Appendix F: Geotechnical Investigation

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CORNERSTONE
EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**Hyatt House - Half Moon Bay**
Location	**Seymour Street and Cabrillo Highway Half Moon Bay, California**
Client	**RGJC South LLC**
Client Address	**P.O. Box 3095 Half Moon Bay, CA**
Project Number	**866-1-1**
Date	**July 13, 2016**

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Type of Services	**Geotechnical Investigation**
Project Name	**Hyatt House - Half Moon Bay**
Location	**Seymour Street and Cabrillo Highway Half Moon Bay, California**

SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of RGJC South LLC for the Hyatt House - Half Moon Bay in Half Moon Bay, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- A set of civil plans titled, "Design Submittal, James Ford – Half Moon Bay, Automotive Dealership," prepared by BKF Engineers/Surveyors/Planners and RYS Architects, Inc., dated December 21, 2015.
- A site plan titled, "Site Plan – Hyatt House," prepared by RYS Architects, Inc., dated November 9, 2015
- A report titled, "Geotechnical Investigation, Proposed Ford Dealership Expansion," prepared by Construction Testing & Engineering, Inc., dated February 18, 2013
- A topographical map titled, "Boundary and Topographic Survey – Main/Seymour Streets/Highway 1," prepared by BGT Land Surveying, dated June 2012
- A set of monitoring well logs prepared by BACE Environmental, October, 1993 through December, 1993

1.1 PROJECT DESCRIPTION

The project will consist of new at-grade, three-story hotel on the approximately 5-acre site. The structure will be of wood and/or steel-frame construction, and appurtenant parking, utilities, landscaping, and other improvements are also planned. Cuts and fills up to 4 feet are anticipated.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated November 25, 2015, revised December 8, 2015, and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of 5 borings drilled on January 12, 2016 with track-mounted hollow-stem auger drilling equipment. The borings were drilled to depths ranging from 15 to 40 feet. The borings were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions. In addition, a location was drilled to 5½ feet to perform a percolation test.

Prior to our drilling program, a location was hand-augered to 8 feet in depth to observe if ground water was present at that depth.

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

1.4 LABORATORY TESTING PROGRAM

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

1.5 CORROSION EVALUATION

Two samples from our borings from depths from 3½ to 6½ feet were tested for saturated resistivity, pH, and soluble sulfates and chlorides. 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

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

SECTION 2: REGIONAL SETTING

2.1 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 Gregorio	2.0	3.2
San Andreas (1906)	6.1	9.8
Monte Vista-Shannon	10.4	16.8

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 undeveloped, and mostly covered in clover plants, grasses, and weeds. It gently slopes from the south side (approximate elevation of 90 feet) and northeast corner (approximate elevation of 89 feet) of the site to the northwest corner (approximate elevation of 85 feet). From historic photographs, it appears that no types of development has ever occurred at this site. During our site visits, some areas around the site appeared to be very saturated, and ponding was also observed on the surface.

The site is bounded by Main Street to the south and east, Cabrillo Highway to the west, and the James Ford dealership to the north.

3.2 SUBSURFACE CONDITIONS

Our explorations encountered predominately soft to medium stiff, dark gray lean clay with organics in the upper 1½ to 3½ feet below the surface. Based on the amount of organic materials, these surficial clays are considered to be inorganic (ASTM 2487). Below the surficial clays, we generally encountered stiff to hard lean clays down to approximately 10 feet; however, EB-2 encountered a thin layer of loose clayey sand, followed by stiff lean clay with sand, after the surficial clays. EB-1 shows mostly very stiff lean clays and sandy lean clays. A layer of dense clayey sand about 8 feet thick was encountered beginning at 13 feet deep. Additionally, a layer of medium dense to dense silty sand about 7 feet thick was encountered at approximately 30 feet deep, followed by very stiff lean clay with sand until the end of the boring at 40 feet. The stiff to hard lean clays continued after 10 feet deep to about 1½ feet before the bottoms of EB-2 and EB-3 where medium dense to dense clayey sands with gravels were encountered. EB-4 and EB-5 showed medium dense to dense clayey sands with gravels from about 12 feet deep until the bottom of the borings at 35 feet and 15 feet, respectively.

A consolidation test was performed on a sample from approximately 9 feet from EB-3 to evaluate the compressibility of the soils under assumed building loads. Based on the results, the subsurface soils at this site have low to moderate compressibility.

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 surficial PI tests indicated a PI of 22, indicating moderate expansion potential to wetting and drying cycles.

3.2.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 4 feet range from 5 to 11 over the estimated laboratory optimum moisture, and 2 to 8 percent from 4 to 10 feet.

3.2.3 Sulfate Contents

Laboratory testing indicated that the soluble sulfate contents were 25 to 46 ppm, indicating negligible corrosion potential to buried concrete.

3.2.4 Organic Contents

Laboratory testing indicated that the top 1½ to 3½ feet of material consists approximately 7 percent of organic materials.

3.3 GROUND WATER

Ground water was encountered in some of our explorations at depths ranging from 15 to 30 feet below current grades, corresponding to Elevations 70.2 to 59.4 feet (MSL datum). 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.

We were provided copies of borings logs by BACE Environmental dated October, 1993 through December, 1993 for the adjacent Ford Dealership. This information indicated that the ground water was about 33 to 34 feet below the ground surface.

Construction Testing & Engineering, Inc. repeated that in January of 2013, the ground water was observed at approximately 4 feet below the existing grade in their borings, and 2.8 feet below the existing grade in their percolation test holes. However, we do not believe this to be reliable data as heavy rainfall was occurring during drilling. We also hand-augered down to 8 feet, but no ground water was encountered shortly after a significant series of rainstorms.

Additionally, according to the GeoTracker website, monitoring wells located approximately 2,000 feet to the north of the site indicate high ground water levels of 20 to 23 feet below the existing grade between 2011 and 2014, though the level rose to approximately 16½ below the existing grade in around January of 2013. Based on this information, we recommend a design ground water level of 12 feet below the existing ground surface which includes 3 feet for variations.

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

Table 2: Depth to Ground Water

Boring/CPT Number	Date Drilled	Depth to Ground Water (feet)	Ground Water Elevation* (feet)	Depth of Boring (feet)
EB-1	1/12/16	30	59.4	40
EB-4	1/12/16	15	70.2	35

*Elevation datum (MSL, from reference, etc.)

3.4 CORROSION SCREENING

We tested two samples collected at depths of 3½ and 6½ feet for resistivity, pH, soluble sulfates, and chlorides. The laboratory test results are summarized in Table 2.

Table 3: Summary of Corrosion Test Results

Sample/Test Location Number	Depth (feet)	Soil pH	Minimum Resistivity (1) (ohm-cm)	Chloride (mg/kg)	Sulfate (% dry wt)
EB-1	3½	6.6	2,236	30	0.0046
EB-4	6½	6.8	2,454	9	0.0025

Notes: (1) Laboratory resistivity measured at 100% saturation

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

3.4.1 Preliminary Soil Corrosion Screening

Based on the laboratory test results summarized in Table 2, the soils are considered moderately corrosive to buried metallic improvements (Palmer, 1989). Other corrosion parameters (pH and chloride content) do not indicate a significant contribution to corrosion potential to buried metallic structures. In accordance with the 2013 CBC, Chapter 19, Section 1904.5, alternative cementitious materials for sulfate exposure shall be in accordance with the following:

- ACI 318-11 - Table 4.2.1, and Table 4.3.1

Based on the laboratory test results, no cement type restriction is required, although, in our opinion, it is generally a good idea to include some sulfate resistance and to maintain a relatively low water-cement ratio. We have summarized applicable design values and parameters from ACI 318, Table 4.3.1 below in Table 3 for your information. We recommend the structural engineer and a corrosion engineer be retained to confirm the information provided and for additional recommendations, as required.

Table 4: Sulfate Soil Corrosion Design Values and Parameters (1)

Category	Water-Soluble Sulfate (SO4) in Soil (% by weight)	Class	Severity	Cementitious Materials (2)
S, Sulfate	< 0.10	S0	not applicable	no type restriction

Notes: (1) above values and parameters are from on ACI 318-08, Table 4.2.1 and Table 4.3.1
(2) cementitious materials are in accordance with ASTM C150, ASTM C595 and ASTM C1157

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 about 0.80g.

4.3 LIQUEFACTION POTENTIAL

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

4.3.1 Background

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

4.3.2 Analysis

As discussed in the “Subsurface” section above, several sand layers were encountered below the design ground water depth of 12 feet. Following the liquefaction analysis framework in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008), incorporating updates in *CPT and SPT Based Liquefaction Triggering Procedures* (Boulanger and Idriss, 2014), and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008) for quantitative analysis, these layers were analyzed for liquefaction triggering and potential post-liquefaction settlement. These methods compare the ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil’s estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.3 are considered to be potentially liquefiable and capable of post-liquefaction re-consolidation (i.e. settlement).

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

The soil’s CRR is estimated from laboratory testing on samples retrieved from our borings. SPT “N” values obtained from hollow-stem auger borings were used in our analyses and corrected for effective overburden stresses. Soils that have corrected SPT blow counts greater than 30 blows per foot are considered too dense to liquefy and have been screened out of our analysis.

4.3.3 Summary

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

4.3.4 Ground Rupture Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground rupture or sand boils. For ground rupture to occur, the pore water pressure within the liquefiable soil layer will need to be great enough to break through the overlying non-liquefiable layer, which could cause significant ground deformation and settlement. Because the potential for liquefaction to occur is very low at this site, ground rupture is not anticipated to be an issue.

4.4 LATERAL SPREADING

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

There are no open faces within a distance considered susceptible to lateral spreading; therefore, in our opinion, the potential for lateral spreading to affect the site is 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.

4.6 TSUNAMI/SEICHE

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

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

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the study of tsunami inundation potential for the San Francisco Bay Area (Ritter and Dupre, 1972), areas most likely to be inundated are marshlands, tidal flats, and former bay margin lands that are now artificially filled, but are still at or below sea level, and are generally within 1½ miles of the shoreline. The site is approximately 1 mile inland from the Pacific Ocean shoreline; however, it is approximately 85 to 90 feet above mean sea level. ABAG also indicates that it is outside of a tsunami evacuation zone. Therefore, the potential for inundation due to tsunami or seiche is considered low.

4.8 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone X, and area “determined to be outside the 0.2 annual chance floodplain.” We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

From a geotechnical viewpoint, the project is feasible provided the concerns listed below are addressed in the project design. Descriptions of each concern with brief outlines of our recommendations follow the listed concerns.

- Presence of Organics and Highly Compressible Surficial Soils
- Potentially Shallow Ground Water
- Presence of Moderately Expansive Soils
- Soil Corrosion Potential

5.1.1 Presence of Organics and Highly Compressible Surficial Soils

Field and laboratory testing indicated that the top 1½ to 3½ feet below the existing grades consists of soft to medium stiff lean clay with approximately 6 to 7 percent organics and highly compressible surficial soils in the upper 6 to 12 inches. Organic material can decompose over time and leave voids in the soil. Additionally, the soil in the upper 1½ to 3½ feet is highly compressible under the proposed building loads. To mitigate this concern, we would recommend the upper 2 to 3 feet (see Figure 2B) be over-excavated and replaced as recompacted engineered fill. The organic-laden soils will be mixed with the underlying soils to reduce the organic content and the resulting mixture will be considered inorganic per ASTM 2487. See the “Earthwork” section of this report for further recommendations to address this issue.

5.1.2 Potentially Shallow Ground Water

Ground water was measured at depths ranging from approximately 15 to 30 feet below the existing ground surface. We anticipate that ground water may rise to depths as high as 12 feet below the existing ground surface. Our experience with similar sites indicates that shallow ground water could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable pavement subgrade, difficulty achieving compaction, and difficult underground utility installation. Shoring of utility trenches may be required in some isolated areas of the site. Detailed recommendations addressing this concern are presented in the “Earthwork” section of this report.

5.1.3 Presence of Moderately Expansive Soils

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

5.1.4 Soil Corrosion Potential

Preliminary soil corrosion data was collected based on the results of the analytical tests on a sample of the near-surface soil. In general, the corrosion potential for buried concrete does not warrant the use of sulfate resistant concrete; however, the corrosion potential for buried metallic structures, such as metal pipes, is considered moderately corrosive. Based on the results of the preliminary soil corrosion screening, special requirements for corrosion control will likely be required to protect metal pipes and fittings. We recommend that a corrosion engineering specialist be retained for corrosion protection recommendations.

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. Surface vegetation and topsoil should be stripped to a sufficient depth and placed in landscaping areas. Based on our site observations, surficial stripping should extend about 3 to 6 inches below existing grade in vegetated areas. If mature grass is present at the time of grading, we would recommend mowing the site a few day before stripping.

6.1.2 Tree and Shrub Removal

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

6.1.3 Abandonment of Existing Utilities

All utilities should be completely removed from within planned building areas. For any utility line to be considered acceptable to remain within building areas, the utility line must be completely backfilled with grout or sand-cement slurry (sand slurry is not acceptable), the ends outside the building area capped with concrete, and the trench fills either removed and replaced as engineered fill with the trench side slopes flattened to at least 1:1, or the trench fills are determined not to be a risk to the structure. The assessment of the level of risk posed by the particular utility line will determine whether the utility may be abandoned in place or needs to be completely removed. The contractor should assume that all utilities will be removed from within building areas unless provided written confirmation from both the owner and the geotechnical engineer.

Utilities extending beyond the building area may be abandoned in place provided the ends are plugged with concrete, they do not conflict with planned improvements, and that the trench fills do not pose significant risk to the planned surface improvements.

The risks associated with abandoning utilities in place include the potential for future differential settlement of existing trench fills, and/or partial collapse and potential ground loss into utility lines that are not completely filled with grout. In general, the risk is relatively low for single utility lines less than 4 inches in diameter, and increases with increasing pipe diameter.

6.2 SOFT TO MEDIUM STIFF SURFICIAL SOILS

As previously mentioned, the site is blanketed by 1½ to 3½ of dark gray lean clay with approximately 6 to 7 percent organics in the upper few inches. To provide a more uniform bearing surface for structural support and to minimize the potential for significant differential settlement beneath new structures, the soft to medium stiff soils should be over-excavated and replaced with engineered fill.

We recommend the upper 2 feet of soils within the new structures be over-excavated and replaced as engineered fill; however, on the north side of the site, the upper 3 feet of soils within the new structures should be over-excavated replaced as engineered fill as shown on Figure 2B. The exposed subgrade should be scarified 6 to 8 inches, moisture conditioned, and compacted in place. The limit of engineered fill should extend at least 5 feet beyond the building footprint and foundations. We note that the soils are about 5 to 11 percent above optimum in the soils to be over-excavated. This will require the soils to be dried out prior to re-use. Subgrade preparation and compaction should be performed in accordance with the following sections.

6.3 REMOVAL OF EXISTING FILLS

While fills were not encountered in our borings, any fills encountered during site grading should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below.

6.4 TEMPORARY CUT AND FILL SLOPES

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

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

6.5 SUBGRADE PREPARATION

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

6.6 SUBGRADE STABILIZATION MEASURES

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

As discussed in the “Subsurface” section in this report, the in-situ moisture contents are about 5 to 11 percent over the estimated laboratory optimum in the upper 4 feet of the soil profile, and 2 to 8 percent from 4 to 10 feet. A compaction curve was performed on the surficial soils on February 24, 2016 to obtain the maximum dry density and optimum moisture content. Based on the results, the material has a maximum dry density of 111 pounds per cubic foot (pcf) with an optimum moisture content of 14½ percent. The contractor should anticipate drying the soils prior to reusing them as fill. In addition, repetitive rubber-tire loading will likely de-stabilize the soils.

There are several methods to address potential unstable soil conditions and facilitate fill placement and trench backfill. Some of the methods are briefly discussed below. Implementation of the appropriate stabilization measures should be evaluated on a case-by-case basis according to the project construction goals and the 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.

6.7 MATERIAL FOR FILL

6.7.1 Re-Use of On-site Soils

As previously discussed, the upper 2 to 3 feet of surficial soils should be removed and replaced with engineered fill. While the surficial soils contain approximately 6 to 7 percent organics, laboratory testing indicated that it may be reused as engineered fill if they are mixed into the soils below. 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 and non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less, and not contain recycled asphalt concrete where it will be used within the building areas. To prevent significant caving during trenching or foundation construction, imported material should have sufficient fines. Samples of potential import sources should be delivered to our office at least 10 days prior to the desired import start date. Information regarding the import source should be provided, such as any site geotechnical reports. If the material will be derived from an excavation rather than a stockpile, potholes will likely be required to collect samples from throughout the depth of the planned cut that will be imported. At a minimum, laboratory testing will include PI tests. Material data sheets for select fill materials (Class 2 aggregate base, ¾-inch crushed rock, quarry fines, etc.) listing current laboratory testing data (not older than 6 months from the import date) may be provided for our review without providing a sample. If current data is not available, specification testing will need to be completed prior to approval.

