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| **TYPE OF SERVICES** | Geotechnical Investigation |
| **PROJECT NAME** | Sunrise Senior Living of Redwood City |
| **LOCATION** | 2991 El Camino Real<br>Redwood City, California |
| **CLIENT** | Sunrise Senior Living |
| **PROJECT NUMBER** | 935-1-2 |
| **DATE** | December 21, 2016 |

GEOTECHNICAL

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| **Client** | **Sunrise Senior Living** |
| **Client Address** | **7902 Westpark Drive**<br>**McLean, Virginia** |
| **Project Number** | **935-1-2** |
| **Date** | **December 21, 2016** |

**Prepared by**

**Matthew J. Schaffer, P.E.**
Project Engineer
Geotechnical Project Manager

REGISTERED PROFESSIONAL ENGINEER
MATTHEW J. SCHAFFER
No. 81968
3/31/18
CIVIL
STATE OF CALIFORNIA

**Danh T. Tran, P.E.**
Senior Principal Engineer
Quality Assurance Reviewer

1259 Oakmead Parkway | Sunnyvale, CA 94085
T 408 245 4600 | F 408 245 4620

1270 Springbrook Road, Suite 101 | Walnut Creek, CA 94597
T 925 988 9500 | F 925 988 9501

www.cornerstoneearth.com

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**Type of Services** | **Geotechnical Investigation**
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**Redwood City, California**

# SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Sunrise Senior Living for the Sunrise Senior Living of Redwood City project in Redwood City, 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 conceptual plans titled “Sunrise Senior Living, Redwood City, CA, Assisted Living Facility,” prepared by HPI Architecture, dated December 5, 2016.

- A set of civil plans, Sheet C1 titled “Topographic Survey of 2915 El Camino Real for Sunrise Senior Living,” Sheet C2 titled “Conceptual Grading and Drainage Plan of 2915 El Camino Real for Sunrise Senior Living,” and Sheet C3 titled “Conceptual Utility Plan of 2915 El Camino Real for Sunrise Senior Living,” prepared by Kier & Wright Civil Engineers & Surveyors, Inc., dated November, 2016.

## 1.1 PROJECT DESCRIPTION

The project will consist of demolishing the existing buildings and improvements on the approximately 1.4 acres, multiple parcel site and constructing a new two and three stories of above-grade, 88-unit assisted living facility over one story of below-grade parking. The building footprint will be approximately 27,810 square feet and we anticipate the parking garage will be of concrete construction while the assisted living facility floors will likely be of wood or steel-frame construction. Associated improvements and amenities necessary for site development will also be constructed as part of the overall project.

Structural loads are not available at the time of our report; however, structural loads are expected to be representative of this type of structure. We anticipate cuts on the order of 12 to 15 feet will be required for the one-level below grade parking.

## 1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated October 23, 2016 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 three borings drilled on November 10 and 11, 2016 with truck-mounted, hollow-stem auger drilling equipment and five Cone Penetration Tests (CPTs) advanced on November 8, 2016. The borings were drilled to depths of approximately 41 to 50 feet; the CPTs were advanced to depths of approximately 40 to 100 feet. Practical refusal was encountered at a depth of approximately 40 feet in CPT-4. Seismic shear wave velocity measurements were collected from CPT-2. Borings EB-1, EB-2, and EB-3 were advanced adjacent to CPT-1, CPT-2, and CPT-3, respectively, for direct evaluation of physical samples to correlated soil behavior.

The borings and CPTs were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

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

## 1.4 LABORATORY TESTING PROGRAM

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

## 1.5 CORROSION EVALUATION

Three samples from our borings at depths of 1½ to 14½ feet were tested for saturated resistivity, pH, and soluble sulfates and chlorides. In general, the on-site soils can be characterized as moderately to severely corrosive to buried metal, and non-corrosive to buried concrete.

## 1.6 ENVIRONMENTAL SERVICES

Cornerstone Earth Group also provided environmental services for this project, including a Phase 1 site assessment; environmental findings and conclusions are provided under a separate report.

# SECTION 2: REGIONAL SETTING

## 2.1 REGIONAL SEISMICITY

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

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) |
| Monte Vista-Shannon | 3.1 | 5.0 |
| San Andreas (1906) | 4.8 | 7.8 |
| San Gregorio | 13.7 | 22.0 |
| Hayward (Total Length) | 14.5 | 23.3 |

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

# SECTION 3: SITE CONDITIONS

## 3.1 SURFACE DESCRIPTION

The multiple parcel site is located at the northwest corner of East Selby Lane and El Camino Real in Redwood City, California. The site is bounded by El Camino Real to the southwest, a one- and two-story building and parking lot to the northwest, residential houses to the north, Markham Avenue to the northeast, and East Selby Lane to the southeast. The site is approximately 1.4 acres and currently occupied by a one-story building in the southern corner, a two-story building in the western corner, and one-story residential homes in the northern corner. The central portion and eastern corner of the site is an asphalt concrete parking lot and an asphalt concrete alley runs northwest-southeast through the middle of the site. The site is relatively level with elevations generally about Elevation 37 to 39 feet (NAVD 88) based on the topographic map provided to us. Various landscaping areas are generally around the perimeter of the site and consist of mature trees and shrubs with some areas of grass.

Surface pavements generally consisted of 2 to 3½ inches of asphalt concrete over 4 inches of aggregate base. Based on visual observations, the existing pavements are in good to fair shape with minor cracking.

## 3.2 SUBSURFACE CONDITIONS

Below the surface pavements, our explorations encountered hard, highly expansive clay to a depth of approximately 4 feet. Below the highly expansive surficial clays, generally stiff to hard lean clays with variable amounts of sand were encountered to the maximum depth explored of 100 feet. Some loose to very dense silty sand, clayey sand, and poorly graded sands with variable amounts of silt and clay were interbedded within the lean clays with some larger sand layers ranging up to about 10 feet thick at depths generally around 25 to 35 feet and 45 to 55 feet below the surface. Sandy silts were also encountered in Boring EB-1 beneath the surficial highly expansive clay to a depth of about 7½ feet and in Boring EB-3 at a depth of about 37½ feet down to the terminal depth in the boring at 41 feet beneath the surface.

### 3.2.1 Plasticity/Expansion Potential

We performed one Plasticity Index (PI) test on a representative sample. The test result was used to evaluate the expansion potential of surficial soils. The results indicated a PI of 35 and a Liquid Limit (LL) of 53, indicating a high 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 15 feet range from about 5 percent below optimum to about 10 percent over the estimated laboratory optimum moisture.

### 3.2.3 Sulfate Contents

Laboratory testing indicated that the soluble sulfate contents were 24 to 51 parts per million (ppm), indicating negligible corrosion potential to buried concrete.

## 3.3 GROUND WATER

Ground water was encountered in our borings (Borings EB-1, EB-2, and EB-3) at depths ranging from approximately 23½ to 30 feet below current grades. Ground water was inferred from CPT pore pressure measurements in CPT-1 and CPT-3 at depths of approximately 21 to 24 feet, respectively, below currents grades. All measurements were taken at the time of drilling and may not represent the stabilized levels that can be higher than the initial levels encountered. Based on review of depth to ground water maps (CGS, Palo Alto 7.5 minute quadrangle, 2006), we anticipate that the high ground water level will be on the order of 20 feet below current grades. We recommend a design ground water depth of 20 feet below the existing ground surface be used. This correlates to about Elevations 17 to 19 feet based on the topographic map provided to us and referenced in Section 1.

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

## 3.4 CORROSION SCREENING

We tested three samples collected at depths ranging from approximately 1½ to 14½ feet for resistivity, pH, soluble sulfates, and chlorides. The laboratory test results are summarized in Table 2.

**Table 2: Summary of Corrosion Test Results**

| Boring/Sample | Depth (feet) | Soil pH[1] | Resistivity[2] (ohm-cm) | Chloride[3,5] (mg/kg) | Sulfate[4,5] (mg/kg) |
|---|---|---|---|---|---|
| EB-1/1A | 1½ | 6.0 | 1,344 | 5 | 24 |
| EB-3/2A | 5½ | 6.4 | 1,836 | 8 | 50 |
| EB-3/4A | 14½ | 6.9 | 2,815 | 5 | 51 |

Notes: [1] ASTM G51
[2] ASTM G57 - 100% saturation
[3] ASTM D4327/Cal 422 Modified
[4] ASTM D4327/Cal 417 Modified
[5] 1 mg/kg = 0.0001 % by dry weight

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

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

In accordance with the 2016 CBC Section 1904A.1, alternative cementitious materials for sulfate exposure shall be determined in accordance with ACI 318-11 Table 4.2.1 and Table 4.3.1. Based on the laboratory test results, no cement type restriction is required, although, in our opinion, it is generally a good practice to include some sulfate resistance and to maintain a relatively low water-cement ratio. We have summarized applicable design values and parameters from ACI 318 Table 4.3.1 below in Table 3.

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

**Table 3: ACI Sulfate Soil Corrosion Design Values and Parameters**

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

Notes: (1) above values and parameters are from on ACI 318-11, 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 $(PGA)_M$ was estimated for analysis using a value equal to $F_{PGA}$ x PGA, as allowed in the 2016 edition of the California Building Code. For our liquefaction analysis we used a PGA of 0.672g.

### 4.3 LIQUEFACTION POTENTIAL

The site is within a State-designated Liquefaction Hazard Zone (CGS, Palo Alto Quadrangle, 2006). Our field and laboratory programs addressed this issue by testing and sampling potentially liquefiable layers to depths of at least 50 feet, performing visual classification on sampled materials, evaluating CPT data, and performing various tests to further classify soil properties.

### 4.3.1 Background

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

### 4.3.2 Analysis

As discussed in the “Subsurface” section above, several sand layers were encountered below the design ground water depth of 20 feet. Following the liquefaction analysis framework in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008), incorporating updates in *CPT and SPT Based Liquefaction Triggering Procedures* (Boulanger and Idriss, 2014), and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008) for quantitative analysis, these layers were analyzed for liquefaction triggering and potential post-liquefaction settlement. These methods compare the ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil’s estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.3 are considered to be potentially liquefiable and capable of post-liquefaction re-consolidation (i.e. settlement).

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

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

In estimating post-liquefaction settlement at the site, we have implemented a depth weighting factor proposed by Cetin (2009). Following evaluation of 49 high-quality, cyclically induced, ground settlement case histories from seven different earthquakes, Cetin proposed the use of a weighting factor based on the depth of layers. The weighting procedure was used to tune the surface observations at liquefaction sites to produce a better model fit with measured data. Aside from the better model fit it produced, the rationale behind the use of a depth weighting factor is based on the following: 1) upward seepage, triggering void ratio redistribution, and resulting in unfavorably higher void ratios for the shallower sublayers of soil layers; 2) reduced induced shear stresses and number of shear stress cycles transmitted to deeper soil layers due to initial liquefaction of surficial layers; and 3) possible arching effects due to nonliquefied soil layers. All these may significantly reduce the contribution of volumetric settlement of deeper soil layers to the overall ground surface settlement (Cetin, 2009).

The results of our CPT analyses (CPT-1 through CPT-5) are presented on Figures 4A through 4E of this report.

### 4.3.3 Summary

Our analyses indicate that several layers could potentially experience liquefaction triggering that could result in post-liquefaction total settlement below the bottom of the one-level basement (estimated to be about 15 feet below the surface) ranging up to ¾-inch based on the Yoshimine (2006) method. As discussed in Special Publication 117A, differential movement for level ground sites over deep soil sites will be up to about two-thirds of the total settlement between independent foundation elements. In our opinion, differential settlements are anticipated to be on the order of ½-inch between independent foundation elements.

### 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. The work of Youd and Garris (1995) indicates that the 25-foot thick layer of non-liquefiable cap is sufficient to prevent ground rupture; therefore the above total settlement estimates are reasonable.

## 4.4 LATERAL SPREADING

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

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 to medium dense unsaturated sandy soils can settle during strong seismic shaking. We evaluated the potential for seismic compaction of the unsaturated soils above the design ground water level of 20 feet based on the work by Robertson and Shao (2010). Based on our analysis, 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 2½ miles inland from the San Francisco Bay shoreline, and is at approximately Elevation 37 to 39 feet. Therefore, the potential for inundation due to tsunami or seiche is considered low.

### 4.7 FLOODING

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

## SECTION 5: CONCLUSIONS

### 5.1 SUMMARY

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

- Potential for liquefaction-induced settlements
- Presence of highly expansive surficial soils
- Differential movement at on-grade to on-structure transitions
- Presence of granular soils
- Soil corrosion potential

### 5.1.1 Potential for Liquefaction-Induced Settlements

As discussed, our liquefaction analysis indicates that there is a potential for liquefaction of localized sand layers during a significant seismic event. Although the potential for liquefied sands to vent through the overlying soils is considered low, our analyses indicates that total liquefaction-induced settlement of up to approximately ¾ inch could occur below the bottom of basement, resulting in differential settlement up to about ½-inch. Foundations should be designed to tolerate the anticipated total and differential settlements. Detail foundation recommendations are presented in the "Foundations" section.

### 5.1.2 Presence of Highly Expansive Surficial Soils

Highly expansive surficial soils generally blanket the site. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. To reduce the potential for damage to the planned structures, slabs-on-grade bearing on expansive soil should have sufficient reinforcement and be supported on a layer of non-expansive fill; any at-grade 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.

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

Some improvements may transition from on-grade support to overlying the basement (on-structure). Where the improvements transition from on-grade to the basement, these transition areas typically experience increased differential movement due to a variety of causes, including difficulty in achieving compaction of retaining wall backfill closest to the wall. We recommend consideration be given to including subslabs beneath flatwork or pavers that can cantilever at least 3 feet beyond the wall. If surface improvements are included that are highly sensitive to differential movement, additional measures may be necessary. We also recommend that retaining wall backfill be compacted to 95 percent where surface improvements are planned (see "Retaining Wall" section).

### 5.1.4 Presence of Granular Soils

As mentioned, the soils encountered at the site were generally clayey in the upper 15 feet of the soil profile. However, some silty sands were encountered at the approximate basement excavation depth in Boring EB-1. These sands at this depth contained high fines content however the fines were silty with low cohesion. If these soils, as well as any sands with lower fines are encountered, contractors may need to form footings as well as prepared slab-on-grade subgrade shortly prior to concrete placement and other similar construction issues as relates to temporary shoring, utility excavations, and granular material at the base of the basement excavation. These concerns are discussed further within the "Earthwork" and "Foundations" sections of this report.

### 5.1.5 Soil Corrosion Potential

As discussed, we performed a preliminary soil corrosion screening based on the results of analytical tests on samples of the near-surface soil. In general, test results indicate the use of sulfate resistant concrete is not required for buried concrete; however, the corrosion potential for buried metallic structures, such as metal pipes, is considered moderately to severely corrosive. We recommend that special requirements for corrosion control be made to protect metal pipes. We recommend the structural engineer and a corrosion engineer be retained to confirm the information provided and for additional recommendations, as required.

### 5.2 PLANS AND SPECIFICATIONS REVIEW

We recommend that we be retained to review the geotechnical aspects of the project structural, civil, and landscape plans and specifications, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction.

### 5.3 CONSTRUCTION OBSERVATION AND TESTING

As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that a Cornerstone representative be present to provide geotechnical observation and testing during earthwork and foundation construction. This will allow us to form an opinion and prepare a letter at the end of construction regarding contractor compliance with project plans and specifications, and with the recommendations in our report. We will also be allowed to evaluate any conditions differing from those encountered during our investigation, and provide supplemental recommendations as necessary. For these reasons, the recommendations in this report are contingent of Cornerstone providing observation and testing during construction. Contractors should provide at least a 48-hour notice when scheduling our field personnel.

## SECTION 6: EARTHWORK

### 6.1 SITE DEMOLITION, CLEARING AND PREPARATION

#### 6.1.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements within the proposed development area. Demolition of existing improvements is discussed in detail below. A detailed discussion of removal of existing fills is provided later in this report. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight.

