHAMMED ALI
No. CR 1035
Exp. June 30, 2015
CORROSION
STATE OF CALIFORNIA

Brendon Hurley

Brendon Hurley
JDH Corrosion Consultants, Inc.
Corrosion Technician

CC: File 15042

San Mateo County Planning and Development Department
n.d. "Historical and Archaeological Resources, Section 5" from the *San Mateo CountyGeneral Plan*.

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1988 *Five Views: An Ethnic Sites Survey for California*. State of California Department of Parks and Recreation and Office of Historic Preservation, Sacramento.

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2012 *Historic Properties Directory*. Listing by City (through April 2012). State of California Office of Historic Preservation, Sacramento.

Williams, James C.
1997 *Energy and the Making of Modern California*. The University of Akron Press, Akron, OH.

Woodbridge, Sally B.
1988 *California Architecture: Historic American Buildings Survey*. Chronicle Books, San Francisco, CA.

******Note that the Office of Historic Preservation's *Historic Properties Directory* includes National Register, State Registered Landmarks, California Points of Historical Interest, and the California Register of Historical Resources as well as Certified Local Government surveys that have undergone Section 106 review.