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

6.7.3 Non-Expansive Fill Using Lime Treatment

As discussed above, non-expansive fill should have a Plasticity Index (PI) of 15 or less. Due to the high clay content and PI of the on-site soil materials, it is not likely that sufficient quantities of non-expansive fill would be generated from cut materials. As an alternative to importing non-expansive fill, chemical treatment can be considered to create non-expansive fill. It has been our experience that for high PI clayey soil and bedrock materials will likely need to be mixed with at least 4 percent quicklime (CaO) or approved equivalent to adequately reduce the PI of the on-site soils to 15 or less. If this option is considered, additional laboratory tests should be performed during initial site grading to further evaluate the optimum percentage of quicklime required.

6.8 COMPACTION REQUIREMENTS

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

Table 5: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Expansive Soils	87 – 92	>3
	Low Expansion Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Expansive Soils	93	>3
	Low Expansion Soils	95	>1
Trench Backfill	On-Site Expansive Soils	87 – 92	>3
Trench Backfill	Low Expansion Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Low Expansion Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum
Flatwork Subgrade	On-Site Expansive Soils	87 - 92	>3
Flatwork Subgrade	Low Expansion Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Expansive Soils	87 - 92	>3

Pavement Subgrade	Low Expansion Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	NA

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

6.8.1 Construction Moisture Conditioning

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

6.9 TRENCH BACKFILL

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

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

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

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

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

6.10 SITE DRAINAGE

Ponding should not be allowed adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 2 percent towards suitable discharge facilities; landscape areas should slope at least 3 percent towards suitable discharge facilities. Roof runoff should be directed away from building areas 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 not mapped in the area, but was encountered as high as 15 feet below grade in our borings; we are using a design ground water level of 12 feet and therefore is expected to be at least 10 feet below the base of the infiltration measure.

- The site is not known, to our knowledge, to have pollutants with the potential for mobilization as a result of stormwater infiltration.

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

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

6.11.1 Storm Water Treatment Design Considerations

If storm water treatment improvements, such as shallow bio-retention swales, basins or pervious pavements, are required as part of the site improvements to satisfy Storm Water Quality (C.3) requirements, we recommend the following items be considered for design and construction.

6.11.1.1 General Bioswale Design Guidelines

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

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

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

6.11.1.2 Bioswale Infiltration Material

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

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

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

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

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

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

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

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

6.11.1.3 Bioswale Construction Adjacent to Pavements

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

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

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

6.12 LANDSCAPE CONSIDERATIONS

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

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

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

SECTION 7: SOIL PERMEABILITY AND GROUND WATER INFILTRATION

7.1 GENERAL

To estimate the infiltration rate and hydraulic conductivity of the soils, we attempted to perform one in-situ field permeability tests using a Guelph permeameter by SoilMoisture Equipment Corp., Model #2800, in accordance with ASTM D5126. Our test location and depth were coordinated with BKF Engineers/Surveyors/Planners and based on the provided sets of plans. Generally, the Guelph permeameter is a constant head device, which uses two water-filled chambers to measure infiltration rate in a shallow borehole. A constant head level is established in the borehole and the rate of water outflow into the surrounding soil is noted. The rate of flow when it reaches a steady state, or constant rate, is used to determine the soil characteristics such as the saturated conductivity and permeability. Our field permeability test was performed at the terminal boring depth of approximately 5½. The location of our permeability test is shown on Figure 2.

To prepare for the test, the borehole was drilled, and then sidewalls were scarified to reduce “smeared” fine-grained soil caused by the drilling process, which can decrease infiltration. Water was then poured into the hole to allow pre-saturation; however, very little water had percolated prior to the first reading several hours later. The hole was then checked three days later and it had percolated approximately one inch. The hole was backfilled after this reading.

The site soils are basically impermeable. Based on our experience and engineering judgment this appears reasonable for the clayey soils encountered in the borings across the site at this depth.

7.1.1 Reliability of Field and Laboratory Test Data

Test results may not be truly indicative of the long-term, in-situ permeability. Other factors including stratifications, heterogenous deposits, overburden stress, and other factors can influence permeability results. In addition, for stratified soils such as those encountered at the site, the average horizontal permeability is typically greater than the average vertical permeability.

7.1.2 Findings and Recommendations

Based on our findings, the percolation rate of our test location appears reasonable for clayey soils encountered in the borings across the site, and appear to not be favorable for the near-surface infiltration or percolation with low permeability rates.

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

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

8.2 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 *Design Maps*, located at http://earthquake.usgs.gov/hazards/designmaps/usdesign.php, based on the site coordinates presented below and the site classification. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 6: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.452486°
Site Longitude	-122.730176°
0.2-second Period Mapped Spectral Acceleration[1], S_S	2.071g
1-second Period Mapped Spectral Acceleration[1], S_1	0.881g

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}	2.071g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.321g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.381g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.881g

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

8.3 SHALLOW FOUNDATIONS

8.3.1 Spread Footings

Spread footings should bear on natural, undisturbed soil or engineered fill, be at least 18 inches wide, and extend at least 24 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil. The deeper footing embedment is due to the presence of moderately expansive soils, and is intended to embed the footing below the zone of significant seasonal moisture fluctuation, reducing the potential for differential movement. In addition, the deeper footing embedment is so that the structure is founded at or below the bottom of the surficial soil with organics.

8.3.2 Footing Settlement

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

Table 7: Assumed Structural Loading

Foundation Area	Range of Assumed Loads
Interior Isolated Column Footing	300 kips
Exterior Isolated Column Footing	150 kips
Perimeter Strip Footing	4 to 6 kips per lineal foot

Based on the above loading and the allowable bearing pressures presented above and recommended remedial grading discussed previously, we estimate that the total static footing settlement will be on the order of ¾-inch, with about ½-inch of post-construction differential settlement between adjacent foundation elements, assumed to be on the order of 30 feet. As our footing loads were assumed, we recommend we be retained to review the final footing layout and loading, and verify the settlement estimates above.

Approximately ¼-inch of the total settlement discussed above is due to primary consolidation of saturated clay layers. The time to the achieve about 90 to 95 percent of the primary consolidation is anticipated to take several months to a year after all the dead and live loads are in place based on the encountered alluvial conditions. The contractor should take this into consideration when scheduling the construction of sensitive finishes.

8.3.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of footing and the supporting subgrade, and also by passive pressures generated against footing sidewalls. An ultimate frictional resistance of 0.40 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

8.3.4 Spread Footing Construction Considerations

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

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

8.4 MAT FOUNDATIONS

As an alternative to spread footings, the structure may be supported on mat foundations bearing on engineered fill prepared in accordance with the “Earthwork” section of this report, and designed in accordance with the recommendations below.

8.4.1 Reinforced Concrete Mats

The mat foundation may be designed for a maximum average allowable bearing pressure of 1,000 pounds per square foot (psf) for dead plus live loads; at column or wall loading locations the maximum localized bearing pressure should not exceed 3,750 psf. When evaluating wind and seismic conditions, the 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.

8.4.2 Mat Foundation Settlement

For our settlement analysis, we estimated an average areal mat pressure of 1,000 psf. Based on this estimated loading, we estimate static settlements would be on the order of approximately ¼-inch at the mat edges and corners and up to approximately ½-inch near the center of the mat. Differential settlement from the center of mat to the edges due to static loads is estimated to be approximately ¼- to $^1/_3$-inch.

Static settlement estimates were developed based on estimated loads as the structural loads have not been provided to us at this time. We recommend we be retained to review the final layout and loading, and verify the settlement estimates above.

If foundations designed in accordance with the above recommendations are not capable of resisting such differential movement, settlement mitigation or an alternative foundation type may be required. Settlement mitigation could possibly include ground improvement to reduce settlement beneath the structures' footprints or the use of a deep foundation system. As mentioned, we recommend we be retained to review the final loading and further evaluate the settlement estimates above.

8.4.3 Mat Modulus of Soil Subgrade Reaction

We recommend using a variable modulus of subgrade reaction to provide a more accurate soil response and prediction of shears and moments in the mat. This will require at least one iteration between our soil model and the structural SAFE (or similar) analysis for the mat. As discussed above, we estimated an average areal mat pressure of 1,000 psf within the structure. Based on this pressure, we calculated preliminary modulus of subgrade reaction values for the mat foundation.

For preliminary SAFE runs, we recommend an initial variable modulus of subgrade reaction of 10 pounds per cubic inch (pci) for the central portion of the mat foundation and 20 pci within 5 feet of the mat edges. As discussed above, these moduli of soil subgrade reaction are intended for use in the first iteration of the structural SAFE analysis for the mat design. Once your initial run is complete, please forward a color graph of contact pressures for the mat (to scale) so that we can provide a revised plan with updated contours of equal modulus of subgrade reaction values. It should be noted that modulus values may change once updated contact pressures are determined.

8.4.4 Lateral Loading

Lateral loads may be resisted by friction between the bottom of mat foundation and the supporting subgrade, and also by passive pressures generated against deepened mat edges. An ultimate frictional resistance of 0.40 applied to the mat dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. The upper 12 inches of soil should be neglected when determining passive pressure capacity.

8.4.5 Mat Foundation Construction Considerations

Due to the presence of expansive soils, mat subgrade areas should be kept moist until concrete placement by regular sprinkling to prevent desiccation. If deep drying is allowed to occur, several days of moisture conditioning (flooding of the pads is not recommended) may be required to allow the moisture to re-penetrate the subgrade. If sever drying occurs, reworking and moisture conditioning of the pad may be required. Prior to placement of any vapor retarder and mat construction, the subgrade should be proof-rolled and visually observed by a Cornerstone representative to confirm stable subgrade conditions. The pad moisture should also be checked at least 24 hours prior to vapor barrier or mat reinforcement placement to confirm that the soil has a moisture content of at least 3 percent over optimum in the upper 12 inches.

8.4.6 Moisture Protection Considerations for Mat Foundations

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

- Place a 10-mil vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete mat; the vapor retarder should extend to within 12 to 18 inches from the mat edges and be sealed at all seams and penetrations in accordance with manufacturer’s recommendations and ASTM E 1643 requirements. For mats 12 inches thick or less, a 4-inch-thick capillary break, consisting of ½- to ¾-inch crushed rock with less than 5 percent passing the No. 200 sieve, should be placed below the vapor retarder and consolidated in place with vibratory equipment.

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

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

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

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

SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

9.1 INTERIOR SLABS-ON-GRADE

As the Plasticity Index (PI) of the surficial soils ranges up to 22, the proposed slabs-on-grade should be supported on at least 8 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. The NEF layer should be constructed over subgrade prepared in accordance with the recommendations in the “Earthwork” section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the “Interior Slabs Moisture Protection Considerations” section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and slab-on-grade or NEF construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned in accordance with the recommendations in the “Earthwork” section.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying soils. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

9.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

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

- Place a minimum 10-mil vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab; the vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer’s recommendations and ASTM E 1643 requirements. A 4-inch-thick capillary break, consisting of ½- to ¾-inch crushed rock with less than 5 percent passing the No. 200 sieve, should be placed below the vapor retarder and consolidated in place with vibratory equipment. The capillary break rock may be considered as the upper 4 inches of the non-expansive fill previously recommended.

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

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

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

- Where floor coverings are planned, all concrete surfaces should be properly cured.
- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869 and F710 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

9.3 EXTERIOR FLATWORK

9.3.1 Pedestrian Concrete 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 6 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

SECTION 10: VEHICULAR PAVEMENTS

10.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 10. The design R-value was chosen based on the previous results of the laboratory testing performed by Construction Testing & Engineering, Inc. on a surficial sample and engineering judgment.

Table 8: Asphalt Concrete Pavement Recommendations, Design R-value = 10

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.0	9.5
4.5	2.5	8.5	11.0
5.0	3.0	9.0	12.0
5.5	3.0	11.0	14.0
6.0	3.5	11.5	15.0
6.5	4.0	13.0	17.0

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

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

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

10.2 PORTLAND CEMENT CONCRETE

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

Table 9: PCC Pavement Recommendations, Design R-value = 10

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

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi, supporting the PCC on at least 6 inches of Class 2 aggregate base compacted as recommended in the "Earthwork" section, and laterally restraining the PCC with curbs or concrete shoulders. Adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Due to the expansive surficial soils present, we recommend that the construction and expansion joints be dowelled.

10.3 PAVEMENT CUTOFF

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

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

SECTION 11: RETAINING WALLS

11.1 STATIC LATERAL EARTH PRESSURES

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

Table 10: Recommended Lateral Earth Pressures

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

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

If adequate drainage cannot be provided behind the wall, an additional equivalent fluid pressure of 40 pcf should be added to the values above for both restrained and unrestrained walls for the portion of the wall that will not have drainage. Damp proofing or waterproofing of the walls may be considered where moisture penetration and/or efflorescence are not desired.

11.2 SEISMIC LATERAL EARTH PRESSURES

The 2013 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls greater than 6 feet in height. At this time, we are not aware of any retaining walls for the project and have not provided seismic earth pressures with this report. Seismic earth pressures can be provided at a later time for walls greater than about 6 feet in height, if requested by the project design team. In our opinion, seismic earth pressure are not warranted for the design of minor landscape retaining walls.

At this time, we are not aware of any retaining walls for the project. However, minor landscaping walls (i.e. walls 4 feet or less in height) may be proposed. In our opinion, design of these walls for seismic lateral earth pressures in addition to static earth pressures is not warranted.

11.3 WALL DRAINAGE

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

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

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

11.4 BACKFILL

We are not aware of any retaining walls. However, if walls are to be constructed, where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 95 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

11.5 FOUNDATIONS

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

SECTION 12: SWIMMING POOLS

12.1 EARTH PRESSURES

The swimming pool should be designed to resist at-rest earth pressures due to adjacent native or engineered backfill materials, hydrostatic pressures, as well as surcharge loads. Pool walls should be designed to resist an equivalent fluid pressure of 85 pcf (at-rest plus hydrostatic pressures) in addition to one-half of any surcharge load applied at the surface. We anticipate that pools less than 5 feet deep will be able to be constructed with vertical cut slopes. Deeper excavations should be temporarily sloped.

12.2 SWIMMING POOL DECKS

Concrete flatwork and/or pavers around swimming pools should be at least 4 inches thick and supported on at least 8 inches of non-expansive fill overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. The upper 4 inches of the non-expansive fill should consist of Class 2 aggregate base.

Proper surface drainage should be provided to divert surface water to closed pipe storm drainage facilities. Flexible bituminous caulking or equivalent should be applied between the pool and deck and deck expansion joints to reduce surface water penetration into the native expansive soils.

12.3 POOL SUB-DRAINAGE

The pool should have pressure relief valves incorporated into the pool bottom to relieve pressure buildup and potential heave during pool draining for maintenance. Consideration should be given to placing at least 4 inches of Caltrans Class 2 Permeable Material below the pool bottom to allow for pressure relief across the pool. Alternatively, ¾-inch clean, crushed rock may be placed provided a layer of filter fabric is placed beneath the crushed rock.

SECTION 13: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of RGJC South LLC specifically to support the design of the Hyatt House - Half Moon Bay project in Half Moon Bay, 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.