#### 6.1.2 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 Demolition of Existing Slabs, Foundations and Pavements

All slabs, foundations, and pavements should be completely removed from within planned building areas. Slabs, foundations, and pavements that extend into planned flatwork, pavement, or landscape areas may be left in place provided there is at least 3 feet of engineered fill overlying the remaining materials, they are shown not to conflict with new utilities, and that asphalt and concrete more than 10 feet square is broken up to provide subsurface drainage. A discussion of recycling existing improvements is provided later in this report.

### 6.1.4 Abandonment of Existing Utilities

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

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

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

## 6.2 REMOVAL OF EXISTING FILLS

Fills were not encountered in our explorations, but we anticipate any existing fill present within the proposed building footprint will be removed for the basement excavation that is anticipated to extend to about 12 to 15 feet below existing surrounding grades. If any fills are encountered in at-grade building areas, they should be completely removed from within the building footprint and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the "Material for Fill" requirements below, the fills may be reused when backfilling the excavations. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should be screened out of the remaining material and removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the "Compaction" section below.

Fills extending into planned pavement and flatwork areas may be left in place provided they are determined to be a low risk for future differential settlement and that the upper 12 to 18 inches of fill below pavement subgrade is re-worked and compacted as discussed in the “Compaction” section below.

## 6.3 TEMPORARY CUT AND FILL SLOPES

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

Excavations performed during site demolition and fill removal should be sloped at 3:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Excavations extending more than 5 feet below building subgrade and excavations in pavement and flatwork areas should be sloped at a 1.5:1 inclination unless the OSHA soil classification indicates differently.

## 6.4 BELOW-GRADE EXCAVATIONS

Below-grade excavations may be constructed with temporary slopes in accordance with the “Temporary Cut and Fill Slopes” section above if space allows. Alternatively, temporary shoring may support the planned cuts up to about 15 feet. We have provided geotechnical parameters for shoring design in the section below. The choice of shoring method should be left to the contractor’s judgment based on experience, economic considerations and adjacent improvements such as utilities, pavements, and foundation loads. Temporary shoring should support adjacent improvements without distress and should be the contractor’s responsibility. A pre-condition survey including photographs and installation of monitoring points for existing site improvements should be included in the contractor’s scope. We should be provided the opportunity to review the geotechnical parameters of the shoring design prior to implementation; the project structural engineer should be consulted regarding support of adjacent structures.

### 6.4.1 Temporary Shoring

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

### Table 4: Suggested Temporary Shoring Design Parameters

| Design Parameter | Design Value |
|---|---|
| Minimum Lateral Wall Surcharge (upper 5 feet) | 120 psf |
| Cantilever Wall – Triangular Earth Pressure | 45 pcf (2) |
| Restrained Wall – Trapezoidal Earth Pressure | Increase from 0 to 25H* psf at ¼ H from top of shoring (1) (2) |
| Passive Pressure – Starting below the bottom of the adjacent excavation(3) | 350 pcf up to 1,400 psf maximum uniform pressure |

(1) H equals the height of the excavation; passive pressures are assumed to act over twice the soldier pile diameter
(2) The cantilever and restrained pressures are for drained designs with dewatering. If undrained shoring is designed, an additional 40 pcf should be added for hydrostatic pressures.
(3) Bottom of adjacent excavation is bottom of mass excavation or bottom of footing excavation, whichever is deeper directly adjacent to the shoring element.

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

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

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

As previously mentioned, we recommend that a monitoring program be developed and implemented to evaluate the effects of the shoring on adjacent improvements. All sensitive improvements should be located and monitored for horizontal and vertical deflections and distress cracking based on a pre-construction survey. The monitoring frequency should be established and agree to by the project team prior to start of shoring construction.

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

## 6.5 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.

Sandier soils may be encountered in areas of the basement subgrade elevation. We recommend that subgrade compaction and proof rolling be performed within 24 hours of capillary break layer or slab-on-grade construction.

## 6.6 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 range from about 5 percent below optimum to about 10 percent over the estimated laboratory optimum in the upper 15 feet of the soil profile. The contractor should anticipate drying and moisture conditioning the soils prior to reusing them as fill. The in-situ moisture contents at the anticipated bottom of basement excavation range up to about 10 percent over the estimated laboratory optimum moisture. Repetitive rubber-tire loading may de-stabilize these soils.

There are several methods to address potentially 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 8 to 12 inches and allowed to dry to near optimum conditions, if sufficient dry weather is anticipated to allow sufficient drying. More than one round of scarification may be needed to break up the soil clods.

### 6.6.2 Removal and Replacement

As an alternative to scarification, the contractor may choose to over-excavate the unstable soils and replace them with dry on-site or import materials. A Cornerstone representative should be present to provide recommendations regarding the appropriate depth of over-excavation, whether a 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

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

### 6.7.2 Re-Use of On-Site Site Improvements

We anticipate that asphalt concrete (AC) grindings and aggregate base (AB) will be generated during site demolition. If the AC grindings are mixed with the underlying AB to meet Class 2 AB specifications, they may be reused within the new pavement and flatwork structural sections. AC/AB grindings may not be reused beneath the habitable areas. Laboratory testing will be required to confirm the grindings meet project specifications.

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

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

## 6.8 COMPACTION REQUIREMENTS

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

**Table 5: Compaction Requirements**

| Description | Material Description | Minimum Relative[1] Compaction (percent) | Moisture[2] Content (percent) |
|---|---|---|---|
| General Fill (within upper 5 feet) | On-Site Expansive Soils | 87 – 92 | >3 |
| | Low Expansion Soils | 90 | >1 |
| General Fill (below a depth of 5 feet) | On-Site Expansive Soils | 95 | >3 |
| | Low Expansion Soils | 95 | >1 |
| Basement Wall Backfill | Without Surface Improvements | 90 | >1 |
| Basement Wall Backfill | With Surface Improvements | 95[4] | >1 |
| Trench Backfill | On-Site Expansive Soils | 87 – 92 | >3 |
| Trench Backfill | Low Expansion Soils | 90 | >1 |
| Trench Backfill (upper 6 inches of subgrade) | On-Site Low Expansion Soils | 95 | >1 |

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

Table 5 Continues

**Table 5: Compaction Requirements (Continued)**

| Description | Material Description | Minimum Relative[1] Compaction (percent) | Moisture[2] Content (percent) |
|---|---|---|---|
| Crushed Rock Fill | ¾-inch Clean Crushed Rock | Consolidate In-Place | NA |
| Non-Expansive Fill | Imported Non-Expansive Fill | 90 | Optimum |
| Flatwork Subgrade | On-Site Expansive Soils | 87 - 92 | >3 |
| Flatwork Subgrade | Low Expansion Soils | 90 | >1 |
| Flatwork Aggregate Base | Class 2 Aggregate Base[3] | 90 | Optimum |
| Pavement Subgrade | On-Site Expansive Soils | 87 - 92 | >3 |
| Pavement Subgrade | Low Expansion Soils | 95 | >1 |
| Pavement Aggregate Base | Class 2 Aggregate Base[3] | 95 | Optimum |
| Asphalt Concrete | Asphalt Concrete | 95 (Marshall) | NA |

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

### 6.8.1 Construction Moisture Conditioning

Expansive soils can undergo significant volume change when dried then wetted. The contractor should keep all exposed expansive soil subgrade (and also trench excavation side walls) moist until protected by overlying improvements (or trenches are backfilled). 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 on splash blocks or in closed conduits, to approved infiltration facilities, or on to hardscaped surfaces that drain to suitable facilities. Retention, detention or infiltration facilities should be spaced at least 10 feet from buildings, and preferably at least 5 feet from slabs-on-grade or pavements. However, if retention, detention or infiltration facilities are located within these zones, we recommend that these treatment facilities meet the requirements in the Storm Water Treatment Design Considerations section of this report.

## 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 are 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 mapped at a depth of about 20 feet, and therefore is expected to be at least 10 feet below the base of the infiltration measure.

- In our opinion, infiltration locations within 10 feet of the basement walls 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.

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

- Using drip irrigation

- Avoiding open planting within 3 feet of the building perimeter or near the top of existing slopes

- Regulating the amount of water distributed to lawns or planter areas by using irrigation timers

- Selecting landscaping that requires little or no watering, especially near foundations.

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

## SECTION 7: FOUNDATIONS

### 7.1 SUMMARY OF RECOMMENDATIONS

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

### 7.2 SEISMIC DESIGN CRITERIA

The project structural design should be based on the 2016 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. Shear wave velocity measurements performed at CPT-2 to a depth of 100 feet resulted in an average shear wave velocity of 902 feet per second (or 275 meters per second). 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 *U.S. Seismic Design Maps*, located at http://earthquake.usgs.gov/designmaps/us/application.php, based on the site coordinates presented below and the site classification. The table below lists the various factors used to determine the seismic coefficients and other parameters.

**Table 6: CBC Site Categorization and Site Coefficients**

| Classification/Coefficient | Design Value |
|---|---|
| Site Class | D |
| Site Latitude | 37.46913° |
| Site Longitude | -122.21085° |
| 0.2-second Period Mapped Spectral Acceleration[1], $S_S$ | 1.689g |
| 1-second Period Mapped Spectral Acceleration[1], $S_1$ | 0.779g |
| 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 - $S_{MS}$ | 1.689g |
| 1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – $S_{M1}$ | 1.168g |
| 0.2-second Period, Design Earthquake Spectral Response Acceleration – $S_{DS}$ | 1.126g |
| 1-second Period, Design Earthquake Spectral Response Acceleration – $S_{D1}$ | 0.779g |
| Mapped MCE Geometric Mean Peak Ground Acceleration - PGA | 0.672g |
| Site Coefficient Based on PGA and Site Class - $F_{PGA}$ | 1.0 |

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

## 7.3 SHALLOW FOUNDATIONS

### 7.3.1 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.

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

### 7.3.2 Footing Settlement

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

**Table 7: Estimated Structural Loading**

| Foundation Area | Range of Assumed Loads |
|---|---|
| Interior Isolated Column Footing | 400 to 500 kips |
| Exterior Isolated Column Footing | 200 to 250 kips |
| Perimeter Strip Footing | 7 to 9 kips per lineal foot |

Based on the above loading and the allowable bearing pressures presented above, we estimate that the total static footing settlement will be on the order of ½ inch, with less than about ½-inch of post-construction differential settlement between adjacent foundation elements. In addition we estimate that differential seismic movement will be on the order of ½-inch over a horizontal distance of 30 feet, resulting in a total estimated differential footing movement of about ¾-inch between foundation elements, assumed to be on the order of 30 feet. As our footing loads were assumed, we recommend we be retained to review the final footing layout and loading, and verify the settlement estimates above.

### 7.3.3 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 0.35 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 350 pcf may be used in design. The structural engineer should apply an appropriate factor of safety 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.

### 7.3.4 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.

## SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

### 8.1 BELOW-GRADE GARAGE SLAB-ON-GRADE

The garage slab-on-grade should be at least 5 inches thick and if constructed with minimal reinforcement intended for shrinkage control only, should have a minimum compressive strength of 3,000 psi. If the slab will have heavier reinforcing because the slab will also serve as a structural diaphragm, the compressive strength may be reduced to 2,500 psi at the structural engineer's discretion. The garage slab should also be supported on subgrade prepared in accordance with the recommendations in the “Earthwork” section of this report, and at least 6 inches of either Class 2 aggregate base or ¾-inch clean, crushed rock placed and compacted in accordance with the “Compaction” section of this report. If there will be areas within the garage that are moisture sensitive, such as equipment and elevator rooms, the recommendations in the “Interior Slabs Moisture Protection Considerations” section below may be incorporated in the project design if desired. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

### 8.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

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

- Place a minimum 10-mil 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 concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement.

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

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

- Where floor coverings are planned, all concrete surfaces should be properly cured.
- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869-98 and F710-98 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

### 8.3 EXTERIOR FLATWORK

Exterior concrete flatwork subject to pedestrian traffic only should be at least 4 inches thick and supported on at least 12 inches of non-expansive fill (NEF) overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. In addition, the upper 4 inches of the NEF should also meet Class 2 aggregate base requirements. As an alternative, the Class 2 aggregate base can also be increased to the full depth of NEF as recommended above.

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 except where limited sections of structural slabs are included to help span irregularities at the transitions between at-grade and on-structure flatwork.

## SECTION 9: VEHICULAR PAVEMENTS

### 9.1 ASPHALT CONCRETE

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

**Table 8: Asphalt Concrete Pavement Recommendations, Design R-value = 5**

| Design Traffic Index (TI) | Asphalt Concrete (inches) | Class 2 Aggregate Base* (inches) | Total Pavement Section Thickness (inches) |
|---|---|---|---|
| 4.0 | 2.5 | 7.5 | 10.0 |
| 4.5 | 2.5 | 9.5 | 12.0 |
| 5.0 | 3.0 | 10.0 | 13.0 |
| 5.5 | 3.0 | 12.0 | 15.0 |
| 6.0 | 3.5 | 13.0 | 16.5 |
| 6.5 | 4.0 | 14.0 | 18.0 |

*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 using the pavements.

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

## 9.2 PORTLAND CEMENT CONCRETE

The exterior Portland Cement Concrete (PCC) pavement recommendations tabulated below are based on methods presented in the Portland Cement Association (PCA) design manual (PCA, 1984). 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 = 5**

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

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

## 9.3 TRASH ENCLOSURES

Trash enclosures and the associated stress pads should be supported on at least 8 inches of Portland cement concrete (PCC) over at least 6 inches of Class 2 aggregate base, where the aggregate base should be compacted to 95 percent relative compaction. The top 6 inches of the underlying subgrade should be moisture conditioned and compacted according to the “Compaction” section of this report. The compressive strength and construction details should be consistent with the above recommendations for PCC pavements.

## 9.4 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 10: RETAINING WALLS

## 10.1 STATIC LATERAL EARTH PRESSURES

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

**Table 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

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

## 10.2 SEISMIC LATERAL EARTH PRESSURES

The 2016 California Building Code (CBC) states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. We developed seismic earth pressures for the proposed basement using interim recommendations generally based on refinement of the Mononobe-Okabe method (Lew et al., SEAOC 2010). Because the walls are greater than 12 feet in height, and peak ground accelerations are greater than 0.40g, we checked the result of the total seismic increment when added to the recommended active earth pressure against the recommended fixed (restrained) wall earth pressures. Because the wall is restrained, or will act as a restrained wall, and will be designed for 45 pcf (equivalent fluid pressure) plus a uniform earth pressure of 8H psf, based on current recommendations for seismic earth pressures, it appears that active earth pressures plus a seismic increment do not exceed the fixed wall earth pressures. Therefore, an additional seismic increment above the design earth pressures is not required as long as the walls are designed for the restrained wall earth pressures recommended above in accordance with the CBC.

## 10.3 WALL DRAINAGE

### 10.3.1 At-Grade Site Walls

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

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

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

### 10.3.2 Below-Grade Walls

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

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

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

## 10.4 BACKFILL

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls with a soil PI less than 20 should be compacted to at least 95 percent relative compaction using light compaction equipment. If the soil's PI is 20 or greater, expansive soil criteria should be used as discussed in the "Compaction" section of this report. Where no surface improvements are planned, backfill should be compacted to at least 90 percent for soils with a PI less than 20. Expansive soil criteria should be followed for soils with a PI of 20 or greater. If heavy compaction equipment is used, the walls should be temporarily braced.

## 10.5 FOUNDATIONS

The basement retaining walls may be supported on continuous spread footings designed in accordance with the recommendations presented in the "Foundations" section of this report.

# SECTION 11: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Sunrise Senior Living specifically to support the design of the Sunrise Senior Living of Redwood City project in Redwood City, 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.

Sunrise Senior Living may have provided Cornerstone with plans, reports and other documents prepared by others. Sunrise Senior Living 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.

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Vicinity Map

**CORNERSTONE**
**EARTH GROUP**

## Sunrise Senior Living of Redwood City
## Redwood City, CA

| | |
|---|---|
| Project Number | 935-1-2 |
| Figure Number | Figure 1 |
| Date | November 2016 |
| Drawn By | RRN |

## Legend

⊕ Approximate location of exploratory boring (EB)

▲ Approximate location of cone penetration test (CPT)

0 50 100

APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 4/5/2016
Overlay by HPI Architecture, Conceptual Site Plan - A1.0, 12/5/2016

Project Number 935-1-2

Figure Number Figure 2

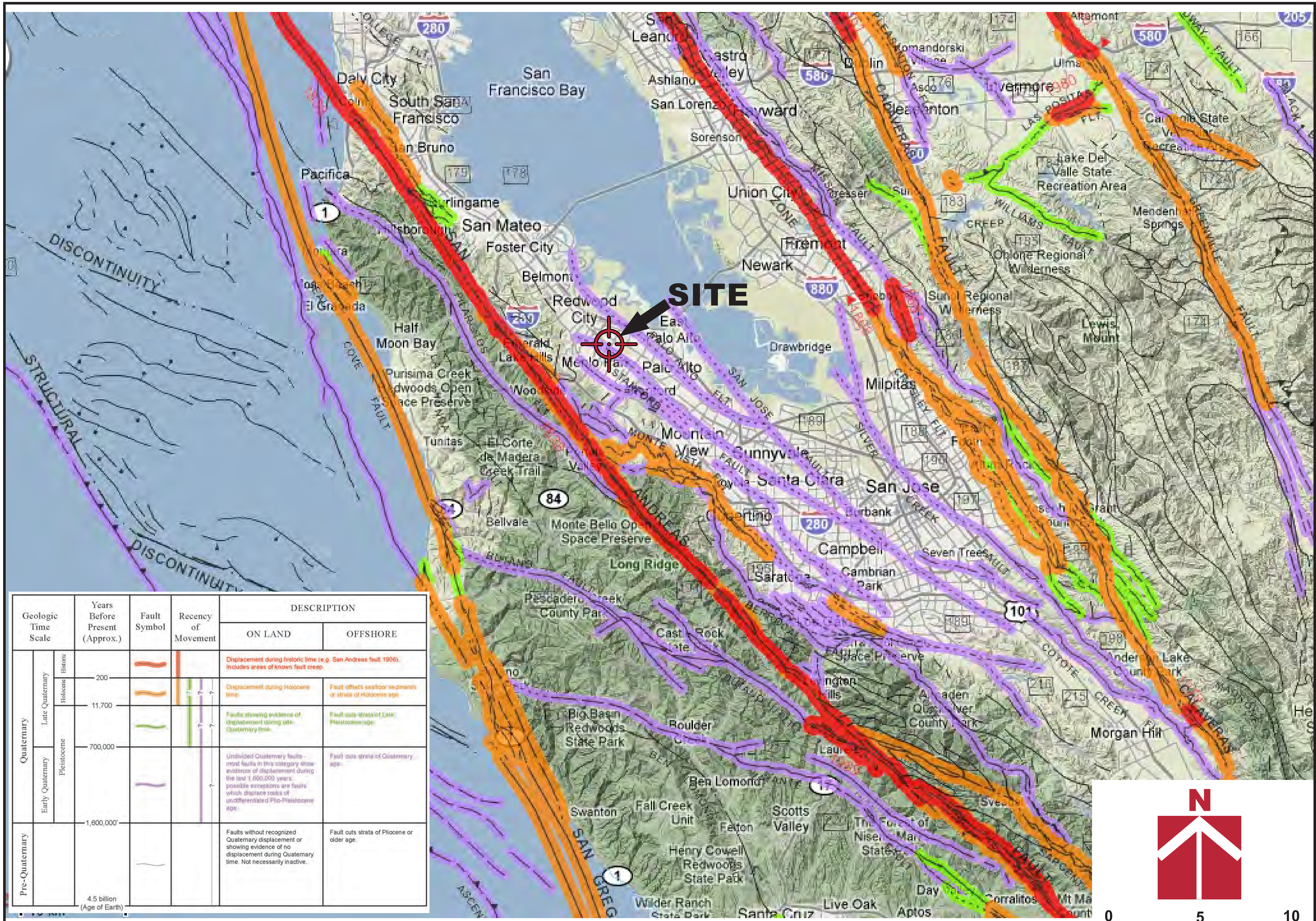
Drawn By RRN

Date December 2016

## Site Plan

## Sunrise Senior Living of Redwood City
Redwood City, CA

CORNERSTONE EARTH GROUP



| Geologic Time Scale | | | Years Before Present (Approx.) | Fault Symbol | Recency of Movement | DESCRIPTION | |
|---|---|---|---|---|---|---|---|
| | | | | | | ON LAND | 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)

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Regional Fault Map

**Sunrise Senior Living of Redwood City**
**Redwood City, CA**

Project Number 935-1-2

Figure Number Figure 3

Date November 2016

Drawn By RRN

**CORNERSTONE**
**EARTH GROUP**

**FIGURE 4A**

**CPT NO. 1**

## PROJECT/CPT DATA

| | |
|---|---|
| Project Title | **Sunrise Senior Living Redwood City** |
| Project No. | **935-1-2** |
| Project Manager | **MJS** |

### SEISMIC PARAMETERS

| | | |
|---|---|---|
| Controlling Fault | **San Andreas** | |
| Earthquake Magnitude (Mw) | **7.9** | |
| PGA (Amax) | **0.672** | (g) |

### SITE SPECIFIC PARAMETERS

| | |
|---|---|
| Ground Water Depth at Time of Drilling (feet) | **23.5** |
| Design Water Depth (feet) | **20** |
| Ave. Unit Weight Above GW (pcf) | **125** |
| Ave. Unit Weight Below GW (pcf) | **120** |

## CPT ANALYSIS RESULTS

**DRY SAND SETTLEMENT FROM** 20 **FEET**

0.00 **(Inches)**

**LIQUEFACTION SETTLEMENT FROM** 50 **FEET**

0.71 **(Inches)**

**TOTAL SEISMIC SETTLEMENT** 0.7 **INCHES**

### POTENTIAL LATERAL DISPLACEMENT

**LDI**[2] 0.00 **L/H** 80.0

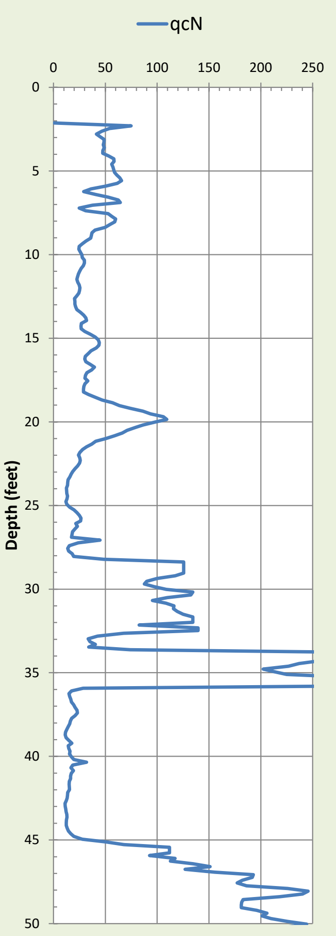
**LDI**[1]$_{\text{Corrected for Distance}}$ 0.00 (4 < L/H < 40)

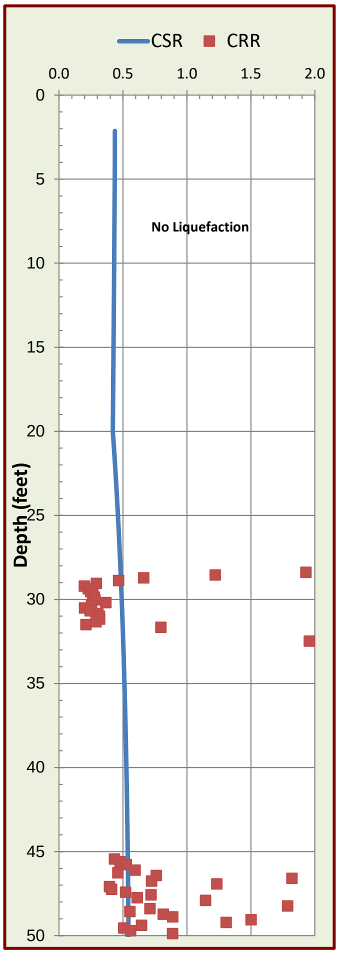
**EXPECTED RANGE OF DISPLACEMENT**

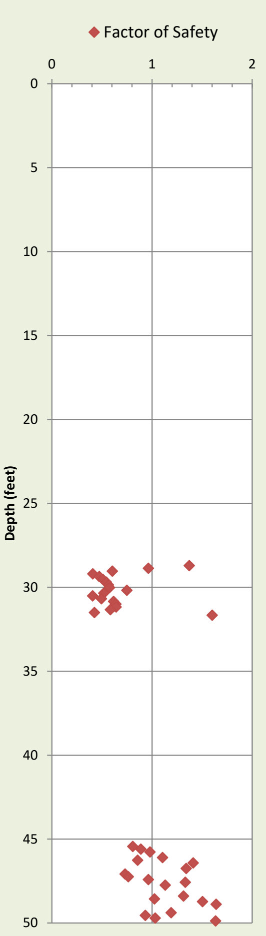
**0.0** **to** **0.0** **feet**

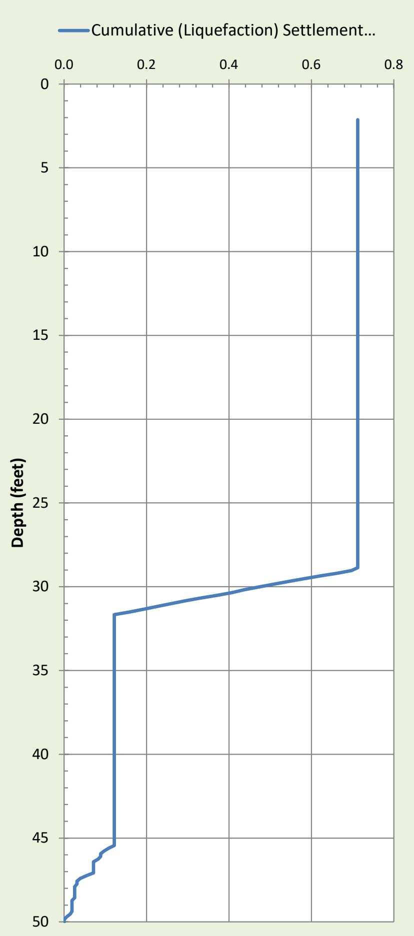
[1] Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE
EARTH GROUP

**FIGURE 4B**

**CPT NO. 2**

## PROJECT/CPT DATA

| | |
|---|---|
| Project Title | **Sunrise Senior Living Redwood City** |
| Project No. | **935-1-2** |
| Project Manager | **MJS** |

### SEISMIC PARAMETERS

| | | |
|---|---|---|
| Controlling Fault | **San Andreas** | |
| Earthquake Magnitude (Mw) | **7.9** | |
| PGA (Amax) | **0.672** | (g) |

### SITE SPECIFIC PARAMETERS

| | |
|---|---|
| Ground Water Depth at Time of Drilling (feet) | **27** |
| Design Water Depth (feet) | **20** |
| Ave. Unit Weight Above GW (pcf) | **125** |
| Ave. Unit Weight Below GW (pcf) | **120** |

## CPT ANALYSIS RESULTS

**DRY SAND SETTLEMENT FROM** 20 **FEET**

0.00 **(Inches)**

**LIQUEFACTION SETTLEMENT FROM** 50 **FEET**

0.02 **(Inches)**

**TOTAL SEISMIC SETTLEMENT** 0.0 **INCHES**

### POTENTIAL LATERAL DISPLACEMENT

**LDI**$^2$ 0.00 **L/H** 80.0

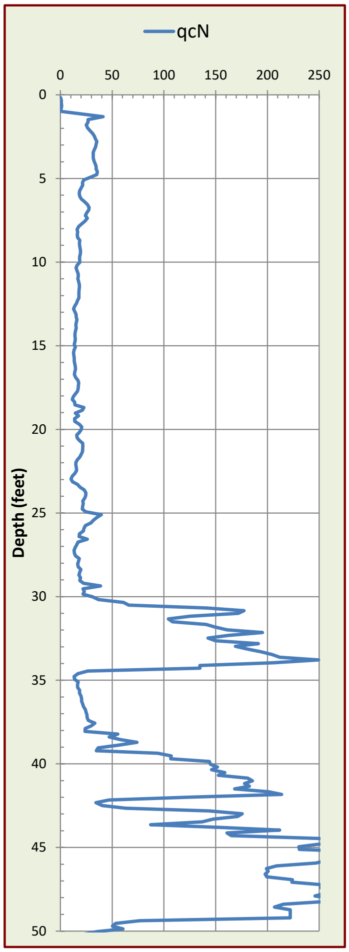
**LDI**$^1_{\text{Corrected for Distance}}$ 0.00 (4 < L/H < 40)

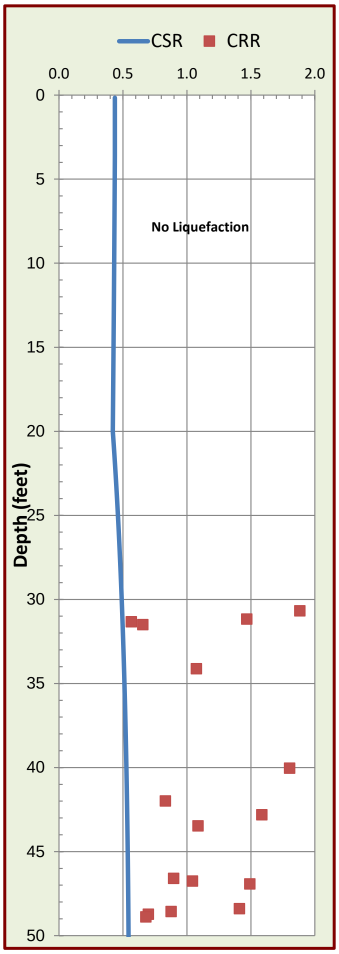
**EXPECTED RANGE OF DISPLACEMENT**

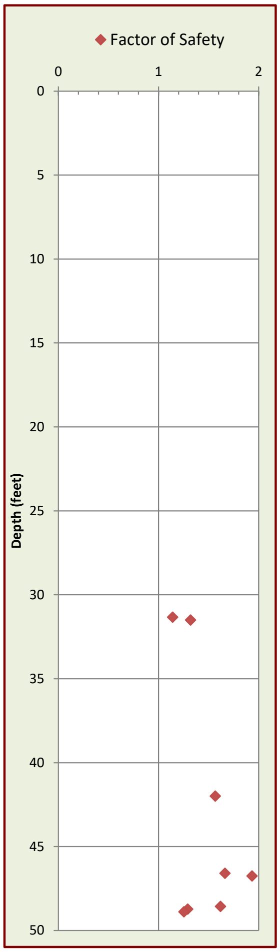
**0.0** **to** **0.0** **feet**

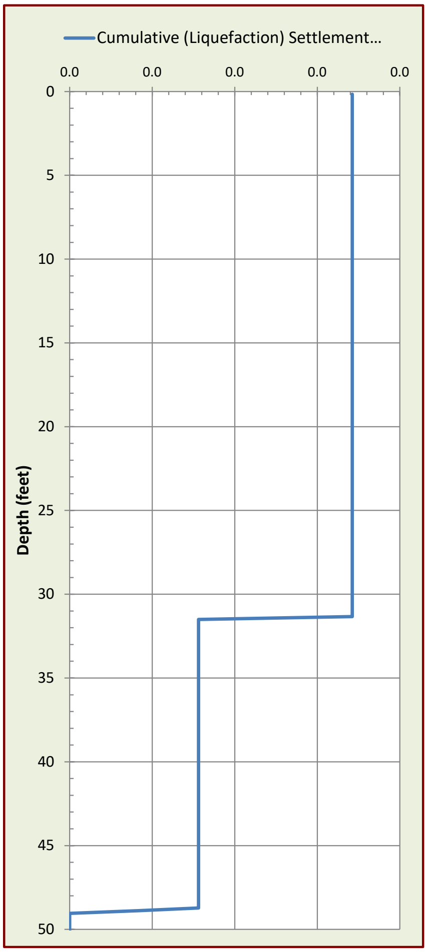
[1] Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE
EARTH GROUP