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

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

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

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

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

SECTION 14: REFERENCES

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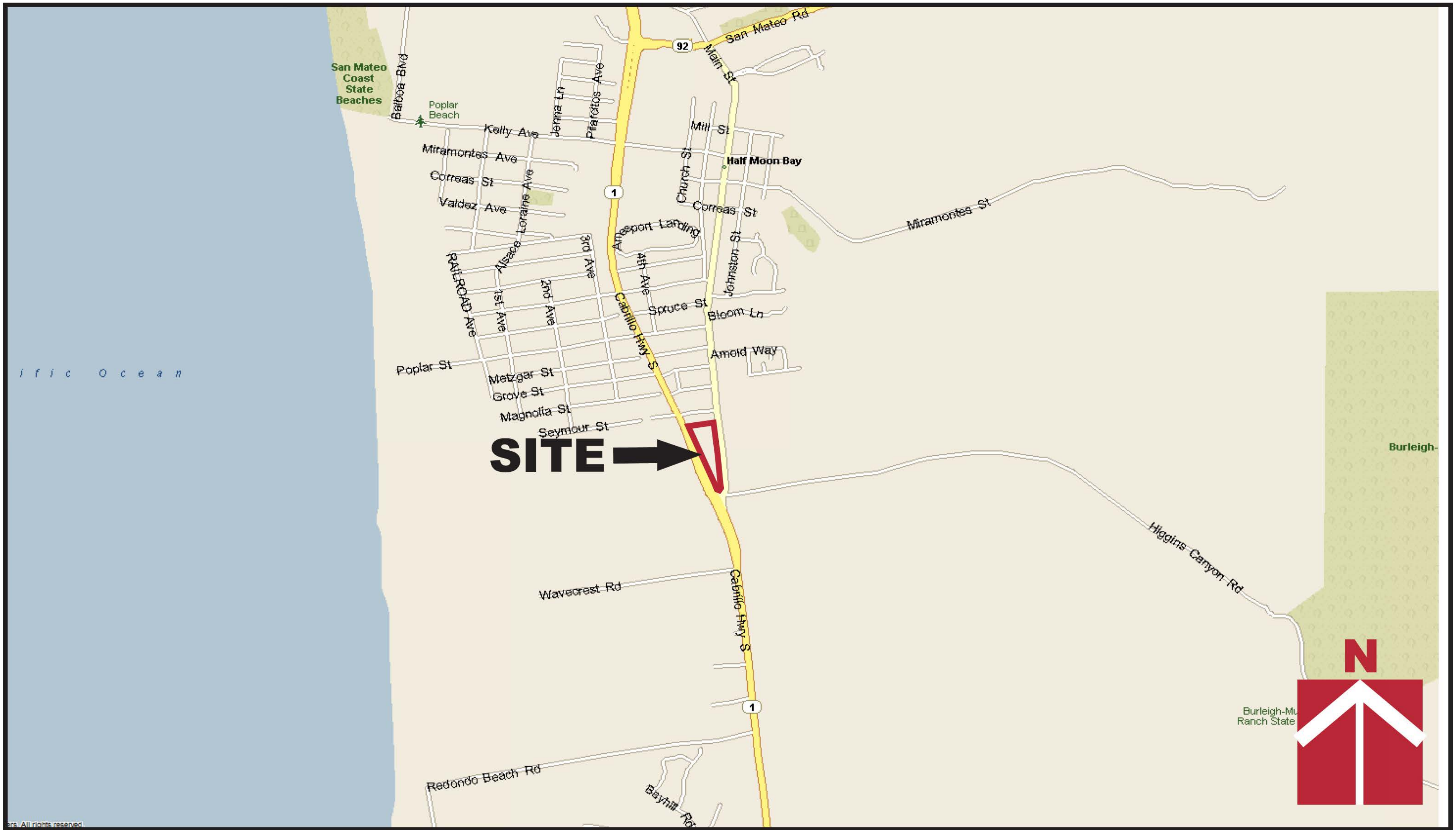


CORNERSTONE EARTH GROUP

Vicinity Map

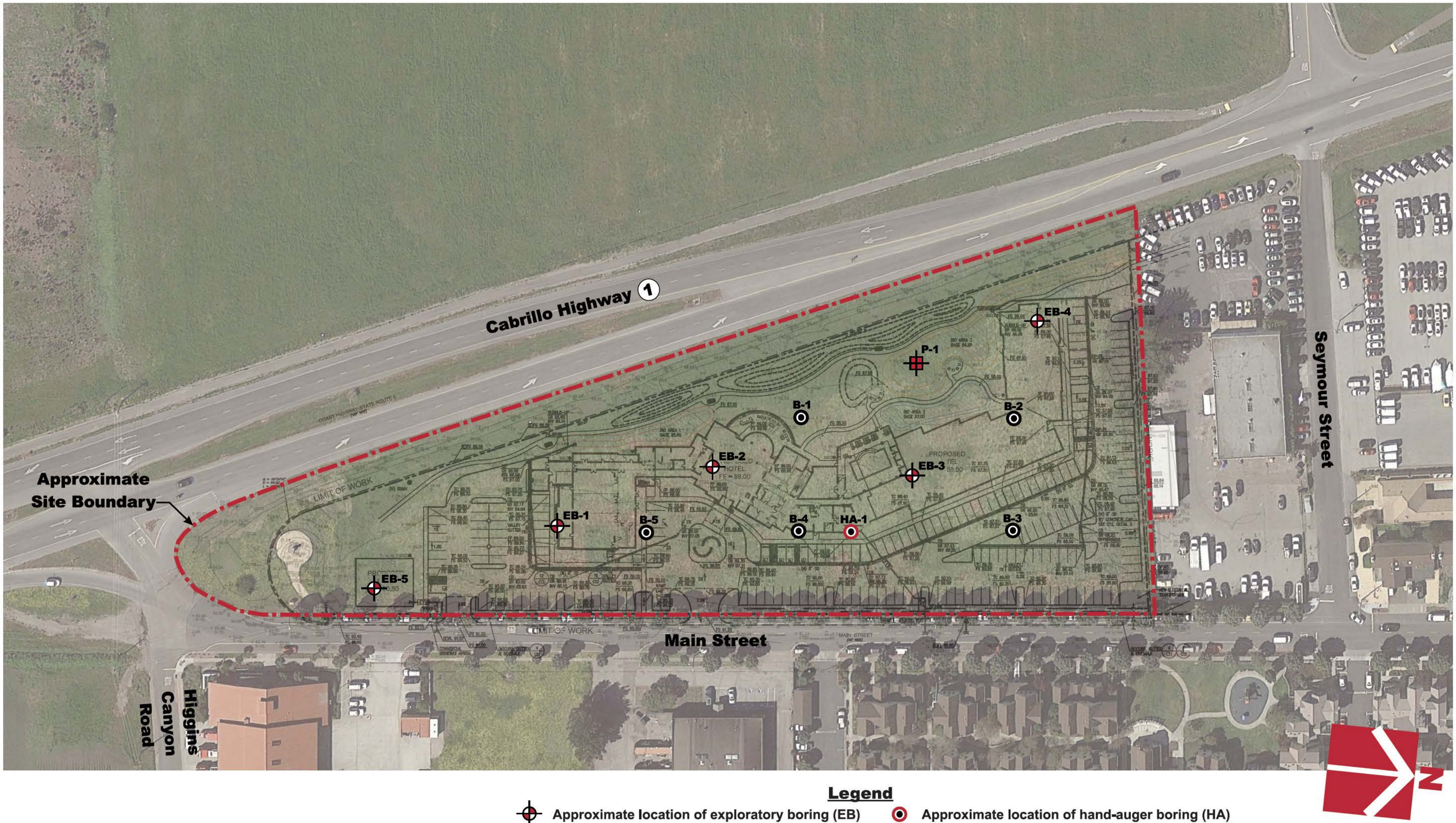
Hyatt House
Half Moon Bay, CA

Project Number
866-1-1

Figure Number
Figure 1

Date
January 2016

Drawn By
RRN



Legend

- ⊕ Approximate location of exploratory boring (EB)
- ⊞ Approximate location of percolation test (P)
- ◉ Approximate location of hand-auger boring (HA)
- ◉ Approximate location of previous boring (B) (Construction Testing & Engineering, Inc, 2013)

0 100 200

APPROXIMATE SCALE (FEET)

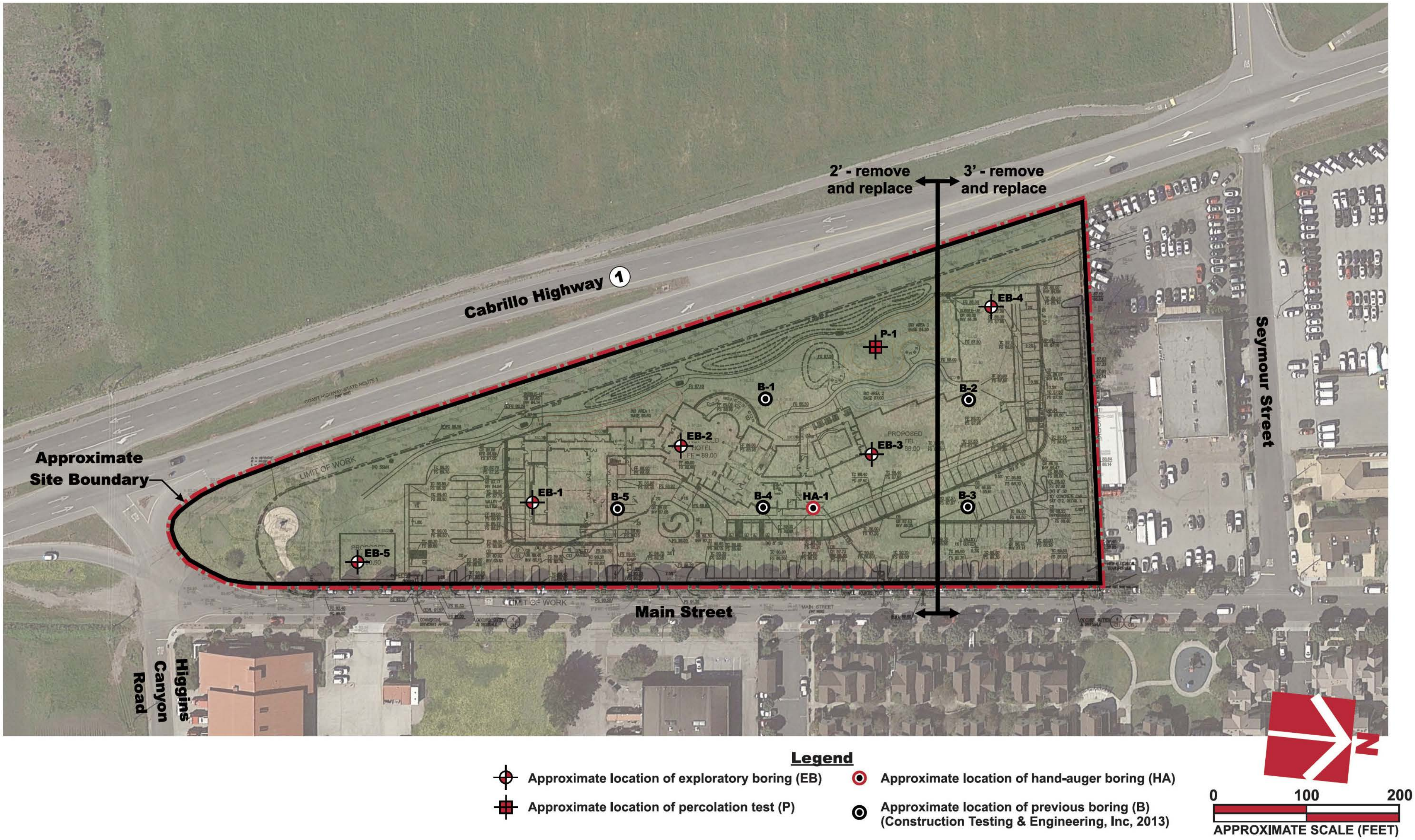
CORNERSTONE
EARTH GROUP

Site Plan

Hyatt House
Half Moon Bay, CA

Project Number	866-1-1
Figure Number	Figure 2A
Date	January 2016
Drawn By	RRN

Base by Google Earth, dated 3/28/2015
Overlay: Proposed Development



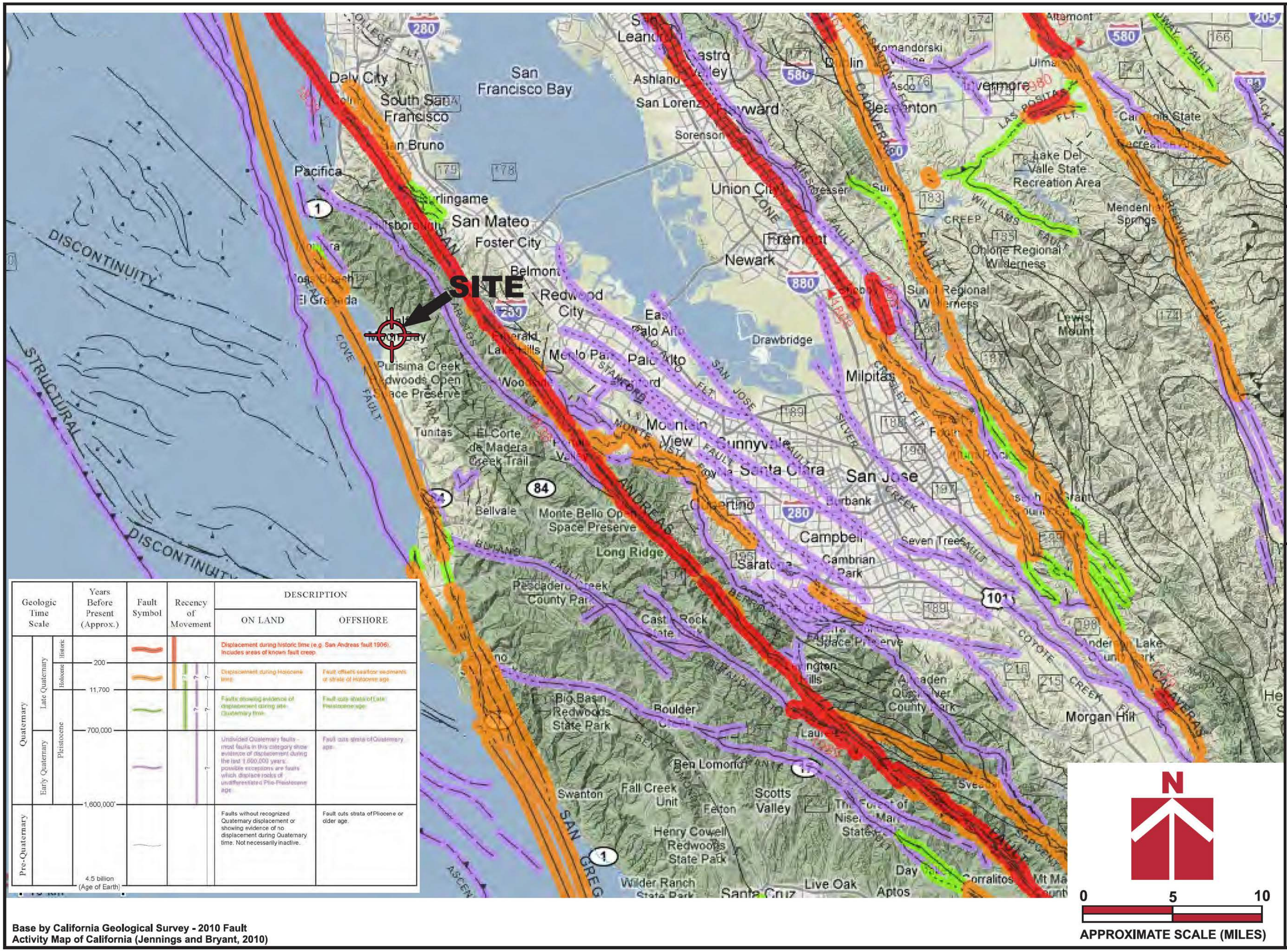
Base by Google Earth, dated 3/28/2015
Overlay: Proposed Development

CORNERSTONE
EARTH GROUP

Site Plan

Hyatt House
Half Moon Bay, CA

Project Number	866-1-1
Figure Number	Figure 2B
Date	January 2016
Drawn By	RRN



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

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

Regional Fault Map

Hyatt House
Half Moon Bay, CA

CORNERSTONE
EARTH GROUP

Project Number
866-1-1

Figure Number
Figure 3

Date
January 2016

Drawn By
RRN

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using track-mounted, hollow-stem auger drilling equipment. Five 8-inch-diameter exploratory borings were drilled on January 12, 2016 to depths of 15 to 45 feet. The approximate locations of exploratory borings are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

Boring locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. Boring elevations were based on interpolation of plan contours. The locations and elevations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Relatively undisturbed samples were also obtained with 2.875-inch I.D. Shelby Tube sampler which were hydraulically pushed. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

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

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

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

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

OTHER MATERIAL SYMBOLS

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

SAMPLER TYPES

☒ SPT

Modified California (2.5" I.D.)