**FIGURE 4C**

**CPT NO. 3**

## PROJECT/CPT DATA

Project Title **Sunrise Senior Living Redwood City**

Project No. **935-1-2**

Project Manager **MJS**

### SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.9**

PGA (Amax) **0.672** (g)

### SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **30**

Design Water Depth (feet) **20**

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

Ave. Unit Weight Below GW (pcf) **120**

## CPT ANALYSIS RESULTS

**DRY SAND SETTLEMENT FROM** 20 **FEET**

0.03 **(Inches)**

**LIQUEFACTION SETTLEMENT FROM** 50 **FEET**

0.11 **(Inches)**

**TOTAL SEISMIC SETTLEMENT** 0.1 **INCHES**

### POTENTIAL LATERAL DISPLACEMENT

**LDI**[2] 0.00 **L/H** 80.0

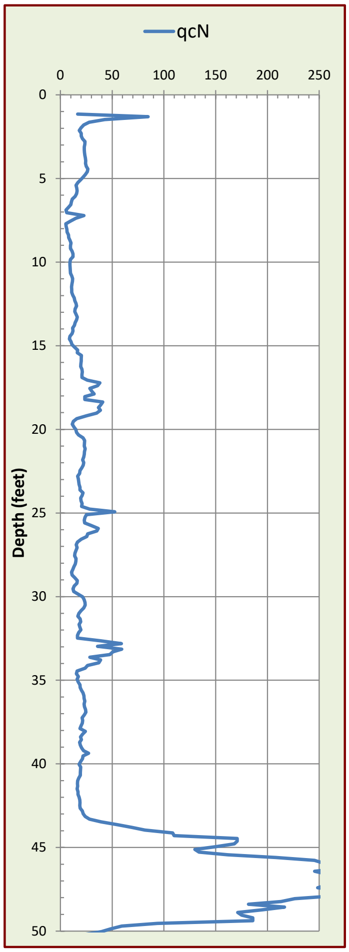
**LDI**[1]**Corrected for Distance** 0.00 (4 < L/H < 40)

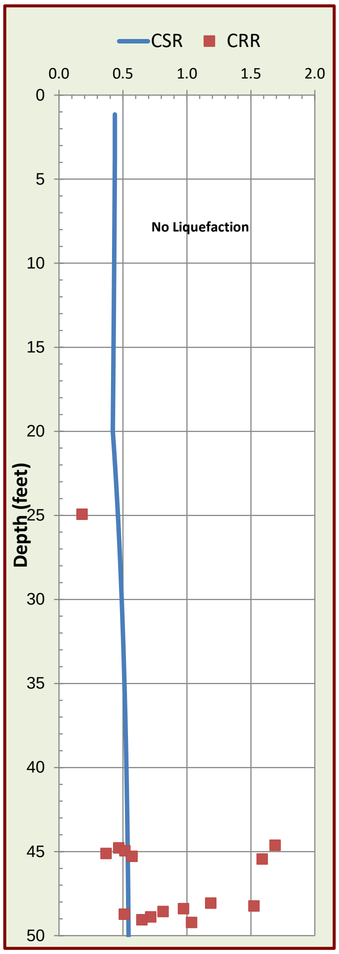
**EXPECTED RANGE OF DISPLACEMENT**

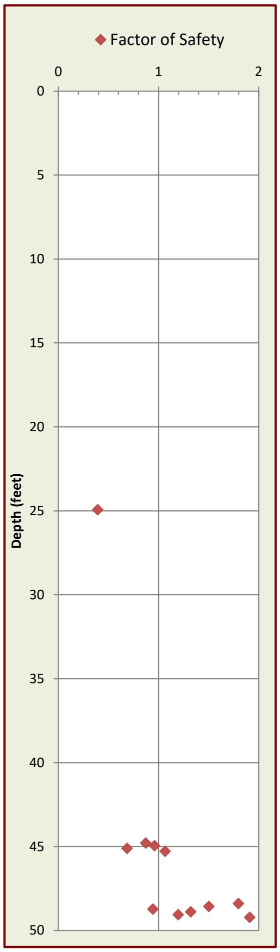
**0.0** **to** **0.0** **feet**

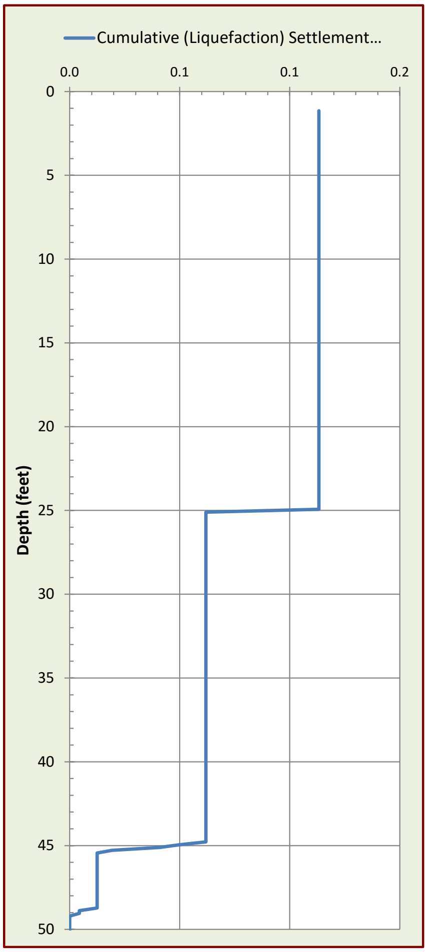
[1] Not Valid for L/H Values < 4 and > 40.

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









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**EARTH GROUP**

**FIGURE** **4D**

**CPT NO.** **4**

## PROJECT/CPT DATA

Project Title **Sunrise Senior Living Redwood City**

Project No. **935-1-2**

Project Manager **MJS**

### SEISMIC PARAMETERS

Controlling Fault **San Andreas**

Earthquake Magnitude (Mw) **7.9**

PGA (Amax) **0.672** (g)

### SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet) **27**

Design Water Depth (feet) **20**

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

Ave. Unit Weight Below GW (pcf) **120**

## CPT ANALYSIS RESULTS

**DRY SAND SETTLEMENT FROM** 20 **FEET**

0.01 **(Inches)**

**LIQUEFACTION SETTLEMENT FROM** 50 **FEET**

0.01 **(Inches)**

**TOTAL SEISMIC SETTLEMENT** 0.0 **INCHES**

### POTENTIAL LATERAL DISPLACEMENT

**LDI[2]** 0.00 **L/H** 80.0

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

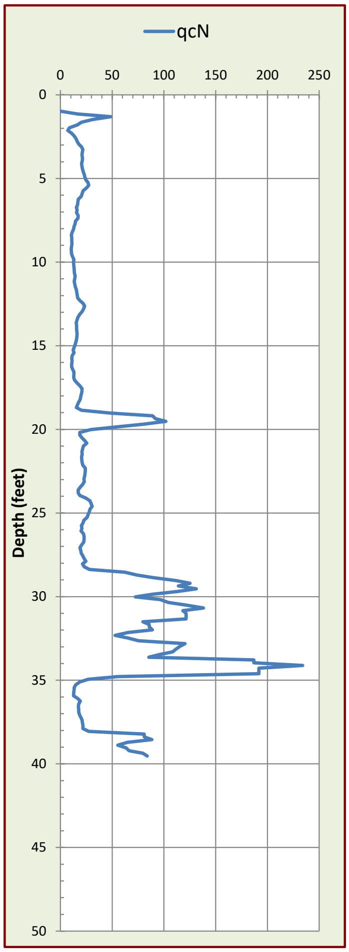
**EXPECTED RANGE OF DISPLACEMENT**

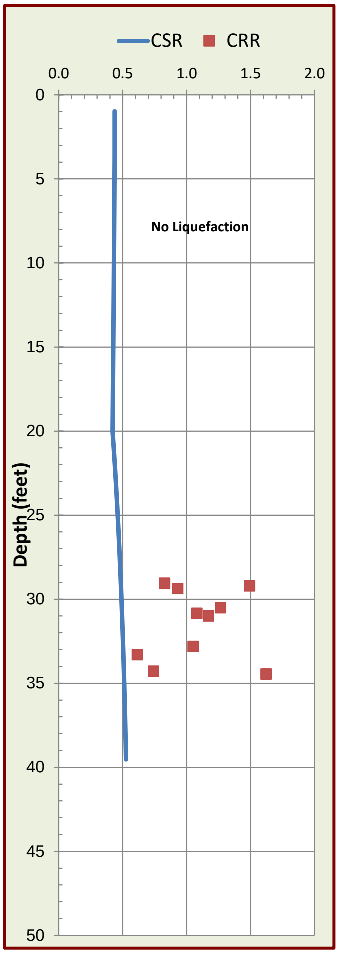
**0.0** **to** **0.0** **feet**

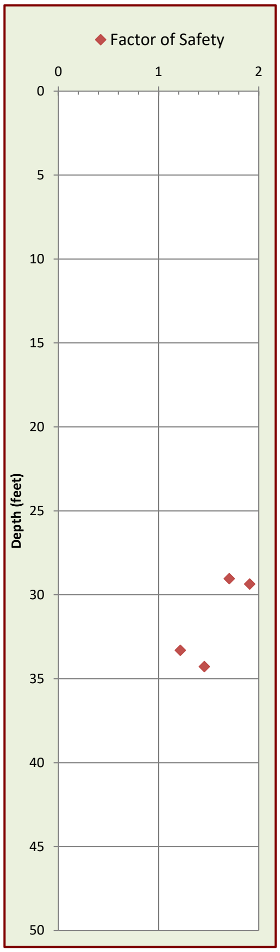
---

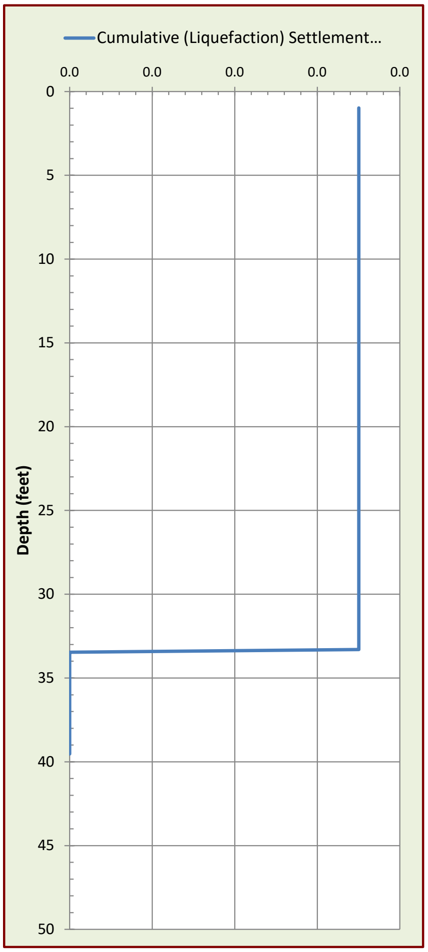
[1] Not Valid for L/H Values < 4 and > 40.

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









CORNERSTONE EARTH GROUP

**FIGURE 4E**

**CPT NO. 5**

## PROJECT/CPT DATA

| | |
|---|---|
| Project Title | **Sunrise Senior Living Redwood City** |
| Project No. | **935-1-2** |
| Project Manager | **MJS** |

### SEISMIC PARAMETERS

| | | |
|---|---|---|
| Controlling Fault | **San Andreas** | |
| Earthquake Magnitude (Mw) | **7.9** | |
| PGA (Amax) | **0.672** | (g) |

### SITE SPECIFIC PARAMETERS

| | |
|---|---|
| Ground Water Depth at Time of Drilling (feet) | **27** |
| Design Water Depth (feet) | **20** |
| Ave. Unit Weight Above GW (pcf) | **125** |
| Ave. Unit Weight Below GW (pcf) | **120** |

## CPT ANALYSIS RESULTS

**DRY SAND SETTLEMENT FROM** 20 **FEET**

0.00 **(Inches)**

**LIQUEFACTION SETTLEMENT FROM** 50 **FEET**

0.06 **(Inches)**

**TOTAL SEISMIC SETTLEMENT** 0.1 **INCHES**

### POTENTIAL LATERAL DISPLACEMENT

**LDI**[2] 0.00 **L/H** 80.0

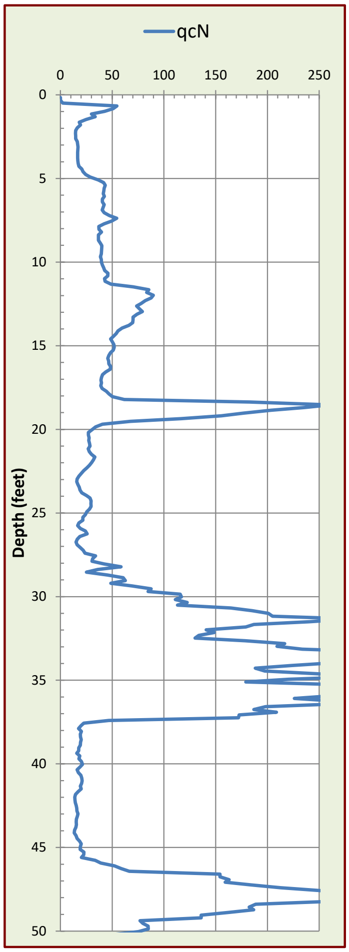
**LDI**[1]**Corrected for Distance** 0.00 (4 < L/H < 40)

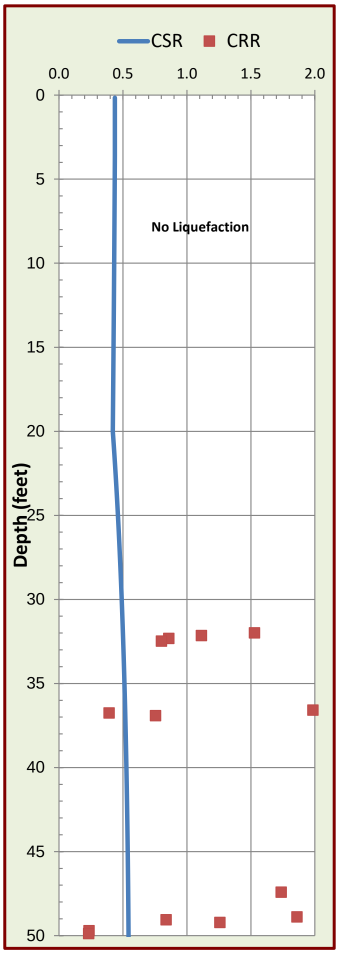
**EXPECTED RANGE OF DISPLACEMENT**

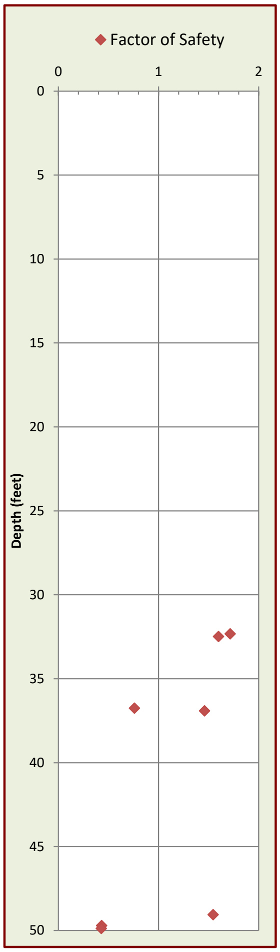
**0.0** **to** **0.0** **feet**

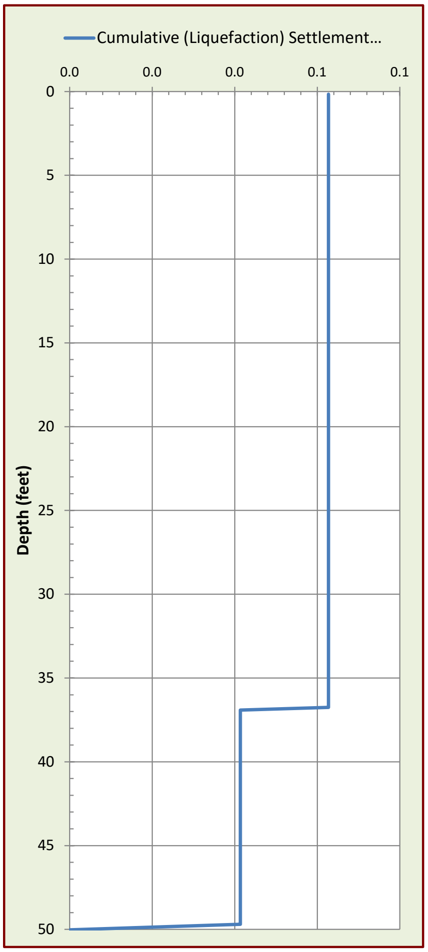
[1] Not Valid for L/H Values < 4 and > 40.