Rock Core

■ Shelby Tube

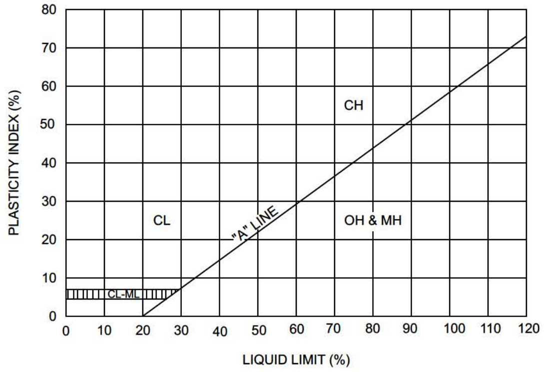
○ No Recovery

Grab Sample

ADDITIONAL TESTS

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

PLASTICITY CHART



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

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

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

CORNERSTONE
EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number
A-1

BORING NUMBER EB-1

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME Hyatt House
PROJECT NUMBER 866-1-1
PROJECT LOCATION Half Moon Bay, CA

DATE STARTED 1/12/16 **DATE COMPLETED** 1/12/16
GROUND ELEVATION 89.4 FT +/- **BORING DEPTH** 40 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.451660° **LONGITUDE** -122.429884°
DRILLING METHOD CME 55 Track Rig, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY FLL
▽ **AT TIME OF DRILLING** 30 ft.
NOTES
▼ **AT END OF DRILLING** 30 ft.

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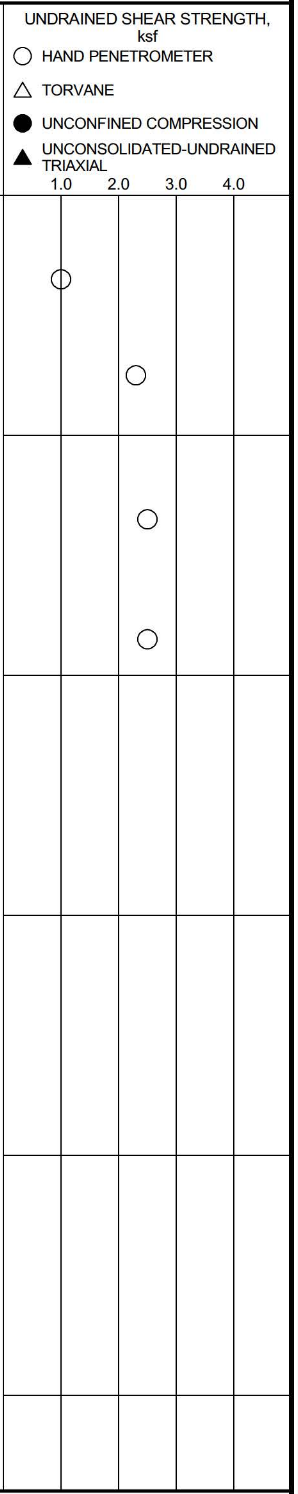
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
89.4	0	**Lean Clay (CL)** medium stiff, moist, dark gray, some fine sand, moderate plasticity Liquid Limit = 39, Plastic Limit = 17	10	MC-1B	99	22	22	
86.9		**Lean Clay (CL)** very stiff, moist, gray and brown mottled, some fine sand, moderate plasticity	19	MC-2B	104	19		
	5		20	MC-3B	109	18		
			23	MC-4B	109	18		
	10							
76.4		**Clayey Sand with Gravel (SC)** dense, moist, gray and reddish brown mottled, fine to medium sand, fine to coarse subangular to subrounded gravel	59	MC-5B	114	16		
	15							
			39	SPT				
	20							
68.4		**Sandy Lean Clay (CL)** very stiff, moist, gray, fine to medium sand, some fine gravel, low plasticity						
		Liquid Limit = 28, Plastic Limit = 19	18	SPT-7		20	9	54
	25							
63.4								



Continued Next Page

CORNERSTONE EARTH GROUP

BORING NUMBER EB-1

PAGE 2 OF 2

PROJECT NAME Hyatt House
PROJECT NUMBER 866-1-1
PROJECT LOCATION Half Moon Bay, CA

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ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
63.4		**Sandy Lean Clay (CL)** very stiff, moist, gray, fine to medium sand, some fine gravel, low plasticity	39	MC-8B	102	21			○
59.4	▼ 30	**Silty Sand (SM)** medium dense to dense, wet, gray, fine to coarse sand, some fine subangular to subrounded gravel	32	SPT-9		18			
	35								
52.4		**Lean Clay with Sand (CL)** very stifff, moist, gray, fine sand, moderate plasticity	15	SPT-10		20			○
49.4	40	Bottom of Boring at 40.0 feet.							
	45								
	50								
	55								

BORING NUMBER EB-2

PAGE 1 OF 1

CORNERSTONE
EARTH GROUP

PROJECT NAME Hyatt House
PROJECT NUMBER 866-1-1
PROJECT LOCATION Half Moon Bay, CA
DATE STARTED 1/12/16 **DATE COMPLETED** 1/12/16
GROUND ELEVATION 89.8 FT +/- **BORING DEPTH** 25 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.452076° **LONGITUDE** -122.430129°
DRILLING METHOD CME 55 Track Rig, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY FLL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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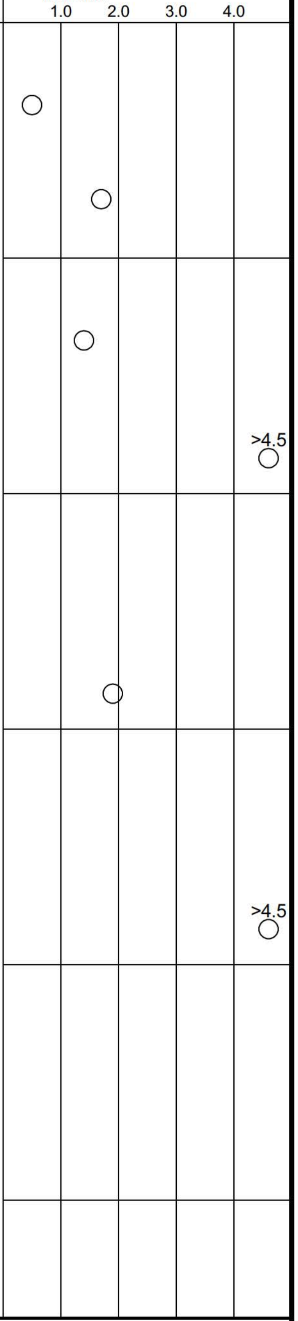
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
89.8	0	**Lean Clay (CL)** medium stiff, moist, dark gray, some fine sand, moderate plasticity						
			9	MC-1B	101	19		
87.6		**Clayey Sand (SC)** loose, moist, gray brown, fine sand						
86.3		**Lean Clay with Sand (CL)** stiff, moist, gray with brown mottles, fine to medium sand, low to moderate plasticity	15	MC-2B	106	19		
	5							
			12	MC-3A	106	18		
82.8		**Lean Clay (CL)** hard, moist, gray and brown mottled, some fine sand, moderate plasticity						
			38	MC-4B	109	17		
	10							
77.8		**Lean Clay with Sand (CL)** very stiff, moist, gray with brown mottles, fine sand, low plasticity						
			19	MC-5B	105	19		
	15							
72.8		**Sandy Lean Clay (CL)** hard, moist, gray with reddish brown mottles, fine to medium sand, some fine subangular gravel, low plasticity						
			24	MC-6B	103	20		
	20							
66.1		**Clayey Sand with Gravel (SC)** dense, moist, brown, fine to medium sand, fine to coarse subangular to subrounded gravel	38	SPT-7		5		
64.8	25	Bottom of Boring at 25.0 feet.						

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



BORING NUMBER EB-3

CORNERSTONE EARTH GROUP

PROJECT NAME Hyatt House
PROJECT NUMBER 866-1-1
PROJECT LOCATION Half Moon Bay, CA

DATE STARTED 1/12/16 **DATE COMPLETED** 1/12/16
GROUND ELEVATION 86.5 FT +/- **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.452637° **LONGITUDE** -122.430166°
DRILLING METHOD CME 55 Track Rig, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY FLL
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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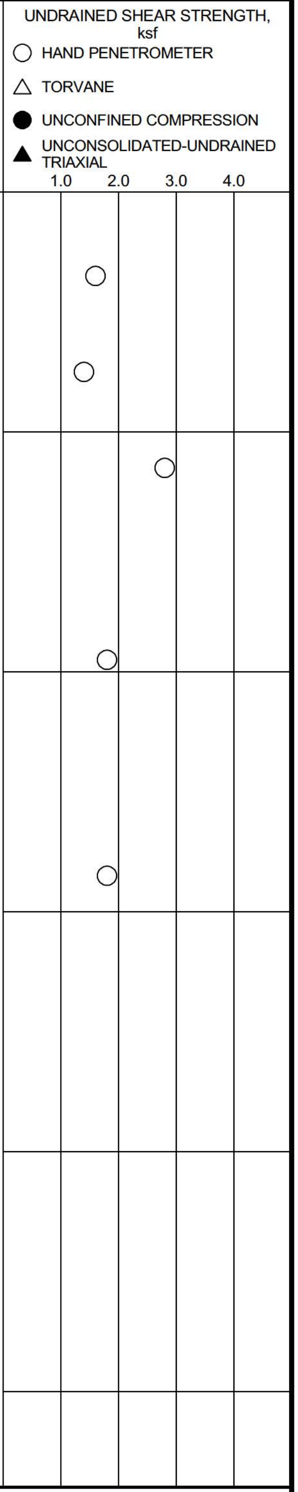
ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
86.5	0	▨	**Lean Clay (CL)** medium stiff, moist, dark gray, some fine sand, moderate plasticity						
85.0		▨	**Lean Clay (CL)** stiff, moist, gray with brown mottles, some fine sand, moderate plasticity	18	MC-1B	103	21		
				20	MC-2B	102	21		
	5								
			very stiff	21	MC-3B	97	25		
					ST-4	109	18		
	10			14	MC-5B	99	25		
				22	MC-6B	95	27		
	15								
68.0		▨	**Clayey Sand with Gravel (SC)** medium dense, moist, brown, fine to medium sand, fine subangular to subrounded gravel	29	MC-7B	95	24		17
66.5	20		Bottom of Boring at 20.0 feet.						
	25								



CORNERSTONE
EARTH GROUP

BORING NUMBER EB-4

PAGE 1 OF 2

PROJECT NAME Hyatt House

PROJECT NUMBER 866-1-1

PROJECT LOCATION Half Moon Bay, CA

DATE STARTED 1/12/16 **DATE COMPLETED** 1/12/16

GROUND ELEVATION 85.2 FT +/- **BORING DEPTH** 35 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE 37.452942° **LONGITUDE** -122.430758°

DRILLING METHOD CME 55 Track Rig, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY FLL

▽ **AT TIME OF DRILLING** 15 ft.

NOTES

▼ **AT END OF DRILLING** 15 ft.

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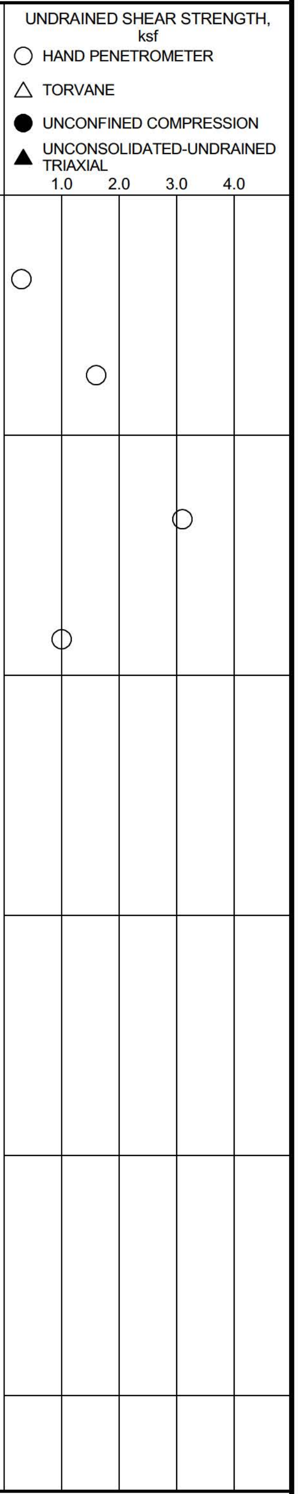
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
85.2	0	**Lean Clay (CL)** soft, moist, dark gray, trace fine sand, moderate plasticity						
			8	MC-1B	96	25		
81.7		**Lean Clay (CL)** stiff, moist, brown, some fine sand, moderate plasticity	21	MC-2B	101	21		
	5							
		very stiff	20	MC-3B	97	25		
76.2		**Lean Clay with Sand (CL)** stiff, moist, brown, fine sand, moderate plasticity	13	MC-4B	104	22		
	10							
73.7		**Clayey Sand with Gravel (SC)** medium dense, moist, reddish brown, fine to coarse sand, fine to coarse subangular to subrounded gravel						
			29	MC-5B	103	22		
	▼ 15							
		becomes dense	33	SPT-6		25		
	20							
63.2		**Clayey Sand (SC)** dense, wet, brown, fine to coarse sand, some fine subangular to subrounded gravel						
			32	SPT				
	25							
59.2		*Continued Next Page*						



CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 3/15/16 06:52 - P:\DRAFTING\GINT FILES\866-1-1 HYATT HOUSE.GPJ

BORING NUMBER EB-4

PAGE 2 OF 2

PROJECT NAME Hyatt House

PROJECT NUMBER 866-1-1

PROJECT LOCATION Half Moon Bay, CA

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ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
59.2		▨	**Clayey Sand (SC)** dense, wet, brown, fine to coarse sand, some fine subangular to subrounded gravel							
		▨	medium dense	27	☒ SPT-8		20			
	30	▨								
		▨	dark brown mottles, increasing fine subangular to subrounded gravel	33	☒ SPT					
50.2	35		Bottom of Boring at 35.0 feet.							
	40									
	45									
	50									
	55									

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-5

PAGE 1 OF 1

PROJECT NAME Hyatt House
PROJECT NUMBER 866-1-1
PROJECT LOCATION Half Moon Bay, CA
DATE STARTED 1/12/16 **DATE COMPLETED** 1/12/16
GROUND ELEVATION 89.5 FT +/- **BORING DEPTH** 15 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.451170° **LONGITUDE** -122.429583°
DRILLING METHOD CME 55 Track Rig, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY FLL
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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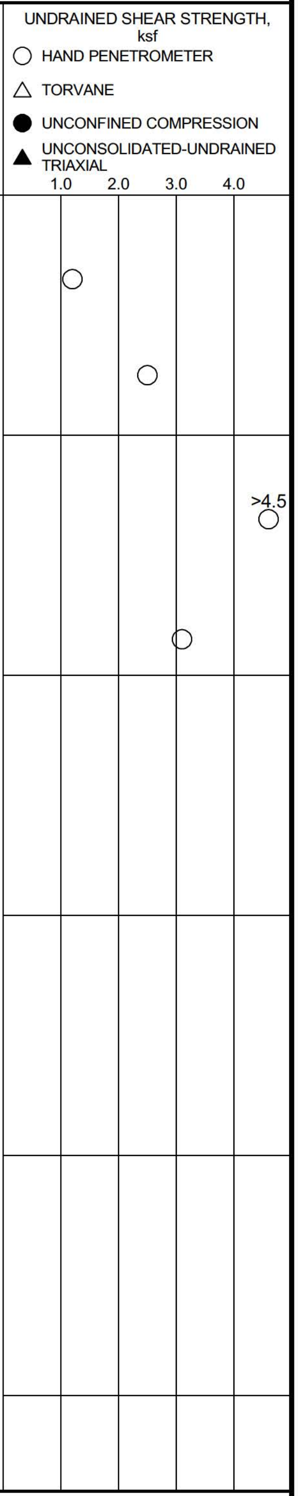
ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
89.5	0		**Lean Clay (CL)** stiff, moist, dark gray, some fine sand, moderate plasticity	12	MC-1B	96	22			
87.0			**Lean Clay (CL)** very stiff, moist, gray and brown mottled, some fine sand, moderate plasticity	22	MC-2B	98	21			
	5		becomes hard	33	MC-3B	102	18			>4.5
				26	MC-4B	99	21			
80.0	10		**Clayey Sand with Gravel (SC)** medium dense, moist, gray with reddish brown mottles, fine to medium sand, fine to coarse subangular to subrounded gravel	44	MC-5B	106	16			
74.5	15		Bottom of Boring at 15.0 feet.							
	20									
	25									