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









## APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment and 20-ton truck-mounted Cone Penetration Test equipment. Three 8-inch-diameter exploratory borings were drilled on November 10 and 11, 2016 to depths of approximately 41 to 50 feet. Five CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on November 8, 2016, to depths ranging from approximately 40 to 100 feet. The approximate locations of exploratory borings and CPTs are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil, are included as part of this appendix.

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

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

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

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

# UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)

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

## OTHER MATERIAL SYMBOLS

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

## SAMPLER TYPES

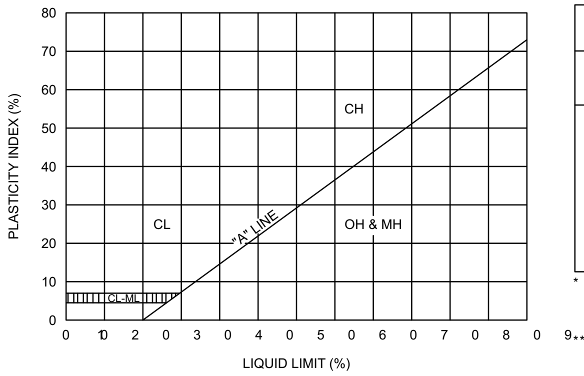
- SPT
- Modified California (2.5" I.D.)
- Rock Core
- Shelby Tube
- No Recovery
- Grab Sample

## ADDITIONAL TESTS

CA - CHEMICAL ANALYSIS (CORROSIVITY)
CD - CONSOLIDATED DRAINED TRIAXIAL
CN - CONSOLIDATION
CU - CONSOLIDATED UNDRAINED TRIAXIAL
DS - DIRECT SHEAR
PP - POCKET PENETROMETER (TSF)
(3.0) - (WITH SHEAR STRENGTH IN KSF)
RV - R-VALUE
SA - SIEVE ANALYSIS: % PASSING #200 SIEVE
▼ - WATER LEVEL

PI - PLASTICITY INDEX
SW SWELL TEST
TC - CYCLIC TRIAXIAL
TV - TORVANE SHEAR
UC - UNCONFINED COMPRESSION
(1.5) - (WITH SHEAR STRENGTH IN KSF)
UU - UNCONSOLIDATED UNDRAINED TRIAXIAL

### PLASTICITY CHART



### PENETRATION RESISTANCE
(RECORDED AS BLOWS / FOOT)

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

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

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

CORNERSTONE
EARTH GROUP

## LEGEND TO SOIL DESCRIPTIONS

Figure Number
A-1

# BORING NUMBER EB-1

CORNERSTONE
EARTH GROUP

**PROJECT NAME** Sunrise Senior Living of Redwood City
**PROJECT NUMBER** 935-1-2
**PROJECT LOCATION** Redwood City, CA

**DATE STARTED** 11/10/16 **DATE COMPLETED** 11/10/16
**GROUND ELEVATION** **BORING DEPTH** 50 ft.
**DRILLING CONTRACTOR** Exploration Geoservices, Inc.
**LATITUDE** **LONGITUDE**
**DRILLING METHOD** Mobile B-40, 8 inch Hollow-Stem Auger
**GROUND WATER LEVELS:**
**LOGGED BY** DL
▽ **AT TIME OF DRILLING** 27 ft.
**NOTES**
▼ **AT END OF DRILLING** 23.5 ft.

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

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

| 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 |
|---|---|---|---|---|---|---|---|
| 3½ inches asphalt concrete over 4 inches aggregate base | | | | | | | |
| **Fat Clay (CH)**<br>hard, moist, dark brown, some fine sand, high plasticity<br>Liquid Limit = 53, Plastic Limit = 18 | 43 | MC-1B | 105 | 15 | 35 | | >4.5 ○ |
| **Sandy Silt (ML)**<br>hard, moist, light brown, fine sand, low plasticity | 17 | MC-2B | 89 | 13 | | | >4.5 ○ |
| **Sandy Lean Clay (CL)**<br>hard, moist, brown, fine sand, low plasticity | 13 | MC-3B | 103 | 14 | | | >4.5 ○ |
| **Silty Sand (SM)**<br>medium dense, moist, brown, fine to medium sand | 25 | MC-4B | 90 | 10 | | 48 | |
| **Lean Clay with Sand (CL)**<br>hard, moist, brown, fine sand, moderate plasticity | 22 | MC-5B | 112 | 15 | | | >4.5 ○ |
| becomes stiff | | ST-6 | 93 | 29 | | | ●▲ |

ELEVATION (ft) DEPTH (ft) SYMBOL

0 5 10 15 20 25

*Continued Next Page*

**PROJECT NAME** Sunrise Senior Living of Redwood City

**PROJECT NUMBER** 935-1-2

**PROJECT LOCATION** Redwood City, CA

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ELEVATION (ft) | DEPTH (ft) | SYMBOL

## DESCRIPTION

∇

**Poorly Graded Sand with Silt (SP-SM)**
loose, moist, brown and gray, fine to coarse sand, some fine gravel

becomes dense

30

becomes medium dense

35

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

40

45

**Poorly Graded Sand with Clay (SP-SC)**
dense, wet, brown and gray, fine to coarse sand, some fine gravel

50

Bottom of Boring at 50.0 feet.

55

| N-Value (uncorrected) blows per foot | SAMPLES TYPE AND NUMBER | DRY UNIT WEIGHT PCF | NATURAL MOISTURE CONTENT | PLASTICITY INDEX, % | PERCENT PASSING No. 200 SIEVE |
|---|---|---|---|---|---|
| 11 | MC-7B | 104 | 21 | | 6 |
| 42 | SPT | | | | |
| 31 | SPT | | | | |
| 23 | SPT | | | | |
| 22 | SPT | | | | |
| 16 | MC-12B | 102 | 20 | | |
| 23 | MC-13B | 99 | 24 | | |
| 63 | MC-14 | | 18 | | |

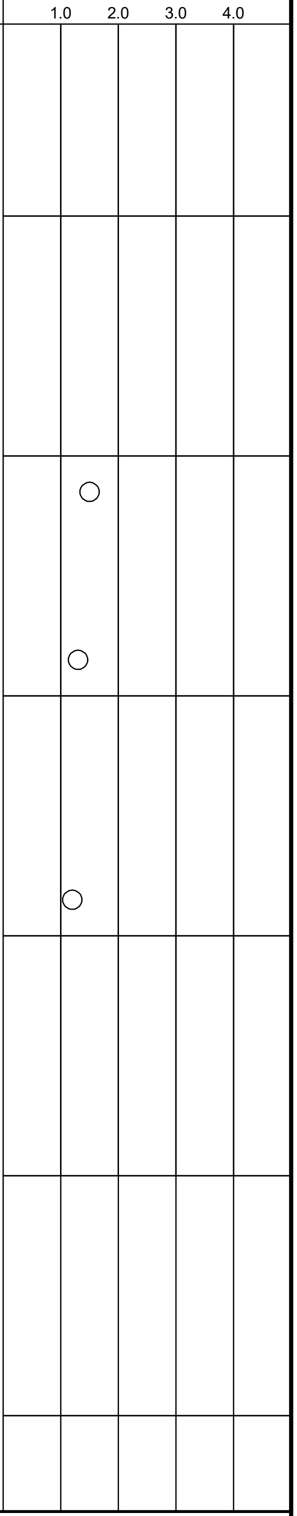
UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL



CORNERSTONE EARTH GROUP

# BORING NUMBER EB-2

PAGE 1 OF 2

**PROJECT NAME** Sunrise Senior Living of Redwood City
**PROJECT NUMBER** 935-1-2
**PROJECT LOCATION** Redwood City, CA
**DATE STARTED** 11/11/16 **DATE COMPLETED** 11/11/16
**GROUND ELEVATION** **BORING DEPTH** 46.5 ft.
**DRILLING CONTRACTOR** Exploration Geoservices, Inc.
**LATITUDE** **LONGITUDE**
**DRILLING METHOD** Mobile B-56, 8 inch Hollow-Stem Auger
**GROUND WATER LEVELS:**
**LOGGED BY** AA
▽ **AT TIME OF DRILLING** 27 ft.
**NOTES**
▼ **AT END OF DRILLING** 27 ft.

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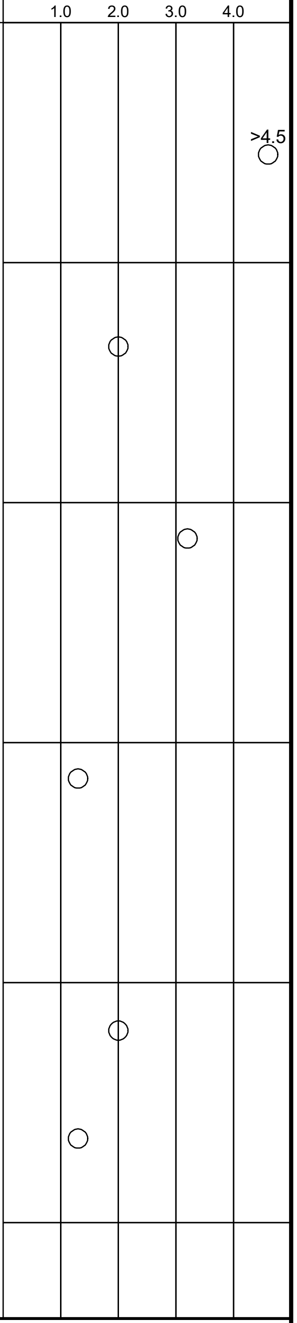
| ELEVATION (ft) | DEPTH (ft) | SYMBOL | DESCRIPTION | N-Value (uncorrected) blows per foot | SAMPLES TYPE AND NUMBER | DRY UNIT WEIGHT PCF | NATURAL MOISTURE CONTENT | PLASTICITY INDEX, % | PERCENT PASSING No. 200 SIEVE |
|---|---|---|---|---|---|---|---|---|---|
| | 0 | | 2 inches asphalt concrete over 4 inches aggregate base | | | | | | |
| | | | **Fat Clay (CH)**<br>hard, moist, dark brown, some fine sand, high plasticity | 57 | MC-1B | 106 | 17 | | |
| | 5 | | **Lean Clay with Sand (CL)**<br>stiff, moist, brown, fine sand, low to moderate plasticity | 9 | MC-2 | | 22 | | |
| | 10 | | **Lean Clay (CL)**<br>very stiff, moist, brown, some fine sand, low to moderate plasticity | 8 | MC-3B | 102 | 17 | | |
| | 15 | | **Lean Clay with Sand (CL)**<br>stiff, moist, brown with gray mottles, fine sand, low to moderate plasticity | 14 | MC-4B | 95 | 24 | | |
| | 20<br>25 | | **Lean Clay (CL)**<br>stiff, moist, brown with gray mottles, some fine sand, moderate plasticity | <br>18 | ST<br>MC-6B | <br>98 | <br>28 | | |

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



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# BORING NUMBER EB-2

PAGE 2 OF 2

CORNERSTONE EARTH GROUP

**PROJECT NAME** Sunrise Senior Living of Redwood City
**PROJECT NUMBER** 935-1-2
**PROJECT LOCATION** Redwood City, CA

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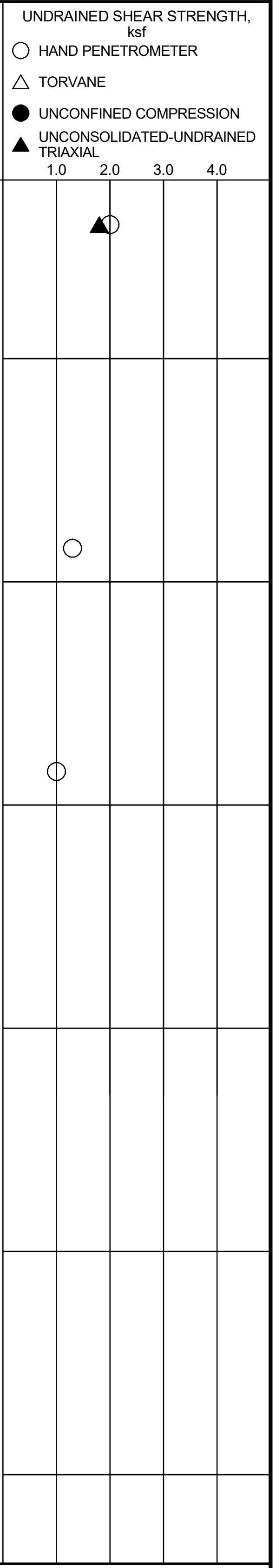
| DESCRIPTION | N-Value (uncorrected) blows per foot | SAMPLES TYPE AND NUMBER | DRY UNIT WEIGHT PCF | NATURAL MOISTURE CONTENT | PLASTICITY INDEX, % | PERCENT PASSING No. 200 SIEVE |
|---|---|---|---|---|---|---|
| **Sandy Lean Clay (CL)**<br>stiff, moist, brown, fine to medium sand, low plasticity | | ST-7 | 115 | 17 | | |
| **Clayey Sand (SC)**<br>medium dense, moist, brown, fine to coarse sand, some fine gravel | 20 | MC-8B | 122 | 14 | | |
| **Poorly Graded Sand with Clay and Gravel (SP-SC)**<br>dense, moist, brown and gray, fine to coarse sand, fine subangular to subrounded gravel | 62 | MC-9B | 127 | 12 | | |
| **Lean Clay with Sand (CL)**<br>stiff, moist, brown, fine sand, moderate plasticity | 15 | MC-10B | 114 | 18 | | |
| **Silty Sand (SM)**<br>medium dense, wet, brown, fine to medium sand | 33 | MC-11 | | 17 | | |
| **Poorly Graded Sand with Gravel (SP)**<br>dense, wet, brown and gray, fine to coarse sand, some fine gravel | 37 | SPT | | | | |

Bottom of Boring at 46.5 feet.



# BORING NUMBER EB-3
PAGE 1 OF 2

**CORNERSTONE EARTH GROUP**

**PROJECT NAME** Sunrise Senior Living of Redwood City
**PROJECT NUMBER** 935-1-2
**PROJECT LOCATION** Redwood City, CA
**DATE STARTED** 11/10/16 **DATE COMPLETED** 11/10/16
**GROUND ELEVATION** ____ **BORING DEPTH** 41 ft.
**DRILLING CONTRACTOR** Exploration Geoservices, Inc.
**LATITUDE** ____ **LONGITUDE** ____
**DRILLING METHOD** Mobile B-40, 8 inch Hollow-Stem Auger
**GROUND WATER LEVELS:**
**LOGGED BY** DL
▽ **AT TIME OF DRILLING** 30 ft.
**NOTES** ____
▼ **AT END OF DRILLING** 30 ft.