CORNERSTONE
EARTH GROUP

BORING NUMBER P-1

PAGE 1 OF 1

PROJECT NAME Hyatt House
PROJECT NUMBER 866-1-1
PROJECT LOCATION Half Moon Bay, CA
DATE STARTED 1/12/16 **DATE COMPLETED** 1/12/16
GROUND ELEVATION 89.5 FT +/- **BORING DEPTH** 5.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.452618° **LONGITUDE** -122.430558°
DRILLING METHOD CME 55 Track Rig, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY FLL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf 1.0 2.0 3.0 4.0
89.5	0	▨	**Lean Clay (CL)** soft, moist, dark gray, some fine sand, moderate plasticity							
86.5		▨	**Lean Clay with Sand (CL)** very stiff, moist, gray with brown mottles, fine sand, moderate plasticity							
	5	▨		22	⧗ MC					○
84.0			Bottom of Boring at 5.5 feet.							
	10									
	15									
	20									
	25									

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 34 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 28 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on 2 samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

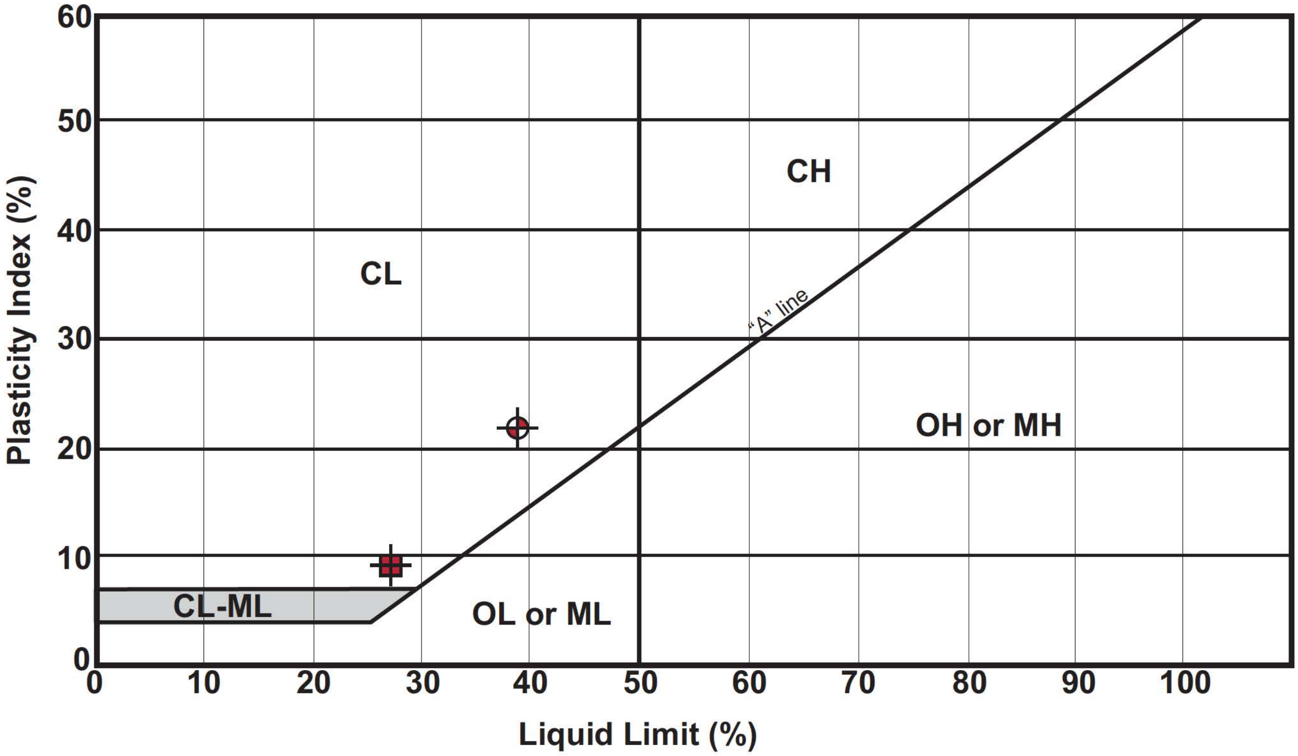
Plasticity Index: Two Plasticity Index determinations (ASTM D4318) were performed on samples of the subsurface soils to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification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

Consolidation: One consolidation test (ASTM D2435) was performed on a relatively undisturbed sample of the subsurface clayey soils to assist in evaluating the compressibility property of this soil. Results of the consolidation test are presented graphically in this appendix.

Corrosion: Two samples were each tested for pH (ASTM G51), resistivity (ASTM G57), chloride (ASTM D4327), and sulfate (ASTM D4327). Results of these tests are attached in this appendix.

Organic Content: Two samples of the surficial soils were tested for their organic content (ASTM D2974-00). Results of these tests are attached in this appendix.

Plasticity Index (ASTM D4318) Testing Summary



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
⊕	EB-1	2.0	22	39	17	22	—	Lean Clay (CL)
⊞	EB-1	23.5	20	28	19	9	54	Sandy Lean Clay (CL)

CORNERSTONE
EARTH GROUP

Plasticity Index Testing Summary

Hyatt House
Half Moon Bay, CA

Project Number
866-1-1

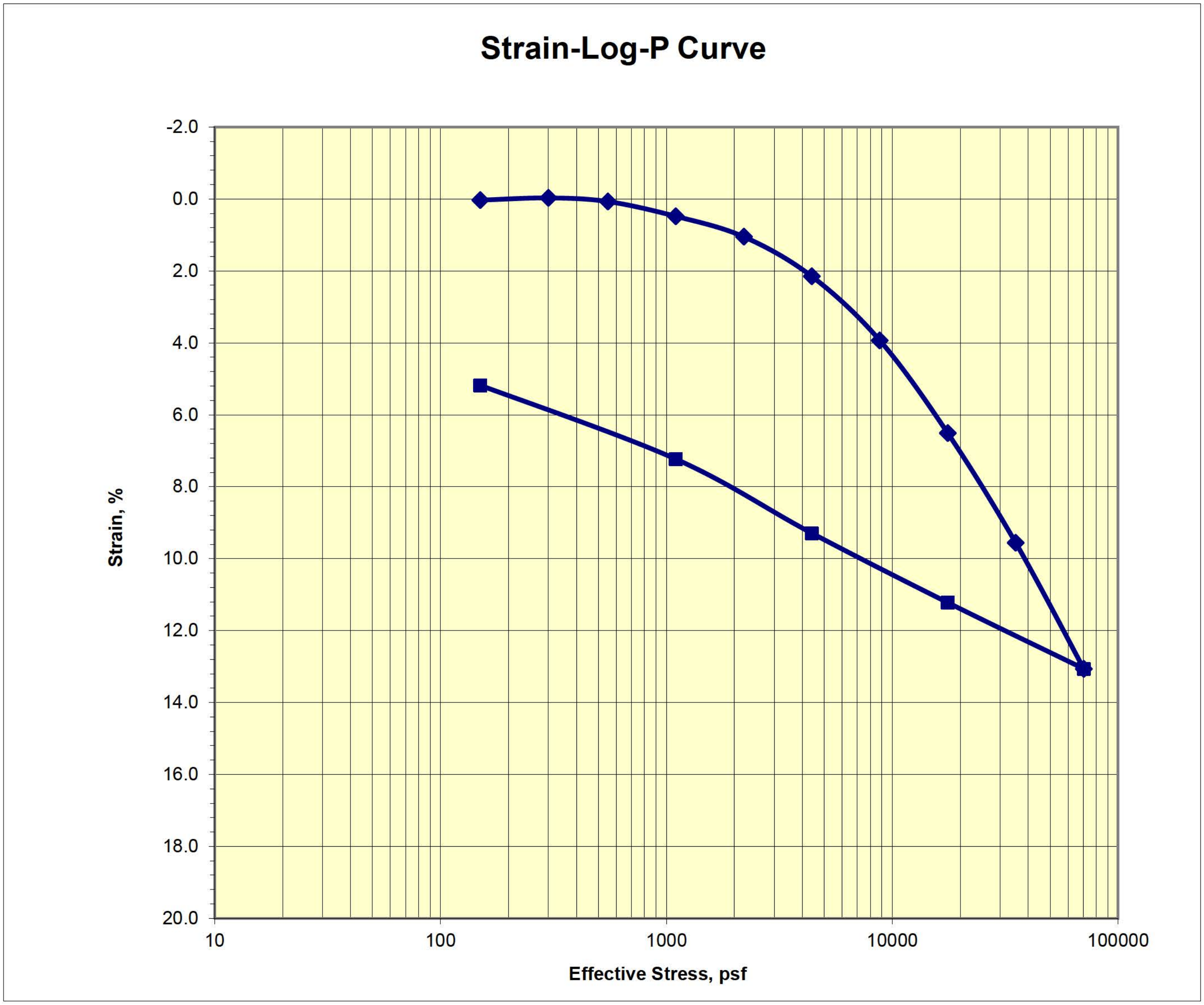
Figure Number
Figure B1

Date
January 2016

Drawn By
FLL

Consolidation Test
ASTM D2435

Job No.:	640-933	**Boring:**	EB-3	**Run By:**	MD
Client:	Cornerstone Earth Group	**Sample:**	4	**Reduced:**	PJ
Project:	Hyatt House - 866-1-1	**Depth, ft.:**	7.0(Tip-3")	**Checked:**	PJ/DC
Soil Type:	Yellowish Brown Sandy CLAY			**Date:**	1/28/2016



Assumed Gs 2.75	Initial	Final
Moisture %:	18.2	18.0
Dry Density, pcf:	108.8	114.9
Void Ratio:	0.577	0.494
% Saturation:	86.6	100.0

Remarks:

Corrosivity Tests Summary

CTL #: 640-933 **Date:** 1/19/2016 **Tested By:** PJ **Checked:** PJ
Client: Cornerstone Earth Group **Project:** Hyatt House **Proj. No:** 866-1-1
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	3.5	-	-	2,236	30	46	0.0046	6.6	-	-	-	20.6	Dark Yellowish Brown Sandy CLAY
EB-4	3A	6.5	-	-	2,454	9	25	0.0025	6.8	-	-	-	22.4	Olive Sandy CLAY

COOPER
TESTING LABORATORY

Organic Content Test
ASTM D 2974-00 (Method C - 440 ^{o}C)

CTL JOB NO.: 640-933
CLIENT : Cornerstone Earth Group
PROJECT: Hyatt House
PROJECT NO.: 866-1-1
DATE: 1/19/2016
BY: RU

Boring :	SS-1	SS-3						
Sample :								
Depth (ft.):	6"	6"						
Visual Description:	Black CLAY w/ organics (Silty)	Black CLAY w/ organics (Silty)						
Dish No.								
Dish wt., gm	80.97	77.23						
Soil, Org, Dish & H_2O, gm	291.70	230.53						
Oven Dry wt (105°C), gm	242.21	197.68						
Furnace Dry wt. (440°C), gm	230.43	189.67						
Moisture Content, % of Oven Dry Mass	30.7	27.3						
Organic Matter, %	7.3	6.7						

Note: ASTM provides no guidelines for including information about the organic content of a sample in the description when the wet/dry liquid limit data is not available. CTL developed the following guidelines to fill this gap:

0-5%:	The organics are either not mentioned or mentioned as being "trace".
5-15%:	The soil is considered as inorganic and is classified, as per ASTM 2487, with "with organics" included in the description.
15-50%:	The soil is considered as organic and is described, per ASTM 2487.
> 50%:	The soil is described as "Peat".

APPENDIX C: PREVIOUS LAB DATA FROM CONSTRUCTION TESTING & ENGINEERING, INC.

APPENDIX B

FIELD EXPLORATION METHODS AND BORINGS LOGS

Soil Boring Methods

Relatively "Undisturbed" Soil Samples

Relatively "undisturbed" soil samples were collected using a modified California-drive sampler (2.4-inch inside diameter, 3-inch outside diameter) lined with sample rings. Drive sampling was conducted in general accordance with ASTM D-3550. The steel sampler was driven into the bottom of the borehole with successive drops of a 140-pound weight falling 30-inches. Blow counts (N) required for sampler penetration are shown on the boring logs in the column "Blows/Foot." The soil was retained in brass rings (2.4 inches in diameter, 1.00 inch in height). The samples were retained and carefully sealed in waterproof plastic containers for shipment to the Construction Testing & Engineering ("CTE") geotechnical laboratory.

Disturbed Soil Sampling

Bulk soil samples were collected for laboratory analysis using two methods. Standard Penetration Tests (SPT) were performed according to ASTM D-1586 at selected depths in the borings using a standard (1.4-inches inside diameter, 2-inches outside diameter) split-barrel sampler. The steel sampler was driven into the bottom of the borehole with successive drops of a 140-pound weight falling 30-inches. Blow counts (N) required for sampler penetration are shown on the boring logs in the column "Blows/Foot." Samples collected in this manner were placed in sealed plastic bags. Bulk soil samples of the drill cuttings were also collected in large plastic bags. Disturbed soil samples were returned to the CTE geotechnical laboratory for analysis.

CONSTRUCTION TESTING & ENGINEERING, INC.
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DEFINITION OF TERMS

PRIMARY DIVISIONS			SYMBOLS	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	**GRAVELS** MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS < 5% FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS OR GRAVEL SAND MIXTURES, LITTLE OF NO FINES
		GRAVELS WITH FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES, NON-PLASTIC FINES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES, PLASTIC FINES
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS < 5% FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES	SM	SILTY SANDS, SAND-SILT MIXTURES, NON-PLASTIC FINES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES, PLASTIC FINES
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	**SILTS AND CLAYS** LIQUID LIMIT IS LESS THAN 50		ML	INORGANIC SILTS, VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, SLIGHTLY PLASTIC CLAYEY SILTS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY, SANDY, SILTS OR LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTY CLAYS
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

GRAIN SIZES

BOULDERS	COBBLES	GRAVEL		SAND			SILTS AND CLAYS
		COARSE	FINE	COARSE	MEDIUM	FINE	

12" 3" 3/4" 4 10 40 200

CLEAR SQUARE SIEVE OPENING U.S. STANDARD SIEVE SIZE

ADDITIONAL TESTS

(OTHER THAN TEST PIT AND BORING LOG COLUMN HEADINGS)

MAX- Maximum Dry Density
GS- Grain Size Distribution
SE- Sand Equivalent
EI- Expansion Index
CHM- Sulfate and Chloride Content , pH, Resistivity
COR - Corrosivity

PM- Permeability
SG- Specific Gravity
HA- Hydrometer Analysis
AL- Atterberg Limits
RV- R-Value
CN- Consolidation

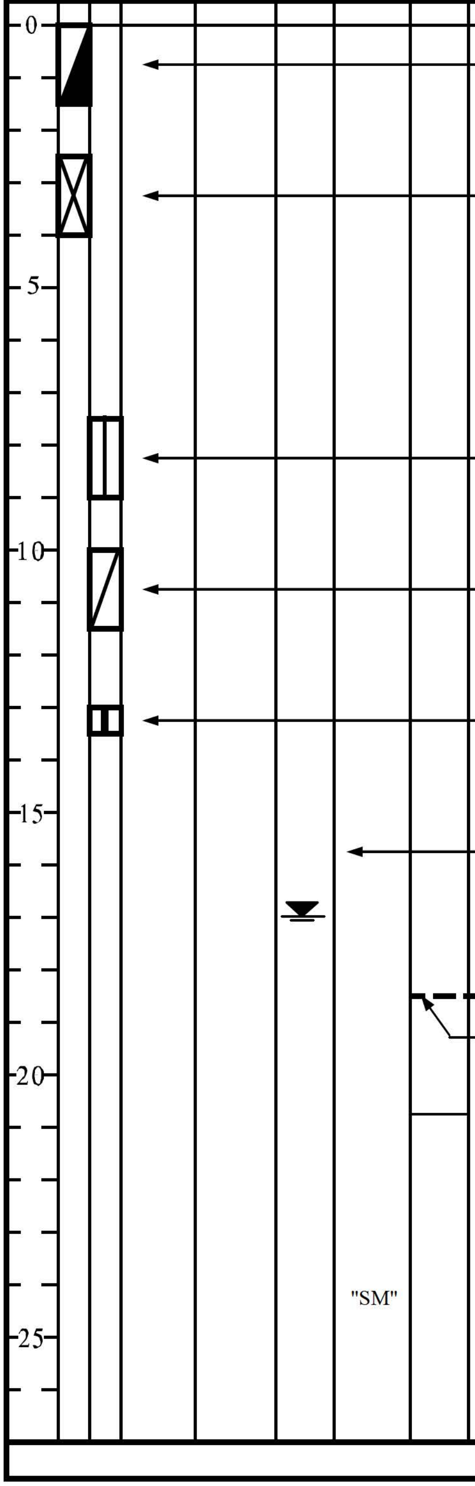
PP- Pocket Penetrometer
WA- Wash Analysis
DS- Direct Shear
UC- Unconfined Compression
MD- Moisture/Density
M- Moisture
SC- Swell Compression
OI- Organic Impurities

FIGURE: BL1

PROJECT:
CTE JOB NO:
LOGGED BY:
DRILLER:
DRILL METHOD:
SAMPLE METHOD:
SHEET: of
DRILLING DATE:
ELEVATION:

BORING LEGEND

Depth (Feet)	Bulk Sample	Driven Type	Blows/Foot	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests



Block or Chunk Sample

Bulk Sample

Standard Penetration Test

Modified Split-Barrel Drive Sampler (Cal Sampler)

Thin Walled Army Corp. of Engineers Sample

Groundwater Table

Soil Type or Classification Change

? —— ? —— ? —— ? —— ? —— ? —— ? –

Formation Change [(Approximate boundaries queried (?)]