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ELEVATION (ft) | DEPTH (ft) | SYMBOL

## DESCRIPTION

DEPTH (ft): 0, 5, 10, 15, 20, 25

2 inches asphalt concrete over 4 inches aggregate base

**Fat Clay (CH)**
hard, moist, dark brown, some fine sand, high plasticity

**Lean Clay with Sand (CL)**
very stiff, moist, brown, fine to medium sand, low plasticity

**Sandy Lean Clay (CL)**
stiff, moist, brown, fine to medium sand, low plasticity

**Lean Clay with Sand (CL)**
stiff, moist, brown, fine to medium sand, low plasticity

**Silty Sand (SM)**
medium dense, moist, brown, fine to medium sand

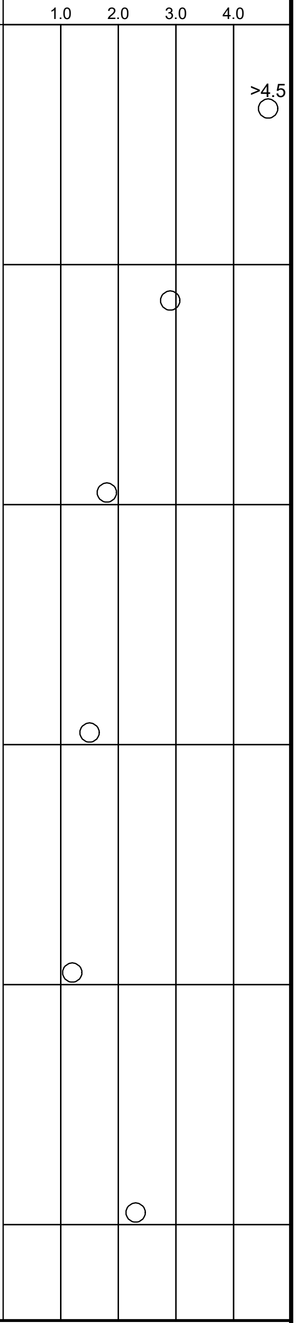
**Lean Clay (CL)**
stiff, moist, brown, some fine sand, moderate plasticity

**Lean Clay with Sand (CL)**
very stiff, moist, gray and brown mottled, fine to medium sand, low to moderate plasticity

*Continued Next Page*

| N-Value (uncorrected) blows per foot | SAMPLES TYPE AND NUMBER | DRY UNIT WEIGHT PCF | NATURAL MOISTURE CONTENT | PLASTICITY INDEX, % | PERCENT PASSING No. 200 SIEVE |
|---|---|---|---|---|---|
| 12 | MC-1B | 107 | 18 | | |
| 11 | MC-2B | 102 | 19 | | |
| 17 | MC-3B | 90 | 33 | | |
| 13 | MC-4B | 96 | 25 | | |
| 13 | MC-5B | 89 | 31 | | |
| 17 | MC-6B | 107 | 20 | | |

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



# BORING NUMBER EB-3

PAGE 2 OF 2

CORNERSTONE
EARTH GROUP

**PROJECT NAME** Sunrise Senior Living of Redwood City

**PROJECT NUMBER** 935-1-2

**PROJECT LOCATION** Redwood City, CA

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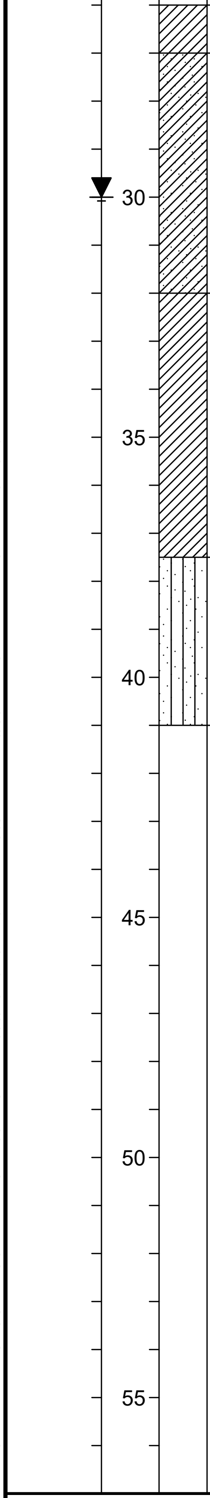


## DESCRIPTION

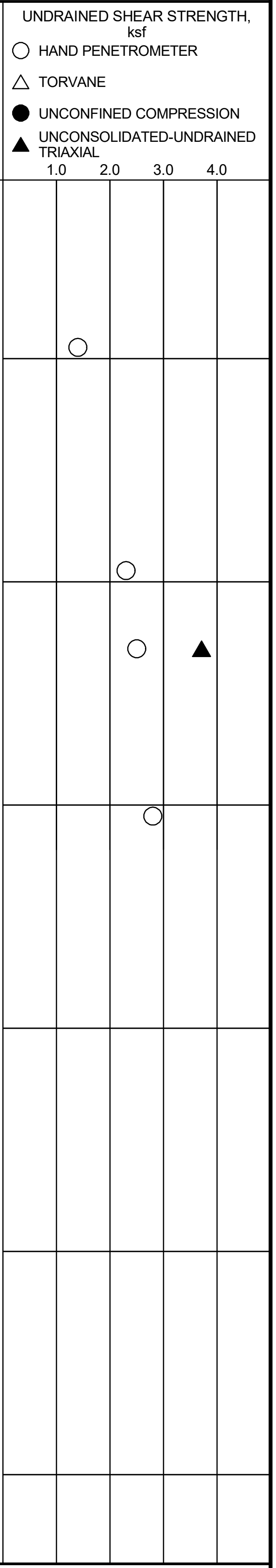
**Sandy Lean Clay (CL)**
stiff, moist, brown, fine to medium sand, low plasticity

**Lean Clay with Sand (CL)**
very stiff, moist, brown and gray mottled, fine sand, moderate plasticity

**Sandy Silt (ML)**
stiff, moist, brown and gray mottled, fine sand, low plasticity

Bottom of Boring at 41.0 feet.

| N-Value (uncorrected) blows per foot | SAMPLES TYPE AND NUMBER | DRY UNIT WEIGHT PCF | NATURAL MOISTURE CONTENT | PLASTICITY INDEX, % | PERCENT PASSING No. 200 SIEVE |
|---|---|---|---|---|---|
| 27 | MC-7B | 115 | 14 | | |
| 34 | MC-8B | 109 | 19 | | |
| | ST-9 | 113 | 18 | | |
| 41 | MC-10B | 98 | 24 | | |



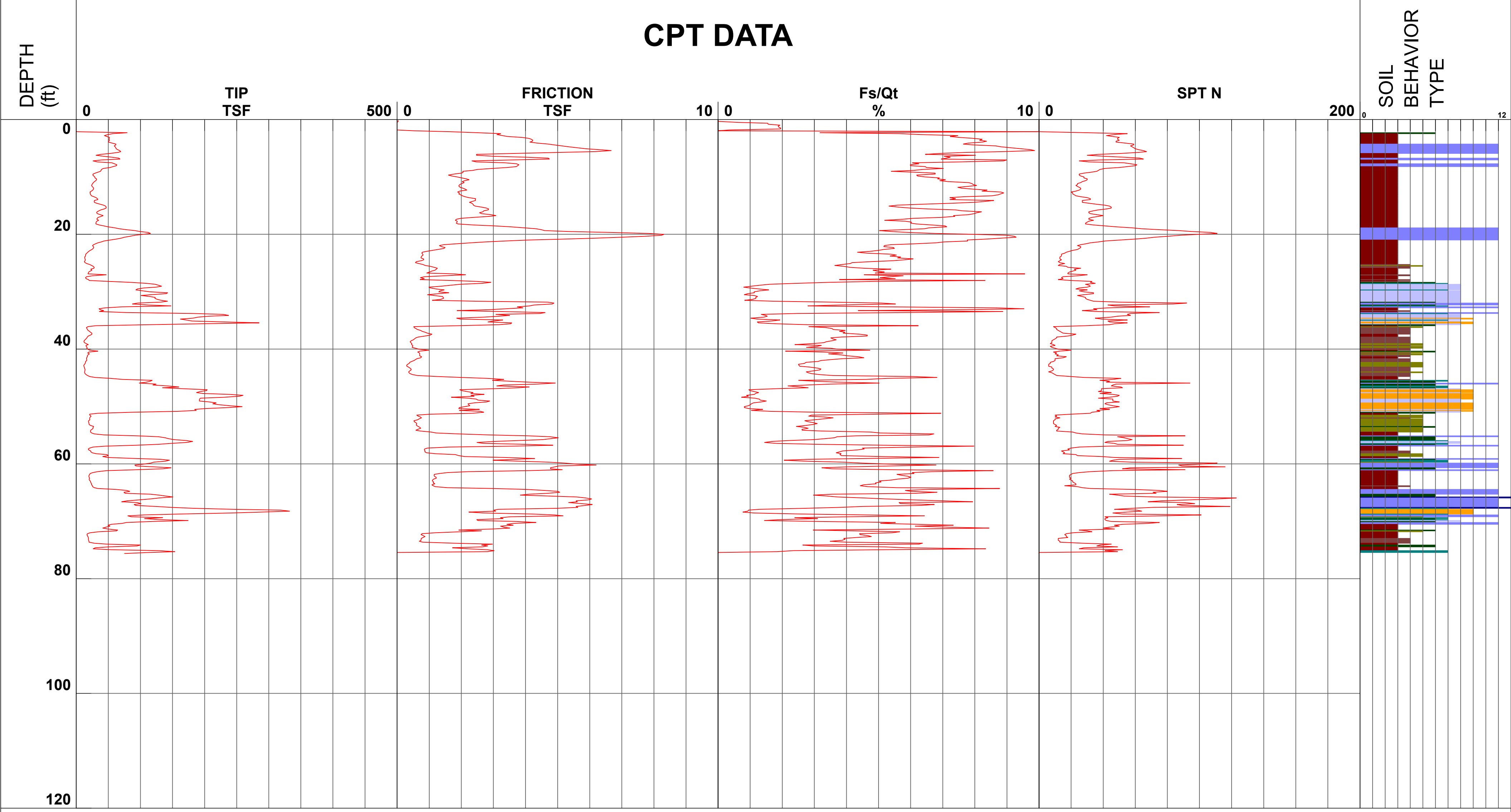
Middle Earth
Geo Testing Inc.

# Cornerstone Earth Group

Project Sunrise Senior Living
Job Number 935-1-2
Hole Number CPT-01
EST GW Depth During Test 21.20 ft
Operator KK-RB
Cone Number DDG1379
Date and Time 11/8/2016 8:52:41 AM
Filename SDF(341).cpt
GPS
Maximum Depth 75.62 ft

Net Area Ratio .8

## CPT DATA



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

Cone Size 10cm squared

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

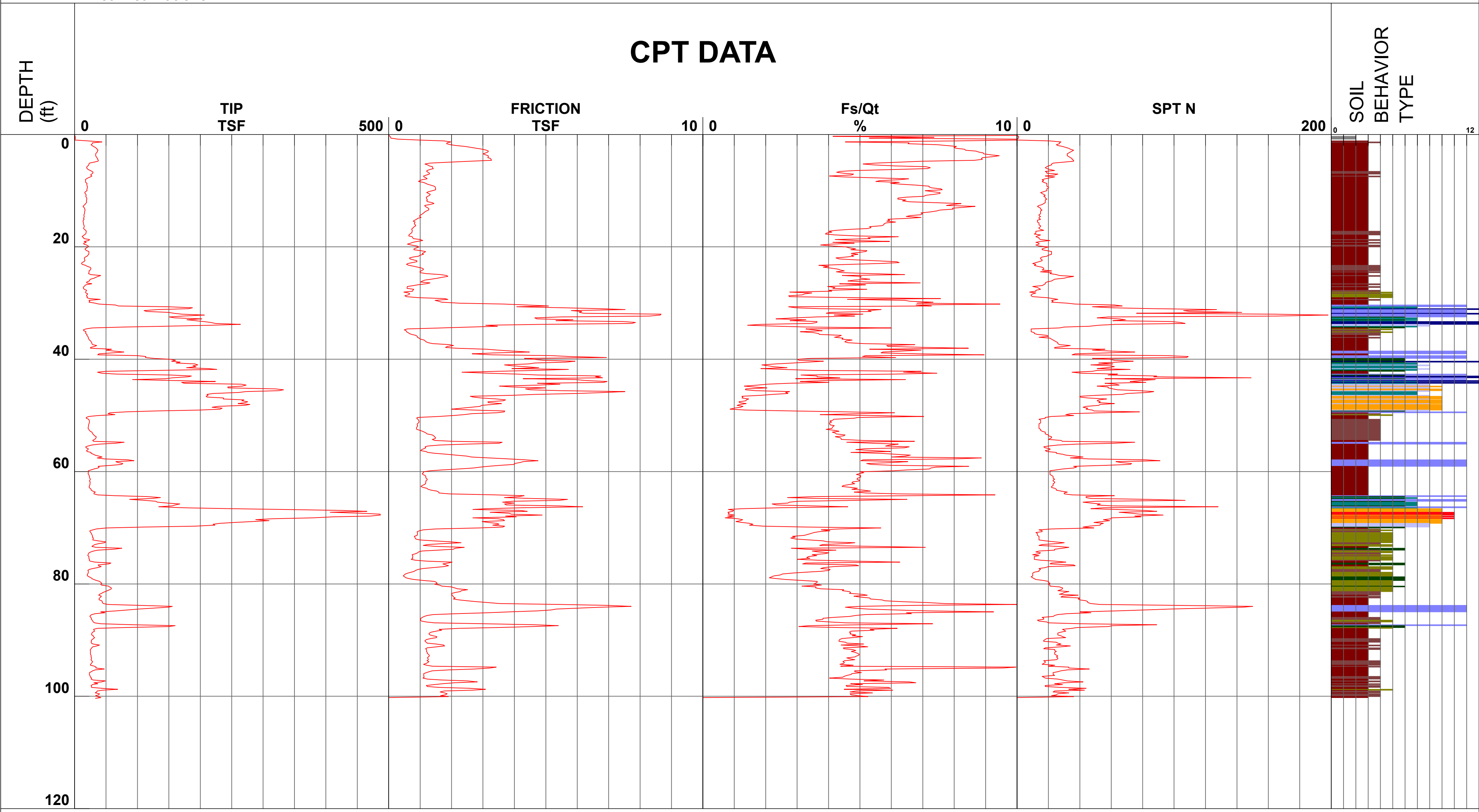
# Cornerstone Earth Group

Middle Earth
Geo Testing Inc.

| | | | | | |
|---|---|---|---|---|---|
| Project | Sunrise Senior Living | Operator | KK-RB | Filename | SDF(343).cpt |
| Job Number | 935-1-2 | Cone Number | DDG1379 | GPS | |
| Hole Number | CPT-02 | Date and Time | 11/8/2016 12:27:21 PM | Maximum Depth | 100.39 ft |
| EST GW Depth During Test | | 22.00 ft | | | |

Net Area Ratio .8

## CPT DATA



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

Cone Size 10cm squared

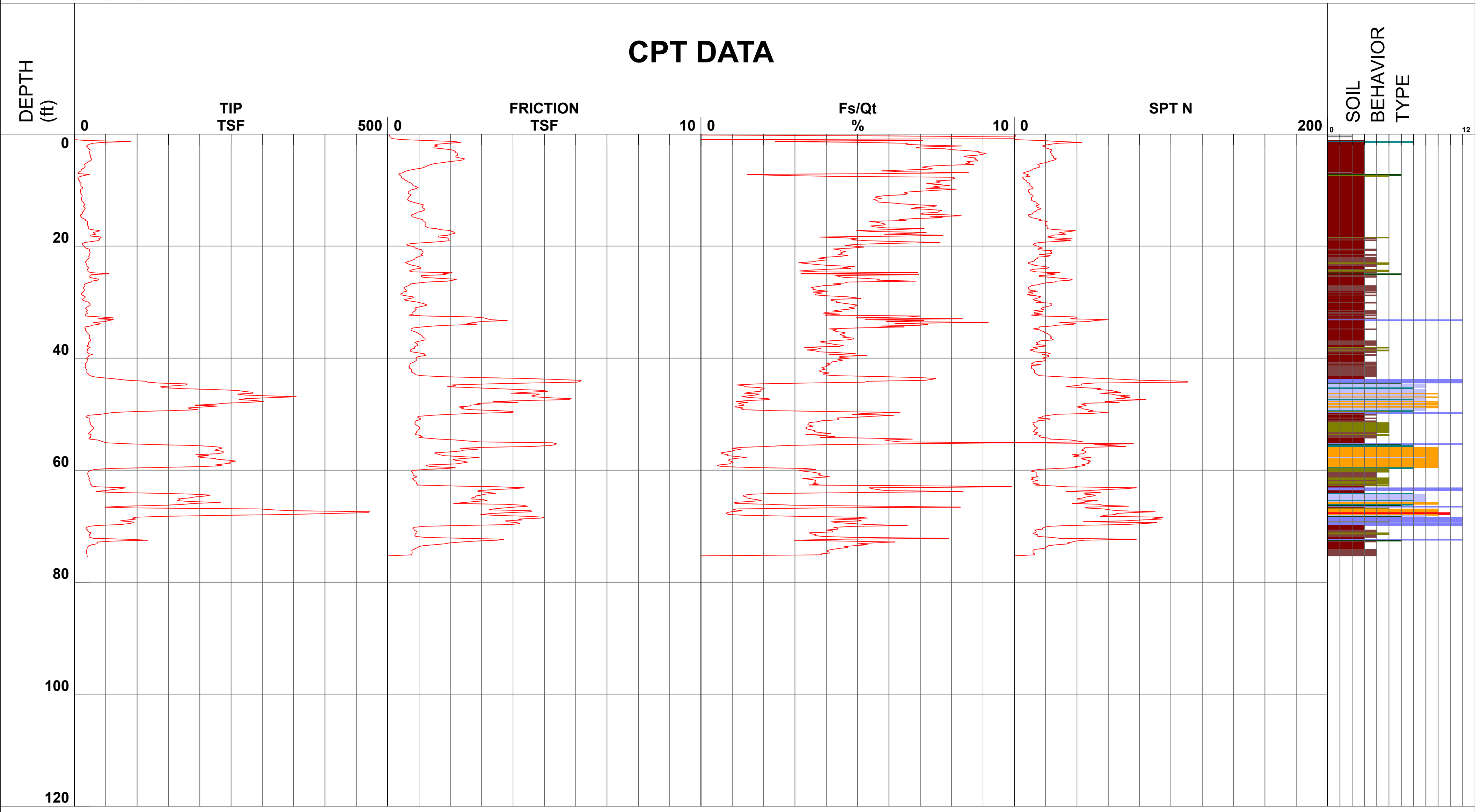
S*Soil behavior type and SPT based on data from UBC-1983

# Cornerstone Earth Group

**Project** Sunrise Senior Living
**Job Number** 935-1-2
**Hole Number** CPT-03
**EST GW Depth During Test** 23.70 ft
**Operator** KK-RB
**Cone Number** DDG1379
**Date and Time** 11/8/2016 2:36:20 PM
**Filename** SDF(344).cpt
**GPS**
**Maximum Depth** 75.46 ft

Net Area Ratio .8

## CPT DATA



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

Cone Size 10cm squared

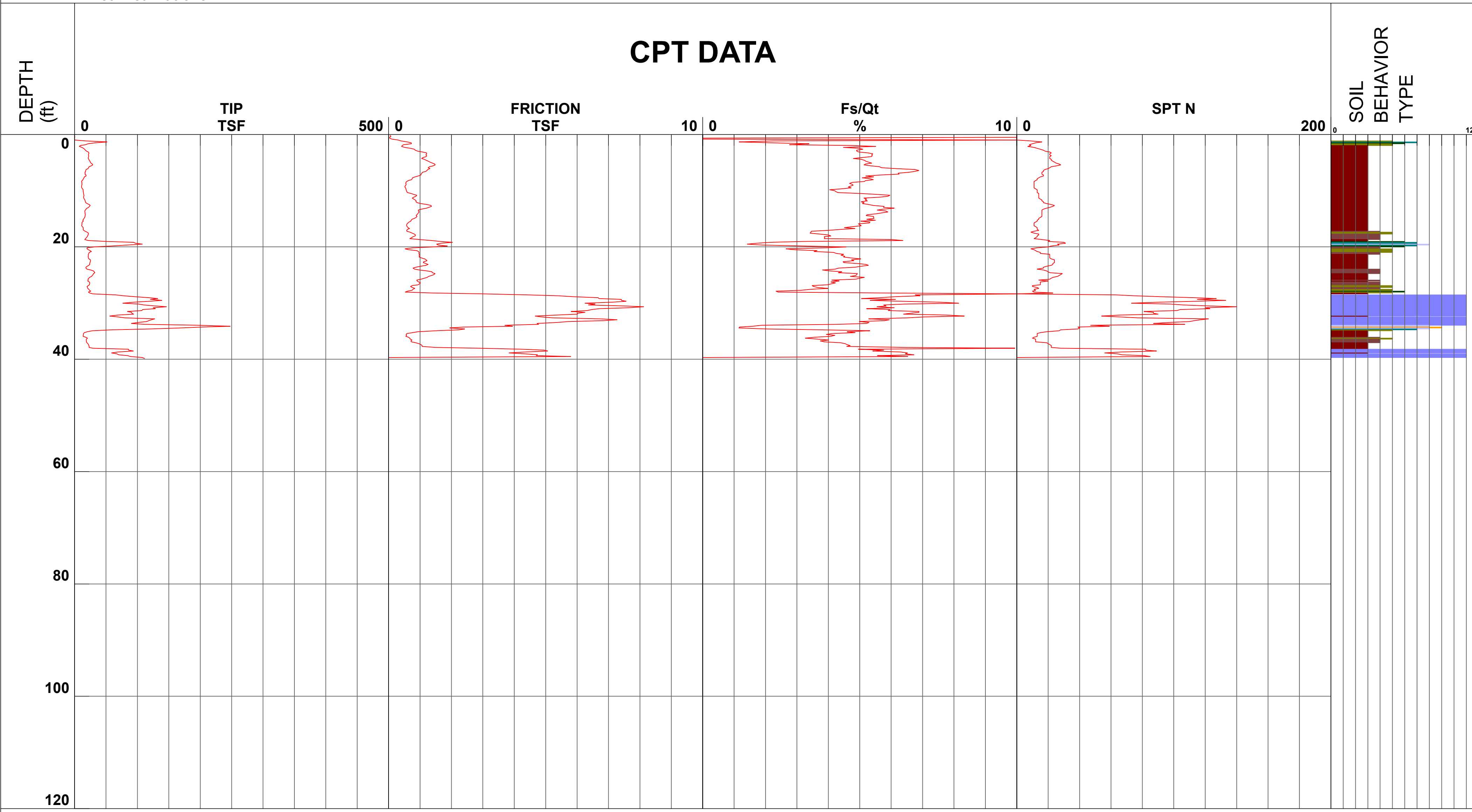
S*Soil behavior type and SPT based on data from UBC-1983

# Cornerstone Earth Group

Middle Earth
GEO TESTING INC.

| | | | | | |
|---|---|---|---|---|---|
| **Project** | Sunrise Senior Living | **Operator** | KK-RB | **Filename** | SDF(339).cpt |
| **Job Number** | 935-1-2 | **Cone Number** | DDG1379 | **GPS** | |
| **Hole Number** | CPT-04 | **Date and Time** | 11/8/2016 8:06:26 AM | **Maximum Depth** | 39.86 ft |
| **EST GW Depth During Test** | | 22.00 ft | | | |

**Net Area Ratio .8**



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

**Cone Size 10cm squared**

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

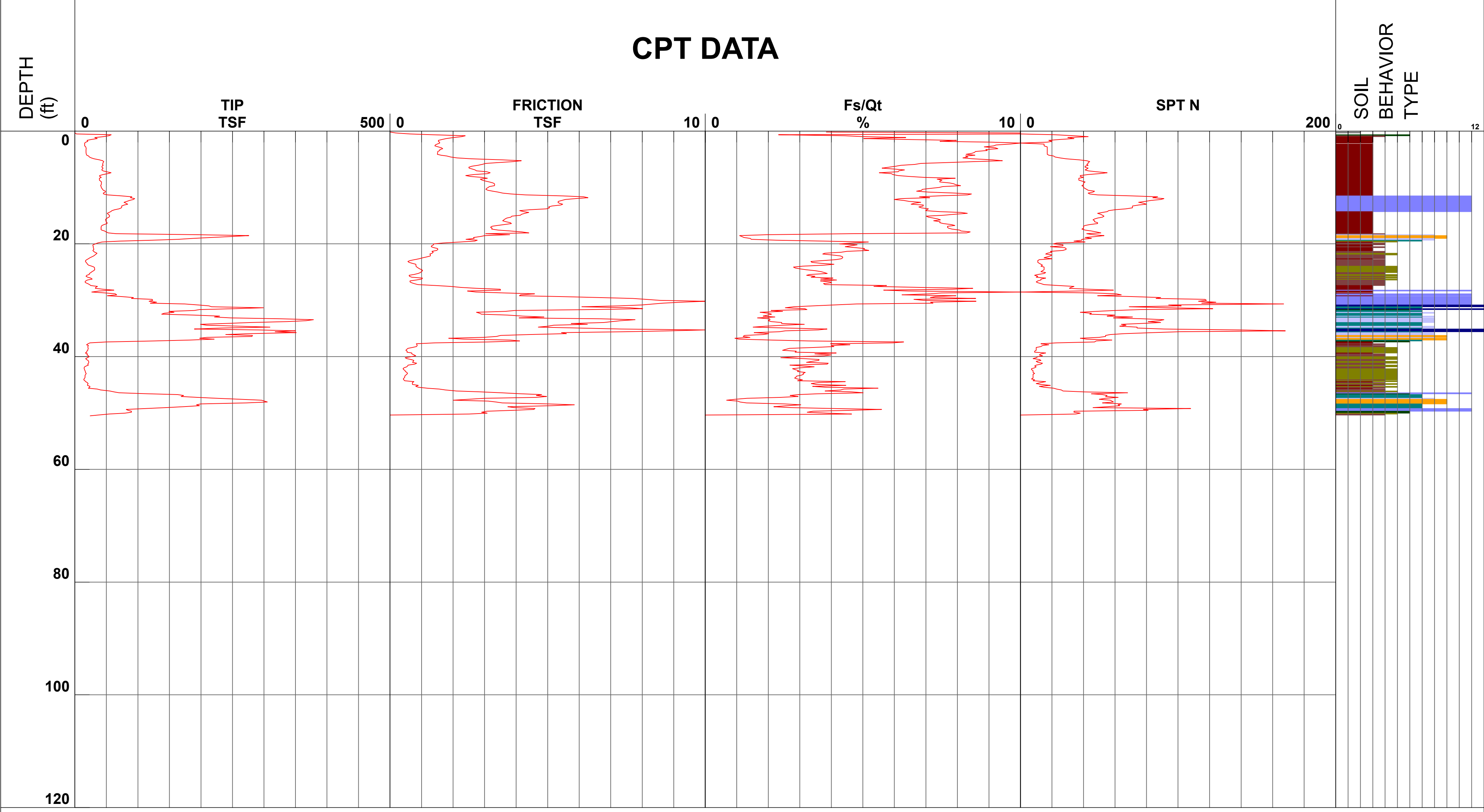
Middle Earth
Geo Testing Inc.

# Cornerstone Earth Group

**Project** Sunrise Senior Living
**Job Number** 935-1-2
**Hole Number** CPT-05
**EST GW Depth During Test** 22.00 ft
**Operator** KK-RB
**Cone Number** DDG1379
**Date and Time** 11/8/2016 10:50:27 AM
**Filename** SDF(342).cpt
**GPS**
**Maximum Depth** 50.52 ft

**Net Area Ratio .8**

## CPT DATA



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

Cone Size 10cm squared

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

## APPENDIX B: LABORATORY TEST PROGRAM

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

**Moisture Content:** The natural water content was determined (ASTM D2216) on 30 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 27 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

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

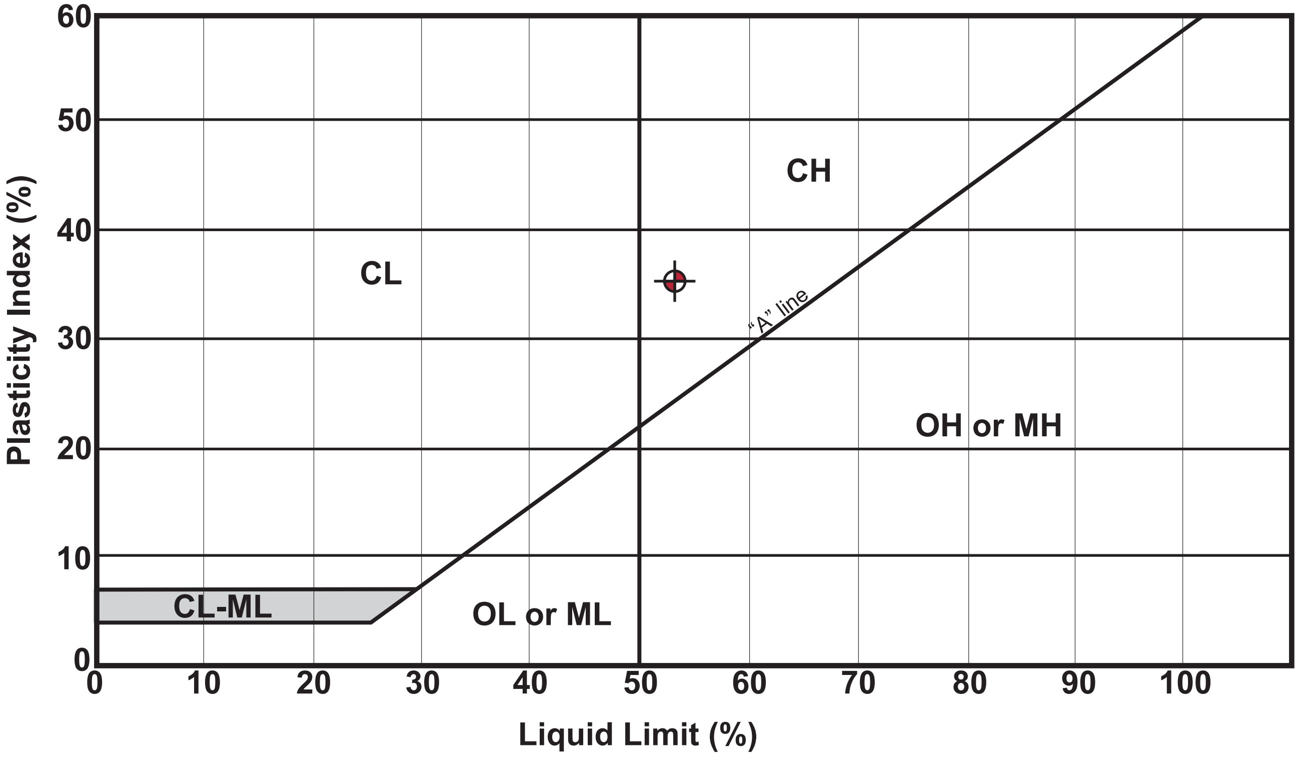
**Plasticity Index:** One Plasticity Index determination (ASTM D4318) was performed on a sample of the subsurface soil to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of this test are shown on the boring log at the appropriate sample depth.

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

**Consolidation:** Two consolidation tests (ASTM D2435) were performed on relatively undisturbed samples of the subsurface clayey soils to assist in evaluating the compressibility property of the soil. Results of the consolidation tests are presented graphically in this appendix.