Quotes are placed around classifications where the soils exist in situ as bedrock

FIGURE: BL2

PROJECT:	James Ford Dealership, Half Moon Bay	DRILLER:	All Star Drilling	SHEET:	1 of 1
CTE JOB NO:	20-2375G	DRILL METHOD:	4" Flight Auger	DRILLING DATE:	1/23/2013
LOGGED BY:	TAK	SAMPLE METHOD:	SPT	ELEVATION:	EGS

BORING: B-1

Depth (Feet)	Bulk Sample	Driven Sample Type	Blows/6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0						CL	▨	Stiff, damp, black organic medium plastic CLAY	
		▯	2 2 2			CL	▨	Soft, damp-moist, brown, medium plasticity, CLAY w/ angular fine gravel	PP=1.0-1.5 tsf
					▼				
5		▯	2 3 4			CL	▨	Medium stiff, brown, moist-wet, medium plastic CLAY with fine gravel	PP=1.5-2.0 tsf
10		▯	4 8 14			CL	▨	Very stiff, dry, brown & orange brown fine gravelly CLAY	PP=3.5-4.5 tsf
15		▯	5 7 8			CL/CH	▨	Stiff, damp, brown & orange brown highly plastic CLAY w/fine gravel	PP=2.5-3.0 tsf
								Total depth = 16.5 feet below grade. Groundwater encountered 4.0 feet depth while augering. Boring backfilled 1/23/13.	
20									
25									

PROJECT:	James Ford Dealership, Half Moon Bay	DRILLER:	All Star Drilling	SHEET:	1 of 1
CTE JOB NO:	20-2375G	DRILL METHOD:	4" Flight Auger	DRILLING DATE:	1/23/2013
LOGGED BY:	TAK	SAMPLE METHOD:	SPT	ELEVATION:	EGS

BORING: B-2

Depth (Feet)	Bulk Sample	Driven Type	Blows/6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0	☒	☐	3 5 7			CL	▨	Medium stiff, damp, black organic medium plastic CLAY	PP=1.5 tsf
						CL	▨	Stiff, damp, brown, medium plastic CLAY w/ angular fine gravel	
					▼		▨		
5		☐	3 4 6			CL	▨	As Above	PP=2.0-2.5 tsf
10		☐	2 3 4			CL	▨	Medium stiff, damp, brown, medium plastic CLAY with fine gravel	PP=1.5-2.0 tsf
								Total depth = 11.5 feet below grade. Groundwater encountered 4.0 feet depth while augering. Boring backfilled 1/23/13.	
15									
20									
25									

PROJECT:	James Ford Dealership, Half Moon Bay	DRILLER:	All Star Drilling	SHEET:	1 of 1
CTE JOB NO:	20-2375G	DRILL METHOD:	4" Flight Auger	DRILLING DATE:	1/23/2013
LOGGED BY:	TAK	SAMPLE METHOD:	SPT	ELEVATION:	EGS

BORING: B-3

Depth (Feet)	Bulk Sample	Driven Type	Blows/6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0							▨	Medium stiff, damp, black organic medium plastic CLAY	
		☐	2 3 3			CL	▨	Stiff, moist, dark brown, fine-coarse sandy, fine gravelly CLAY	PP=1.5 tsf
						CL	▨	Stiff, damp, brown, highly plastic CLAY with fine gravel	
5		☐	2 7 6			CL	▨		PP=2.0 tsf
								Total depth = 6.5 feet below grade. Groundwater encountered at 4.0 feet depth. Boring backfilled with cuttings 1/23/13	
10									
15									
20									
25									

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PROJECT:	James Ford Dealership, Half Moon Bay	DRILLER:	All Star Drilling	SHEET:	1 of 1
CTE JOB NO:	20-2375G	DRILL METHOD:	4" Flight Auger	DRILLING DATE:	1/23/2013
LOGGED BY:	TAK	SAMPLE METHOD:	SPT	ELEVATION:	EGS

BORING: B-4

Depth (Feet)	Bulk Sample	Driven Type	Blows/6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0							▨	Medium stiff, damp, black organic medium plastic CLAY	
						CL	▨		
5		▯	2 5 6			CL	▨	Stiff, damp, brown, highly plastic CLAY with fine gravel	
10		▯	5 9 13			CL	▨	Very stiff, damp, brown & orange brown highly plastic CLAY with fine gravel	
								Total depth = 11.5 feet below grade. Groundwater encountered at 4.0 feet depth. Boring backfilled with cuttings 1/23/13	
15									
20									
25									

PROJECT:	James Ford Dealership, Half Moon Bay	DRILLER:	All Star Drilling	SHEET:	1 of 1
CTE JOB NO:	20-2375G	DRILL METHOD:	4" Flight Auger	DRILLING DATE:	1/23/2013
LOGGED BY:	TAK	SAMPLE METHOD:	Bulk	ELEVATION:	EGS

BORING: B-5

Depth (Feet)	Bulk Sample	Driven Type	Blows/6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0	☒					CL	▨	Medium stiff, damp, black organic medium plastic CLAY	
						CL	▨	Stiff, damp, brown, CLAY with fine gravel	
5								Total depth = 5.0 feet below grade. Groundwater encountered at 4.0 feet depth. Boring backfilled with cuttings 1/23/13	
10									
15									
20									
25									

APPENDIX C

LABORATORY METHODS AND RESULTS

APPENDIX C
LABORATORY METHODS AND RESULTS

Laboratory tests were performed on representative soil samples to evaluate their engineering properties. Tests were performed following test methods of the American Society for Testing Materials or other accepted standards. The following presents a brief description of the various test methods used. Laboratory results are presented in the following section of this Appendix.

Classification

Soils were classified visually according to the Unified Soil Classification System. Visual classifications were supplemented by laboratory testing of selected samples according to ASTM D2487.

Particle-Size Analysis

Particle-Size Analyses were performed on selected representative samples in accordance with ASTM D1140 and C136.

Expansion Index

Expansion testing was performed on selected samples of the onsite soils according to ASTM D4829.

In-Place Moisture/Density

The in-place moisture content and dry unit weight of selected samples were determined using relatively undisturbed soil samples.

Atterberg Limits

The procedure of ASTM D4318 was used to measure the liquid limit, plastic limit and plasticity index of representative samples.

Modified Proctor

Laboratory compaction tests were performed according to ASTM D1557. A mechanically operated rammer was used during the compaction process.

Resistance "R"-Value

The resistance "R"-value was determined by the California Materials Method No. 301 for representative subbase soils. Samples were prepared and exudation pressure and "R"-value determined. The graphically determined "R"- value at exudation pressure of 300 psi is the value used for pavement section calculation.

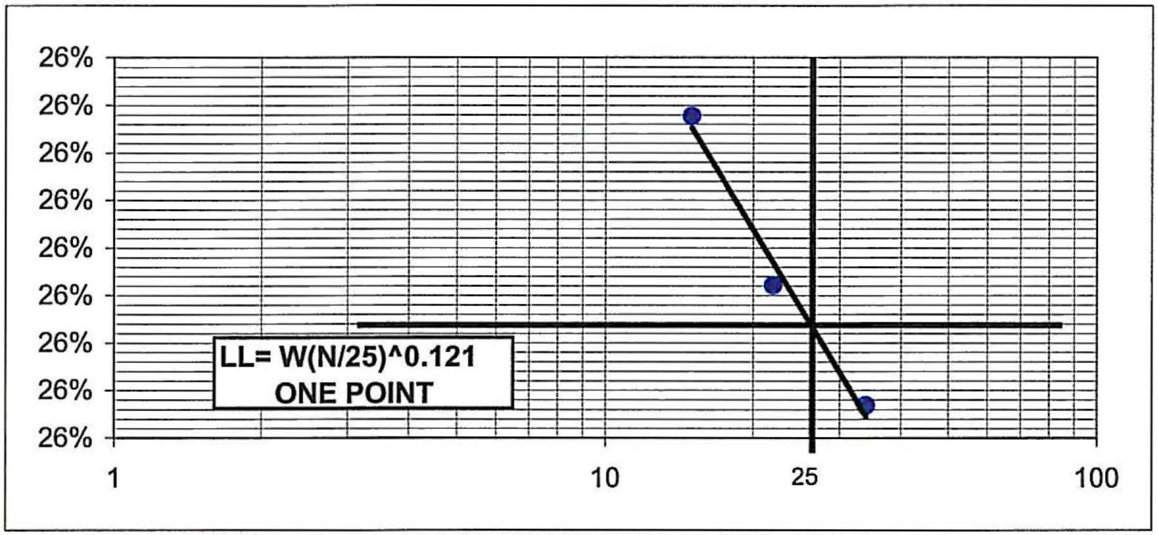
Job Name: James Ford Dealership **Date:** 02/12/13

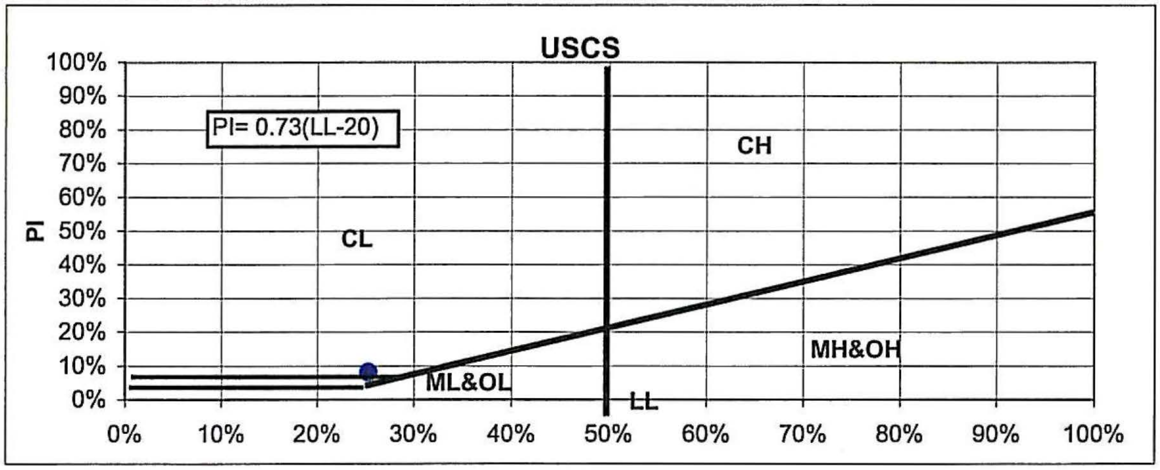
Job Number: 20-2375G **Lab #:** 3388

Sample Number: B1 @ 2'

ATTERBERG

	LIQUID LIMITS					PLASTIC LIMIT	
WET SOIL	13.52	13.32	13.88			9.90	
DRY SOIL	10.77	10.6	11.03			8.44	
WATER	2.75	2.72	2.85			1.46	
# BLOWS	34	22	15				
% MOIST	25.53%	25.66%	25.84%			17.30%	
					LL	PL	PI
ONE POINT	26.5%	25.3%	24.3%		25.4%	17.30%	8.05%





REVIEWED BY: **DATE:** 02/12/2013

Job Name: James Ford Dealership

Date: 02/12/13

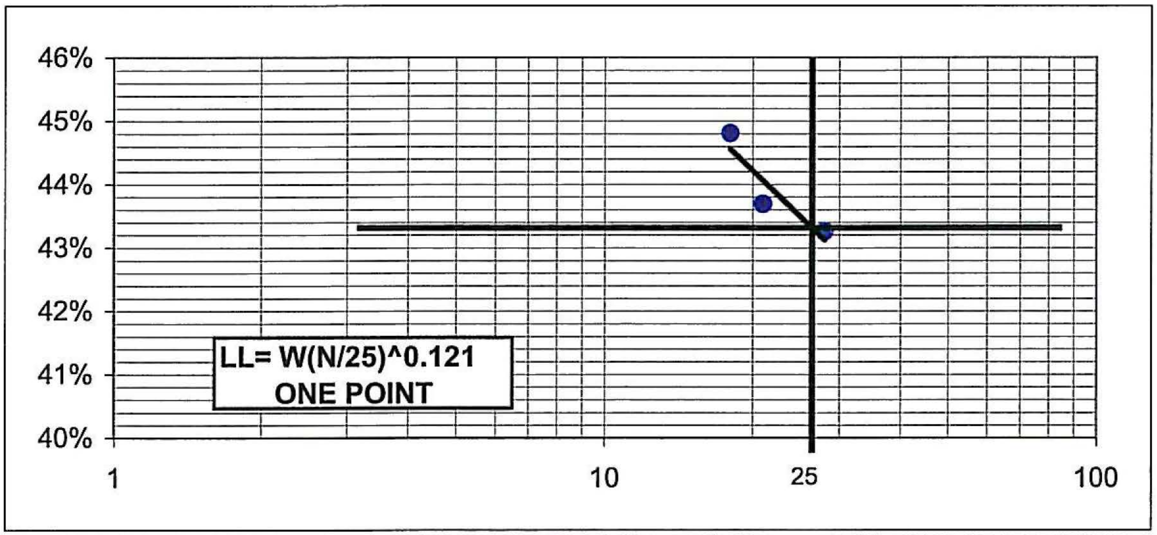
Job Number: 20-2375G

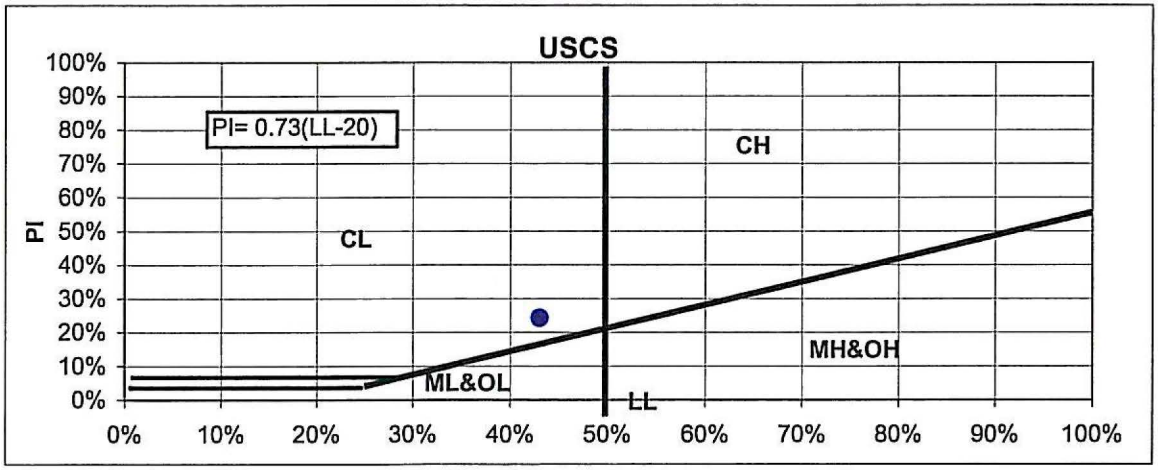
Lab #: 3388

Sample Number: B1 @ 5'

ATTERBERG

	LIQUID LIMITS					PLASTIC LIMIT	
WET SOIL	10.50	9.14	15.14			9.35	
DRY SOIL	7.34	6.37	10.47			7.87	
WATER	3.16	2.77	4.67			1.48	
# BLOWS	**28**	**21**	**18**				
% MOIST	**43.05%**	**43.49%**	**44.60%**			**18.81%**	
					LL	**PL**	**PI**
ONE POINT	**43.6%**	**42.6%**	**42.9%**		**43.0%**	**18.81%**	**24.22%**