**Corrosion:** Three 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.

# Plasticity Index (ASTM D4318) Testing Summary



| Symbol | Boring No. | Depth (ft) | Natural Water Content (%) | Liquid Limit (%) | Plastic Limit (%) | Plasticity Index | Passing No. 200 (%) | Group Name (USCS - ASTM D2487) |
|---|---|---|---|---|---|---|---|---|
| ⊕ | EB-1 | 2.0 | 15 | 53 | 18 | 35 | — | Fat Clay (CH) |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |

CORNERSTONE
EARTH GROUP

**Plasticity Index Testing Summary**

**Sunrise Senior Living of Redwood City**
**Redwood City, CA**

Project Number
935-1-2

Figure Number
Figure B1

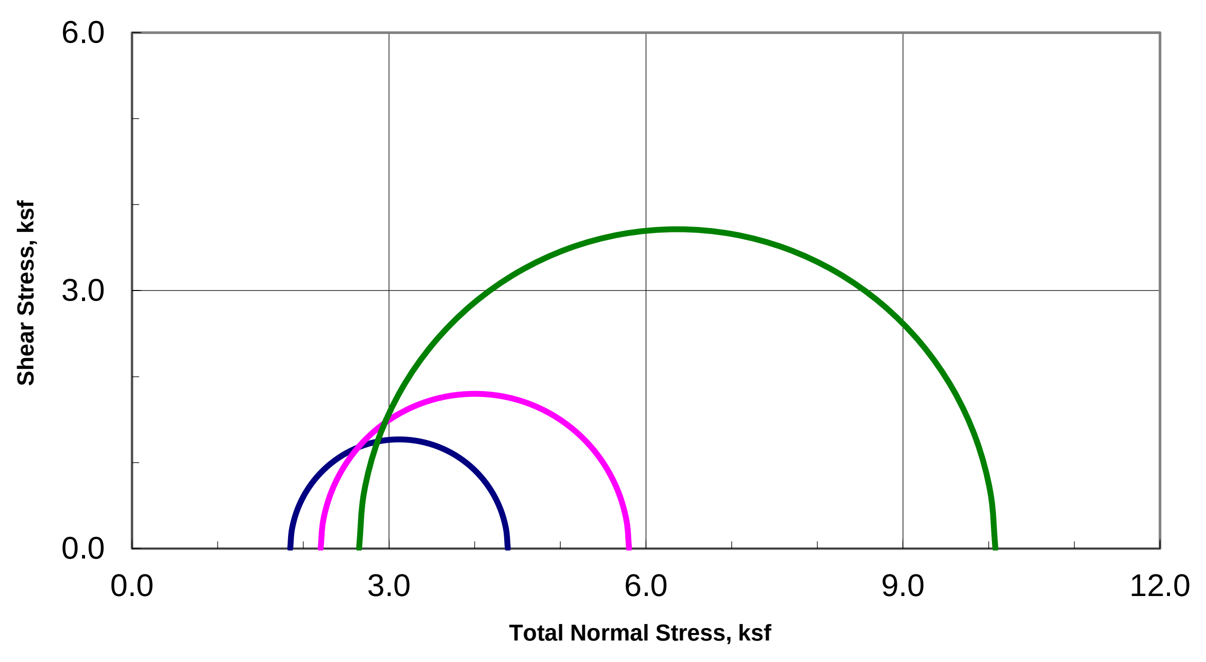
Date
November 2016

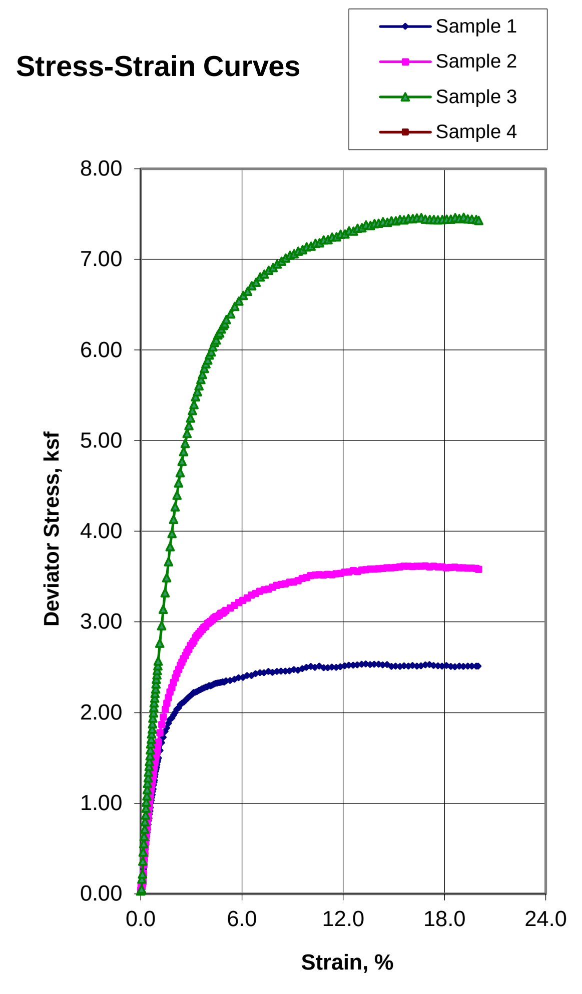
Drawn By
FLL

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

# Unconsolidated-Undrained Triaxial Test

## ASTM D2850





**Sample Data**

| | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| **Moisture %** | 29.1 | 16.8 | 18.1 | |
| **Dry Den,pcf** | 93.4 | 114.8 | 113.2 | |
| **Void Ratio** | 0.806 | 0.468 | 0.489 | |
| **Saturation %** | 97.5 | 96.9 | 99.7 | |
| **Height in** | 6.06 | 6.08 | 6.02 | |
| **Diameter in** | 2.88 | 2.88 | 2.87 | |
| **Cell psi** | 12.8 | 15.3 | 18.4 | |
| **Strain %** | 13.34 | 15.00 | 15.00 | |
| **Deviator, ksf** | 2.539 | 3.599 | 7.427 | |
| **Rate %/min** | 1.00 | 1.00 | 1.00 | |
| **in/min** | 0.061 | 0.061 | 0.060 | |
| **Job No.:** | **640-1054** | | | |
| **Client:** | **Cornerstone Earth Group** | | | |
| **Project:** | **Sunrise Senior Living - 935-1-2** | | | |
| **Boring:** | EB-1 | EB-2 | EB-3 | |
| **Sample:** | 6 | 7 | 9 | |
| **Depth ft:** | 23.5(Tip-5") | 26(Tip-6") | 35.5(Tip-1") | |

**Visual Soil Description**

| **Sample #** | |
|---|---|
| **1** | Olive Brown Sandy CLAY |
| **2** | Olive Brown Clayey SAND w/ Gravel |
| **3** | Olive Gray Sandy CLAY |
| **4** | |

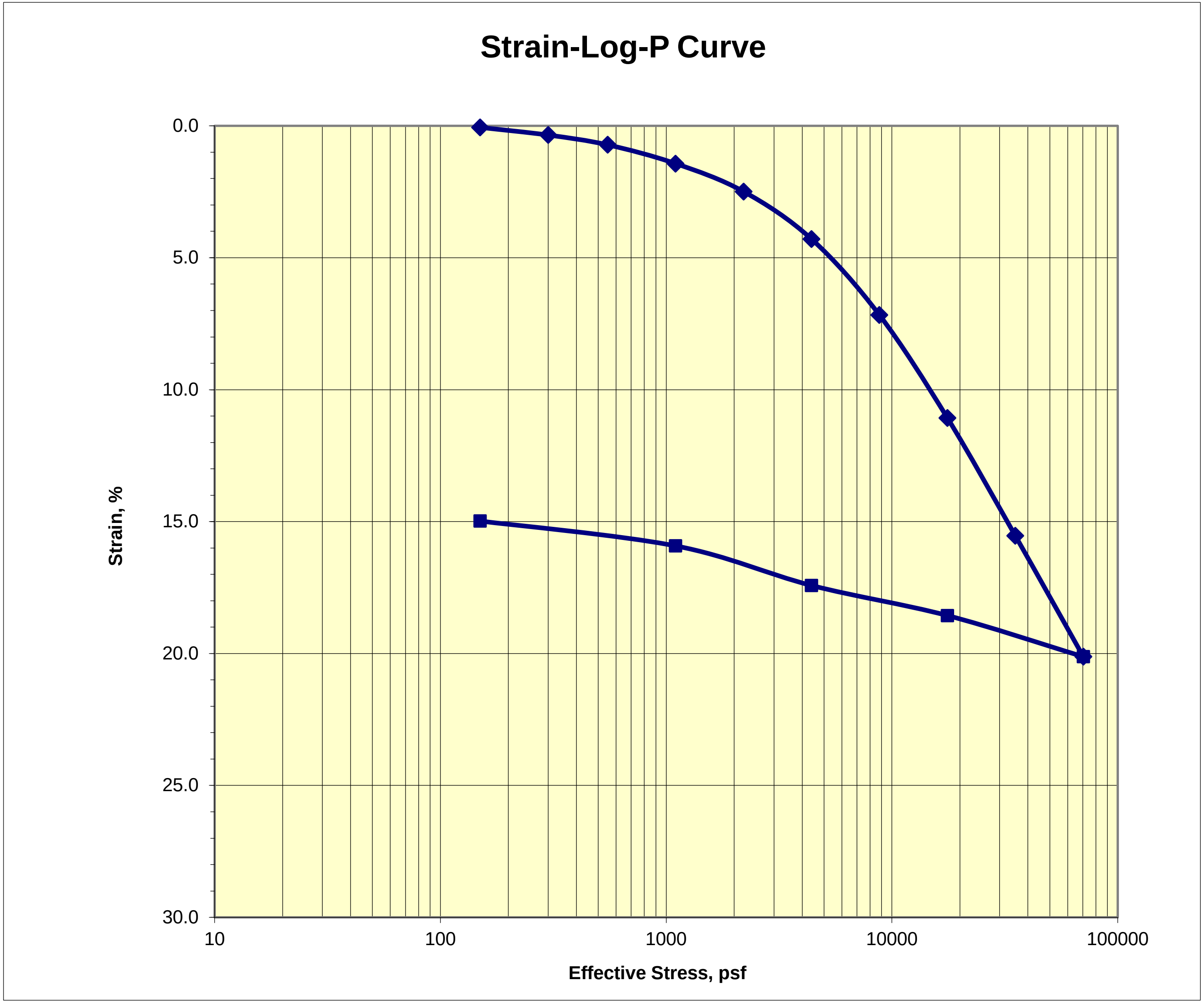
Remarks:

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

# Consolidation Test

## ASTM D2435

| | | | | | |
|---|---|---|---|---|---|
| **Job No.:** | 640-1054 | **Boring:** | EB-1 | **Run By:** | MD |
| **Client:** | Cornerstone Earth Group | **Sample:** | 6 | **Reduced:** | PJ |
| **Project:** | 935-1-2 | **Depth, ft.:** | 23.5(Tip-3") | **Checked:** | PJ/DC |
| **Soil Type:** | Olive Brown Sandy CLAY | | | **Date:** | 11/29/2016 |



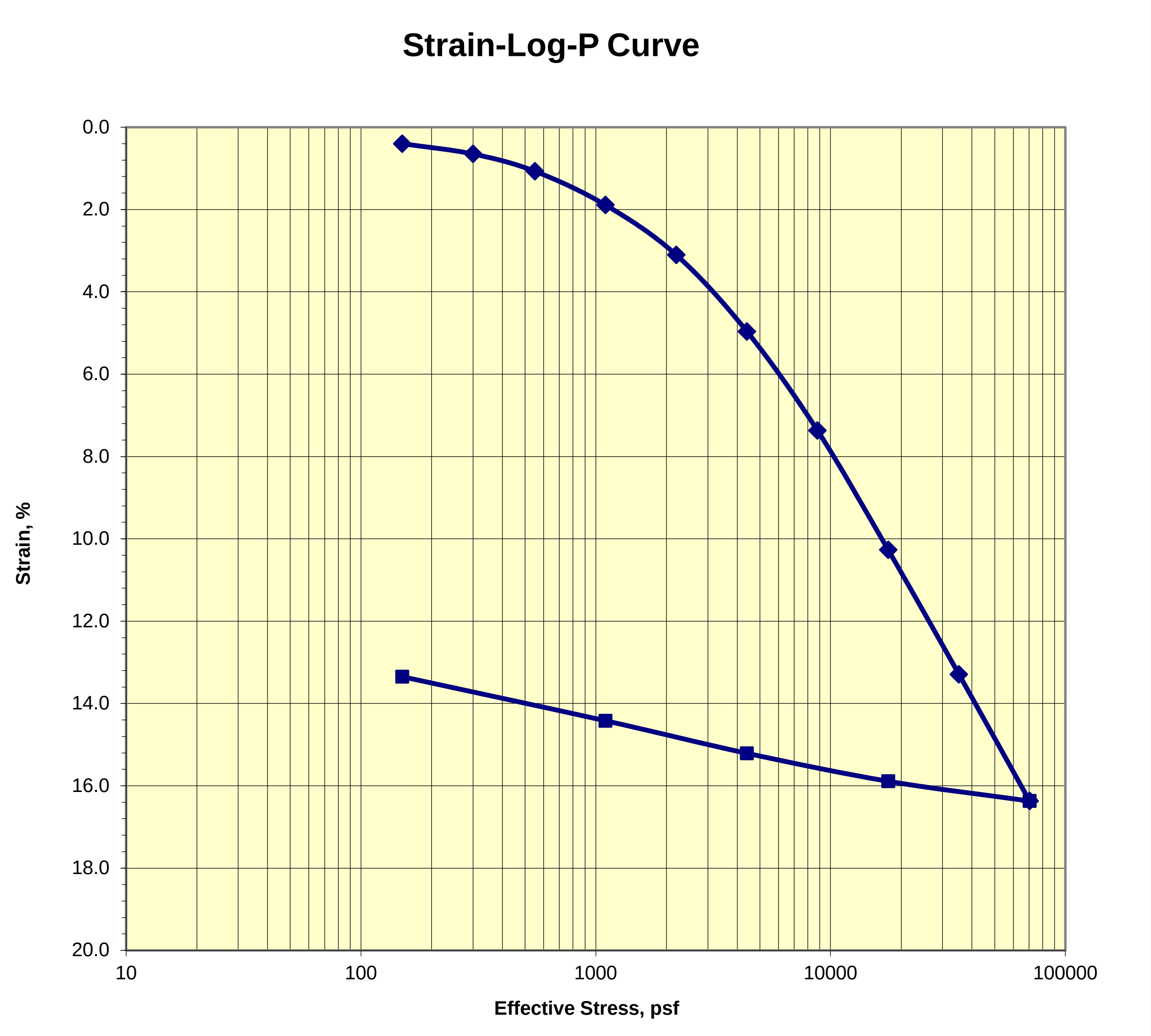
| **Assumed Gs** 2.7 | **Initial** | **Final** |
|---|---|---|
| **Moisture %:** | 23.6 | 17.8 |
| **Dry Density, pcf:** | 97.5 | 113.9 |
| **Void Ratio:** | 0.729 | 0.479 |
| **% Saturation:** | 87.5 | 100.0 |

**Remarks**:

# Consolidation Test

## ASTM D2435

| | | | | | |
|---|---|---|---|---|---|
| **Job No.:** | 640-1054 | **Boring:** | EB-2 | **Run By:** | MD |
| **Client:** | Cornerstone Earth Group | **Sample:** | 7 | **Reduced:** | PJ |
| **Project:** | 935-1-2 | **Depth, ft.:** | 26.0(Tip-4") | **Checked:** | PJ/DC |
| **Soil Type:** | Olive Brown Clayey SAND w/ Gravel | | | **Date:** | 11/30/2016 |



| **Assumed Gs** 2.75 | **Initial** | **Final** |
|---|---|---|
| **Moisture %:** | 17.2 | 13.7 |
| **Dry Density, pcf:** | 107.7 | 124.7 |
| **Void Ratio:** | 0.594 | 0.377 |
| **% Saturation:** | 79.7 | 100.0 |

**Remarks**:

COOPER TESTING LABORATORY

# Corrosivity Tests Summary

**CTL #** 640-1054 **Date:** 11/16/2016 **Tested By:** PJ **Checked:** PJ

**Client:** Cornerstone Earth Group **Project:** Sunrise Senior Living **Proj. No:** 935-1-2

**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 | 1A | 1.5 | - | - | 1,344 | 5 | 24 | 0.0024 | 6.0 | - | - | - | 16.0 | Dark Olive Brown CLAY w/ Sand |
| EB-3 | 2A | 5.5 | - | - | 1,836 | 8 | 50 | 0.0050 | 6.4 | - | - | - | 18.5 | Olive Brown Sandy CLAY |
| EB-3 | 4A | 14.5 | - | - | 2,815 | 5 | 51 | 0.0051 | 6.9 | - | - | - | 24.4 | Yellowish Brown Sandy CLAY (Silty) |