REVIEWED BY: ______________________

DATE: 02/12/2013

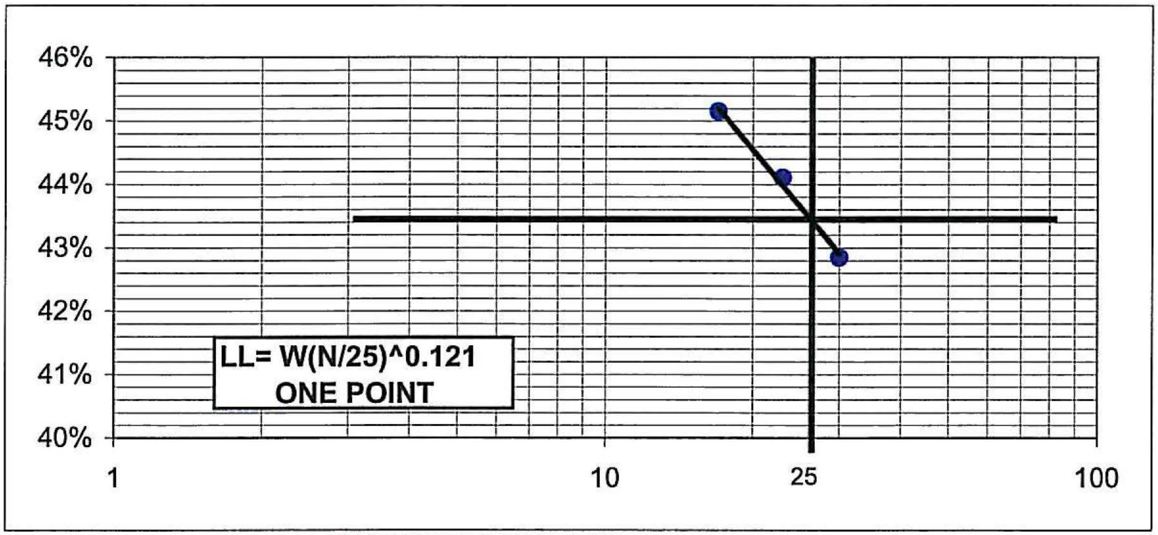
Job Name: James Ford Dealership **Date:** 02/12/13

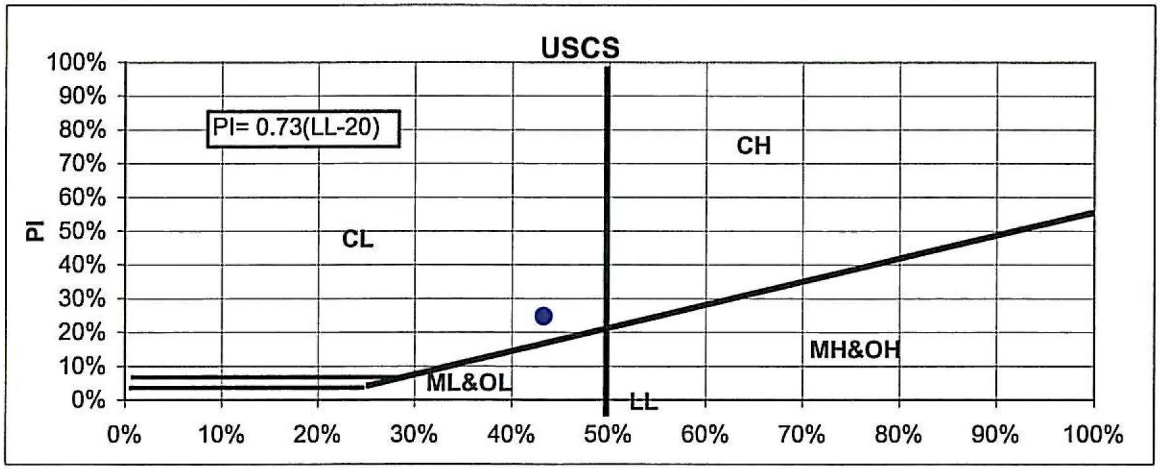
Job Number: 20-2375G **Lab #:** 3388

Sample Number: B1 @ 10'

ATTERBERG

	LIQUID LIMITS					PLASTIC LIMIT	
WET SOIL	10.57	9.08	15.06			9.43	
DRY SOIL	7.41	6.31	10.39			7.95	
WATER	3.16	2.77	4.67			1.48	
# BLOWS	30	23	17				
% MOIST	42.65%	43.90%	44.95%			18.62%	
					LL	PL	PI
ONE POINT	43.6%	43.5%	42.9%		43.3%	18.62%	24.70%





REVIEWED BY: **DATE:** 02/12/2013

JOB No: **20-2375G**
JOB NAME: **James Ford Dealership**
LAB No: **3388**

DATE: **02/05/2013**

EXPANSION INDEX TEST

ASTM D-4829

B1 2' & 5' Combined EI

		TRIAL 1	TRIAL 2
WET WEIGHT	(g)	28.7	
DRY WEIGHT	(g)	26.6	
% MOISTURE	(%)	7.9%	
WEIGHT OF RING & SOIL	(g)	**622.7**	
WEIGHT OF RING	(g)	200.8	
WEIGHT OF SOIL	(lbs.)	0.9293	
VOLUME OF SOIL	(cf)	0.0073	
WET DENSITY	(pcf)	127.8	
DRY DENSITY	(pcf)	118.4	
% SATURATION	(%)	**50.4%**	

EXPANSION READING

DATE TIME: INITIAL READING INCH
Feburary 6 11:05pm 0.000 0
DATE TIME: INITIAL READING
Feburary 7 10:00am 0.027 0.027
FINAL READING
27
EXPANSION INDEX
27

VERY LOW	**0-20**
LOW	**21-50**
MEDIUM	**51 -90**
HIGH	**91-130**
VERY HIGH	**130>**

POST TEST, MOISTURE

WET SOIL, (g): 421.9
DRY WT SOIL, (g): 371.7
% MOISTURE: **13.5%**

NOTES:

1.- 2.7 SP. GR. = 1/2.7= 0.3704
2.- % SATURATION MUST BE BETWEEN 45% AND 55%

EI at saturation between 40-60%

Measured EI: 27
Measured Saturation: 50.4

EI at 50% Saturation: **27**

Michael Mahurin - Lab Manager

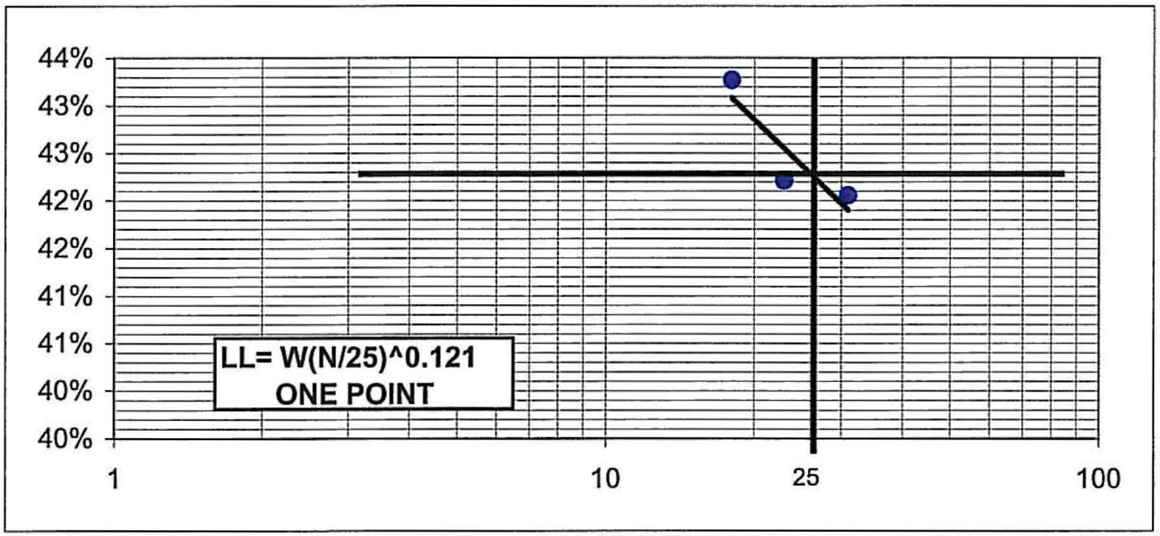
Date: February 7, 2013

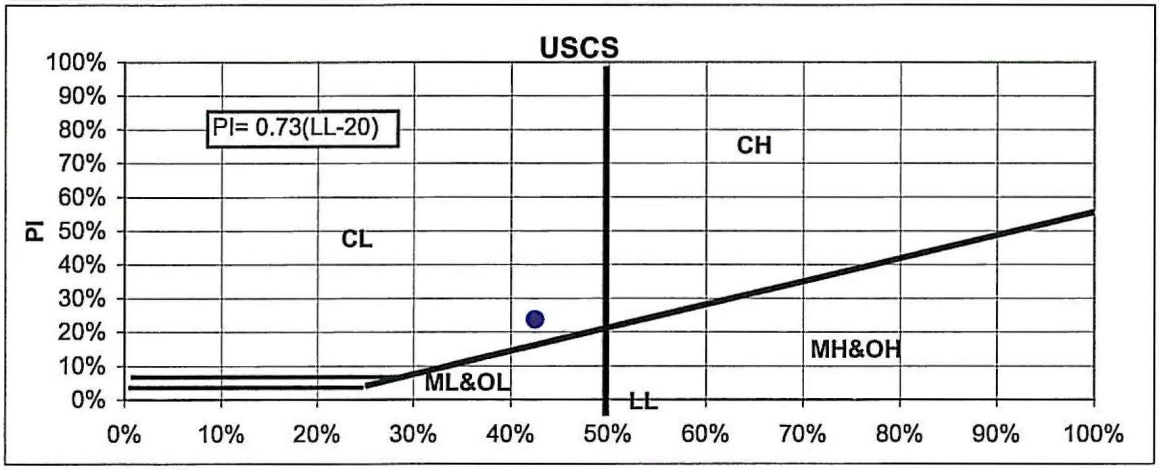
Construction Testing & Engineering, Inc.
North Highlands, CA
916-331-6030

Job Name: James Ford Dealership
Date: 02/12/13
Job Number: 20-2375G
Lab #: 3388
Sample Number: B2 @ 5'

ATTERBERG

	LIQUID LIMITS					PLASTIC LIMIT	
WET SOIL	10.56	9.31	15.29			9.35	
DRY SOIL	7.41	6.54	10.65			7.87	
WATER	3.15	2.77	4.64			1.48	
# BLOWS	**23**	**31**	**18**				
% MOIST	**42.51%**	**42.35%**	**43.57%**			**18.81%**	
					LL	**PL**	**PI**
ONE POINT	**42.1%**	**43.5%**	**41.9%**		**42.5%**	**18.81%**	**23.67%**





REVIEWED BY:

DATE: 02/12/2013

JOB No: **20-2375G**
JOB NAME: **James Ford Dealership**
LAB No: **3388**

DATE: **02/05/2013**

EXPANSION INDEX TEST

ASTM D-4829

B2 0' - 3' Bulk EI

		TRIAL 1	TRIAL 2
WET WEIGHT	(g)	163.6	
DRY WEIGHT	(g)	143.3	
% MOISTURE	(%)	14.2%	
WEIGHT OF RING & SOIL	(g)	**558.1**	
WEIGHT OF RING	(g)	202.0	
WEIGHT OF SOIL	(lbs.)	0.7844	
VOLUME OF SOIL	(cf)	0.0073	
WET DENSITY	(pcf)	107.9	
DRY DENSITY	(pcf)	94.5	
% SATURATION	(%)	**48.8%**	

EXPANSION READING

DATE	TIME:	INITIAL READING	INCH
Feburary 5	1:45pm	0.000	0

DATE	TIME:	INITIAL READING	
Feburary 6	9:00am	0.018	0

FINAL READING: 18

EXPANSION INDEX: 18

VERY LOW	**0-20**
LOW	**21-50**
MEDIUM	**51 -90**
HIGH	**91-130**
VERY HIGH	**130>**

POST TEST, MOISTURE

WET SOIL, (g): 394.1
DRY WT SOIL, (g): 316.0
% MOISTURE: **24.7%**

NOTES:

1.- 2.7 SP. GR. = 1/2.7= 0.3704
2.- % SATURATION MUST BE BETWEEN 45% AND 55%

EI at saturation between 40-60%

Measured EI: 18
Measured Saturation: 48.8

EI at 50% Saturation: **17**

Michael Mahurin - Lab Manager

Date: February 6, 2013

Construction Testing & Engineering, Inc.

North Highlands, CA
916-331-6030

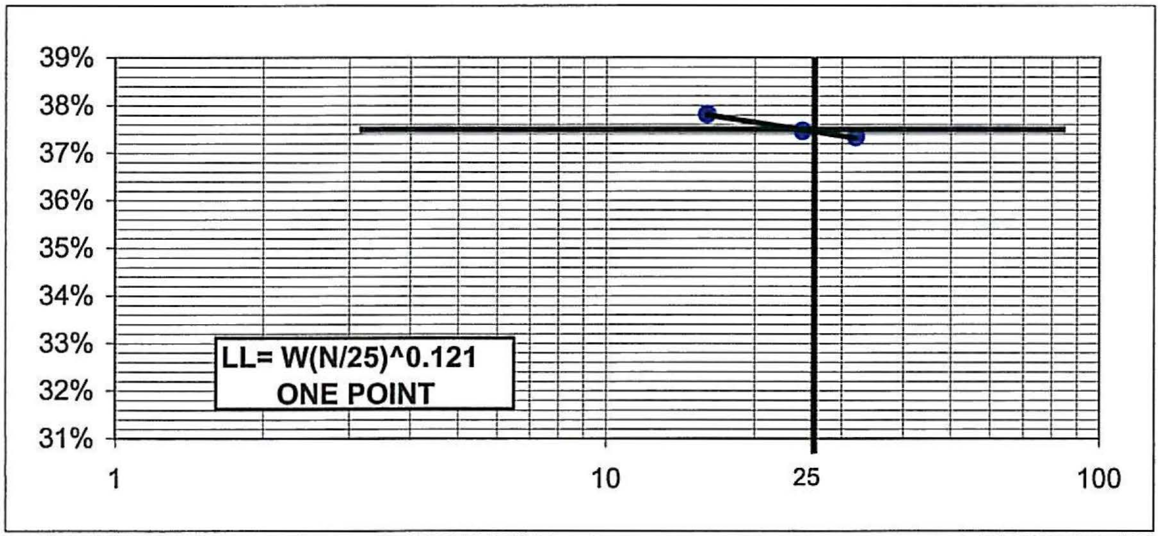
Job Name: James Ford Dealership
Date: 02/12/13

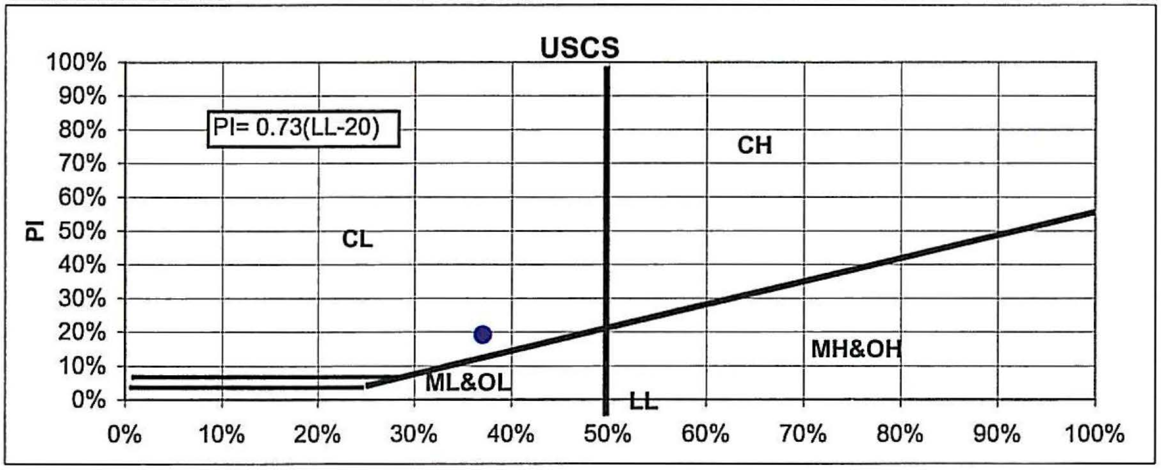
Job Number: 20-2375G
Lab #: 3388

Sample Number: B3 @ 2'

ATTERBERG

	LIQUID LIMITS					PLASTIC LIMIT	
WET SOIL	11.16	9.43	14.81			9.32	
DRY SOIL	8.11	6.87	10.8			7.90	
WATER	3.05	2.56	4.01			1.42	
# BLOWS	**16**	**25**	**32**				
% MOIST	**37.61%**	**37.26%**	**37.13%**			**17.97%**	
					LL	PL	PI
ONE POINT	**35.6%**	**37.3%**	**38.3%**		**37.0%**	**17.97%**	**19.08%**





REVIEWED BY:

DATE: 02/12/2013

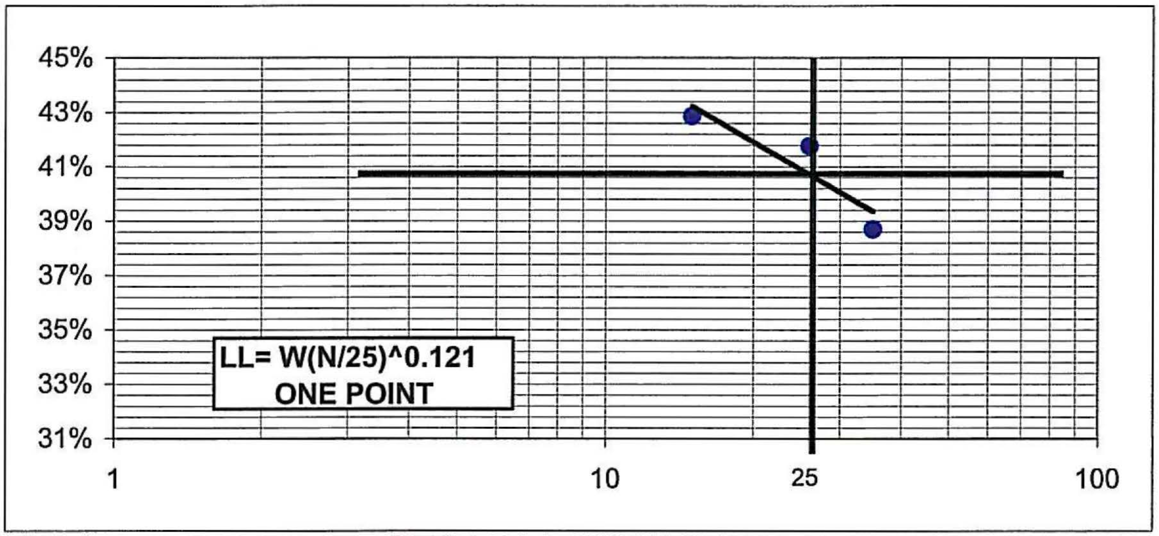
Job Name: James Ford Dealership **Date:** 02/12/13

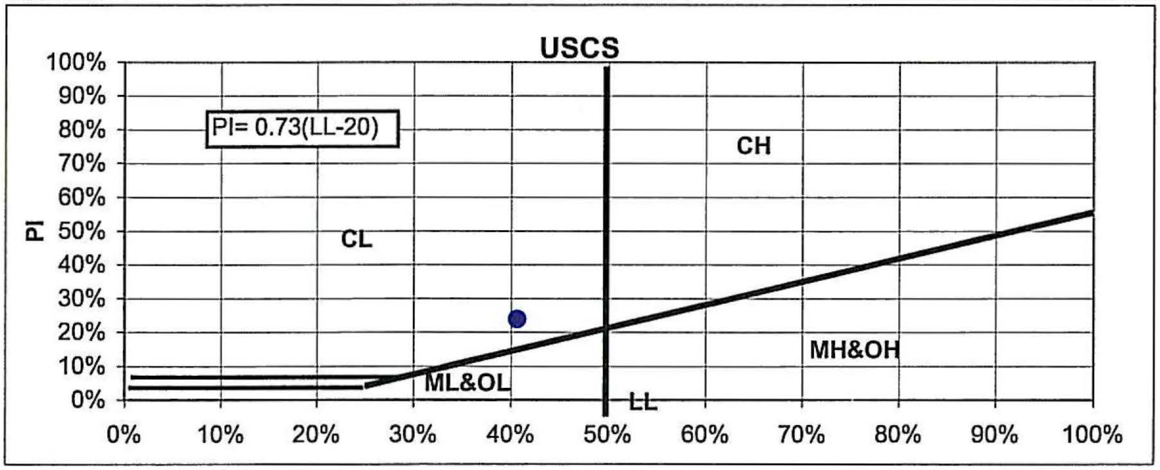
Job Number: 20-2375G **Lab #:** 3388

Sample Number: B3 @ 5'

ATTERBERG

	LIQUID LIMITS					PLASTIC LIMIT	
WET SOIL	11.07	8.96	15.87			9.38	
DRY SOIL	7.76	6.33	11.46			8.03	
WATER	3.31	2.63	4.41			1.35	
# BLOWS	**15**	**26**	**35**				
% MOIST	**42.65%**	**41.55%**	**38.48%**			**16.81%**	
					LL	PL	PI
ONE POINT	**40.1%**	**41.7%**	**40.1%**		**40.6%**	**16.81%**	**23.83%**





REVIEWED BY: ______________________ **DATE:** 02/12/2013

MOISTURE & DENSITY TEST

In Accordance with ASTM D2937

Project Name: James Ford Dealership
Job Number: 23-2375G **Date Sampled:** January 23, 2013
Lab Number: 23091 **Date Tested:** February 11, 2013

BORING NO.	B-2				
DEPTH (ft.)	2'				
SAMPLE HT. (in.)	6.0				
SOIL+RING (g)	866.7				
WT. OF RINGS(g)	0.0				
WT. OF SOIL (g)	866.7				
WT. OF SOIL (lb.)	1.9108				
VOL. OF RINGS (ft.^{3})	0.01592				
WET DENSITY (pcf)	120.0				
WET WT. (g)	162.3				
DRY WT. (g)	135.3				
% MOISTURE	**20.0**				
DRY DENSITY (pcf)	**100.1**				

Tested By: Chase V. Reviewed By:
Erik Campbell, Laboratory Manager

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1441 MONTIEL ROAD, SUITE 115 | ESCONDIDO, CA 92026 | 760.746.4955

REPORT OF RESISTANCE 'R' VALUE-EXPANSION PRESSURE

Job Name: James Ford Dealership
Project No. 20-2375G
Lab No. 23091
Type of Material: Drk brn clayey sand
Source of Material: B-5 @ 0-2'
Test Procedure: Cal 301

Sampled By: Tracy CTE **Date** 1/23/2013
Submitted By: Tracy CTE **Date:** 1/30/2013
Tested/ Calc.By: Stewart Sloan **Date:** 2/8/2013
Reviewed By: Erik Campbell **Date:** 2/12/2013

Specimen/ Mold No.	12	9	8	
Compactor Air Pressure, ft.lbs.	180	155	95	
Initial Moisture, %	19.8	19.8	19.8	
Wet weight and Dry weight, g	997.5 832.5	997.5 832.5	997.5 832.5	
Water Added, ml	-20	-10	0	
Moisture at Compaction, %	17.4	18.6	19.8	
Wt. Of Briquette and Mold, g	3164	3127	3113	
Wt. Of Mold, g	2117	2097	2098	
Wt. Of Briquitte,g	1047	1030	1015	
Height of Briquette, in	2.52	2.50	2.49	
Dry Density, pcf	107.3	105.3	103.1	
Stabilometer PH @ 1000 lbs	58	57	60	
Stabilometer PH @ 2000 lbs	135	137	136	
Displacement	3.88	4.32	4.53	
R' Value	10	8	8	
Corrected 'R' Value	10	8	8	
Exudation Pressure, lbs	3880	3000	1900	
Exudation Pressure, psi	310	240	152	
Stabilometer Thickness - ft	0.86	0.88	0.88	
Expansion Pressure	0.0022	0.0019	0.0013	
Expansion Press, Thick-ft	0.73	0.63	0.43	

Exudation 10

Expansion 13

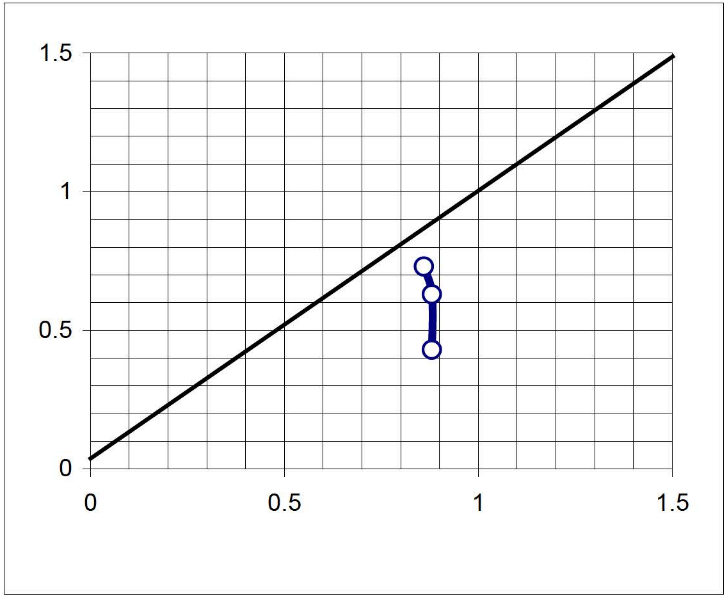
R-value **10**

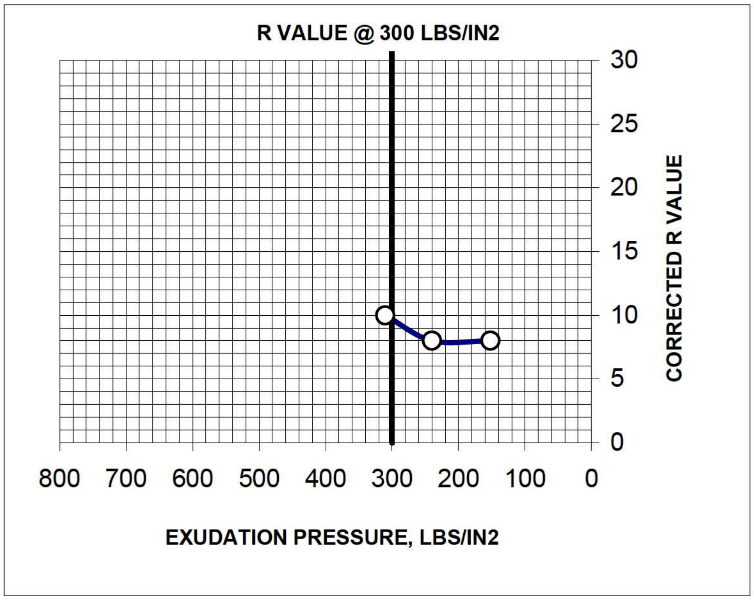
TI	4.5
Expansion	13

Initial Wt. Sample,g ____

Dry wt. Sample, g ____

Wet Wt. Sample, g ____

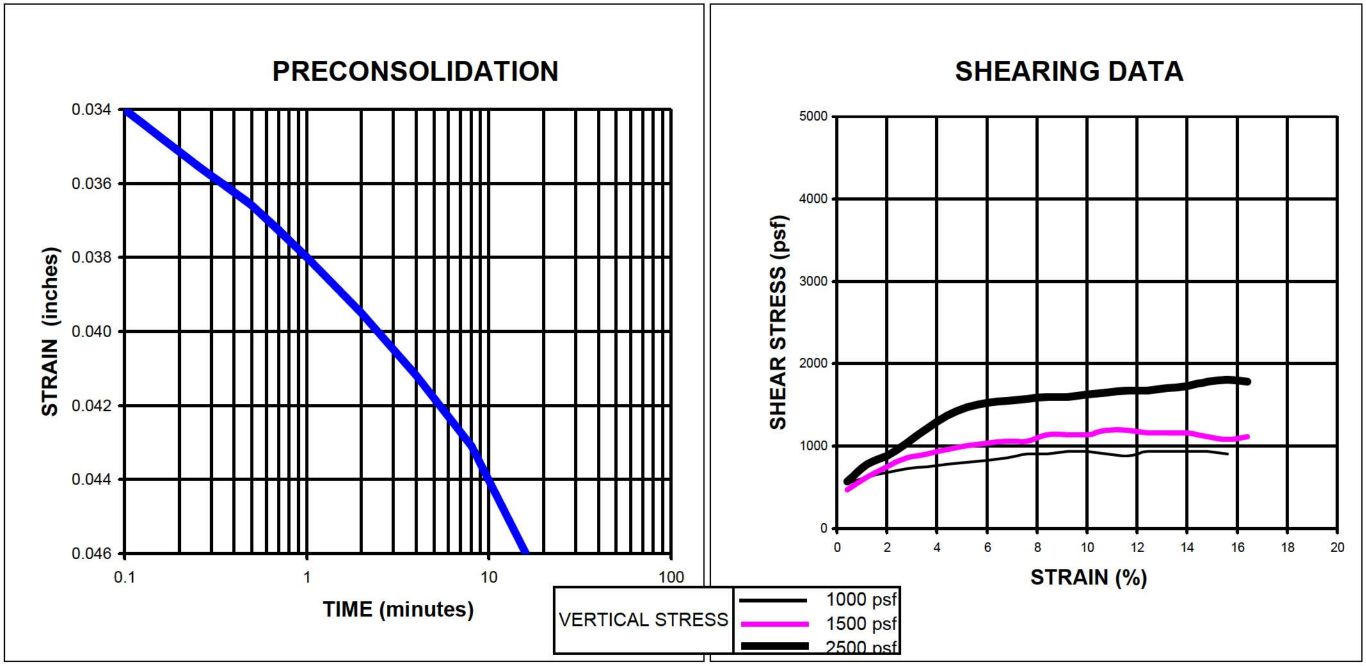


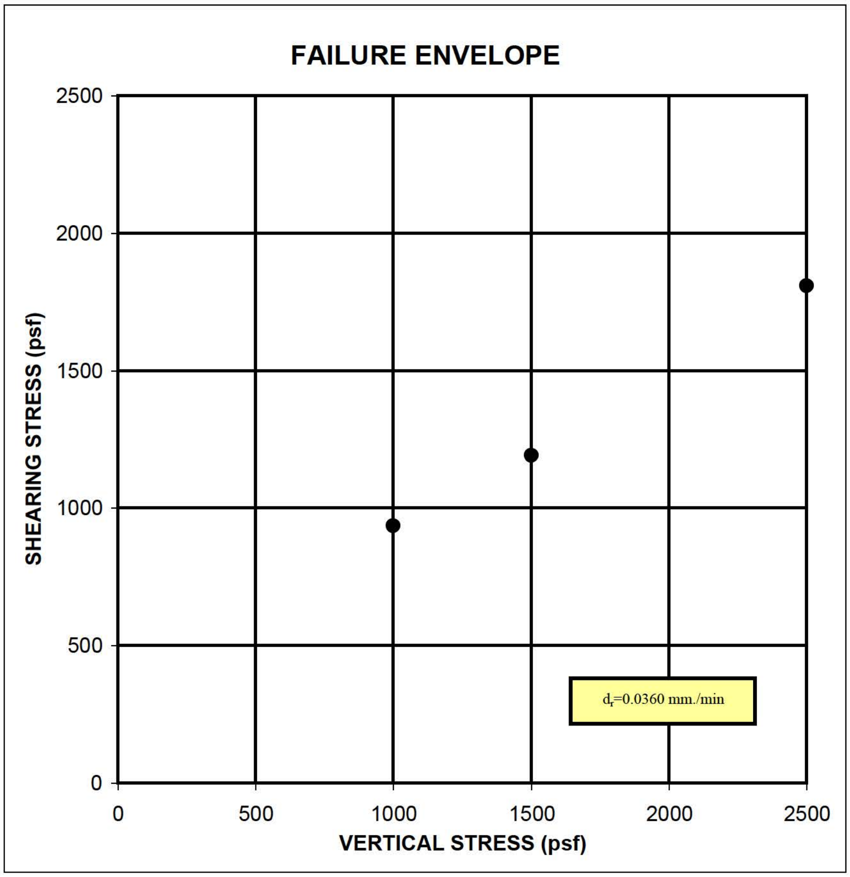


Cover Thickness by Expansion Pressure-Feet

Expansion From Graph:	0.84

Erik Campbell
Laboratory Manager





SHEAR STRENGTH TEST

Sample Designation	Depth (ft)	Cohesion	Angle of Friction	Sample Description	
B-2	2'	330 psf	30.4	Dark brown clay	
Initial Moisture (%):	20.0	Initial Dry Density (pcf)	100.1	CTE Job No:	20-2375G
Final Moisture (%):	23.6			Lab No:	23091