CORNERSTONE EARTH GROUP

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| **Type of Services** | **Geotechnical Investigation** |
| **Project Name** | **Osgood Road Warehouse** |
| **Location** | **43510 Osgood Road**<br>**Fremont, California** |
| **Client** | **Seefried Industrial Properties, Inc.** |
| **Client Address** | **2201 E. Camelback Road, Ste 225B**<br>**Phoenix, Arizona** |
| **Project Number** | **791-3-2** |
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# DRAFT


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|---|---|
| | **DRAFT** |
| **Type of Services** | **Geotechnical Investigation** |
| **Project Name** | **Osgood Road Warehouse** |
| **Location** | **43510 Osgood Road** |
| | **Fremont, California** |

## SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Seefried Industrial Properties, Inc. for the Osgood Road Warehouse project in Fremont, 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 plan titled, "Conceptual Site Plan – Scheme 8, 43510 Osgood Road, Fremont, CA," prepared by HPA Architects dated August 13, 2015.
- A topographic plan prepared by Kier & Wright dated October 2015.
- A preliminary grading and drainage plan prepared by Kier & Wright dated October 2015

### 1.1 PROJECT DESCRIPTION

The 7.8-acre project site is located at 43510 Osgood Road in Fremont, California. The site is currently vacant, but was previously occupied by a former equestrian building. We understand that a new warehouse is currently planned for the site.

The planned warehouse will be of tilt-up concrete construction with a concrete slab-on-grade floor. The planned building will have a footprint of approximately 119,261 square feet. Appurtenant parking, truck docks, utilities, landscaping and other improvements necessary for site development are also planned.

We assume that wall and column loads for the building will be typical for this type of construction, and that cuts and fills on the order of 2 to 4 feet will be required to construct the building pad, truck dock and parking areas.

### 1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated August 19, 2015, and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building

foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

### 1.3 EXPLORATION PROGRAM

Field exploration consisted of four Cone Penetration Tests (CPTs) advanced on October 1, 2015, and four borings drilled on October 7, 2015, with truck-mounted, hollow-stem auger drilling equipment. The CPTs were advanced to depths of approximately 50 feet; the borings were drilled to depths of approximately 10 to 50 feet. One of the borings (Boring EB-1) was advanced adjacent to CPT-4 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, grain size analyses, washed sieve analyses, Plasticity Index tests and triaxial compression tests. Details regarding our laboratory program are included in Appendix B.

### 1.5 ENVIRONMENTAL SERVICES

Cornerstone Earth Group also provided environmental services for this project, including Phase 1 and 2 site assessments; environmental findings and conclusions are provided under separate covers.

## SECTION 2: REGIONAL SETTING

### 2.1 GEOLOGICAL SETTING

The site is located at the northern end of the Santa Clara Valley, which is a broad alluvial plane between the Santa Cruz Mountains to the southwest and west, and the Diablo Range to the northeast. The San Andreas Fault system, including the Monte Vista-Shannon Fault, exists within the Santa Cruz Mountains and the Hayward and Calaveras Fault systems exist within the Diablo Range. Helley & Graymer (1997) have mapped the site vicinity as being underlain by Holocene-aged alluvial and fluvial deposits (Qhaf). The Mission Hills to the east are generally mapped by Graymer et. al. (1996) as Irvington Gravels (QTi), Orinda Formation (Tor), and Briones Formation (Tbr).

### 2.2 REGIONAL SEISMICITY

The San Francisco Bay area is one of the most seismically active areas in the Country. While seismologists cannot predict earthquake events, the U.S. Geological Survey's Working Group on California Earthquake Probabilities 2007 estimates there is a 63 percent chance of at least one magnitude 6.7 or greater earthquake occurring in the Bay Area region between 2007 and 2036. As seen with damage in San Francisco and Oakland due to the 1989 Loma Prieta earthquake that was centered about 50 miles south of San Francisco, significant damage can occur at considerable distances. Higher levels of shaking and damage would be expected for earthquakes occurring at closer distances.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The table below presents the State-considered active faults within 25 kilometers of the site.

**Table 1: Approximate Fault Distances**

| Fault Name | Distance | |
|---|---|---|
| | (miles) | (kilometers) |
| Hayward (Total Length) | <0.1 | <0.2 |
| Hayward (Southeast Extension) | 4.3 | 7.0 |
| Calaveras North | 5.3 | 8.5 |
| Calaveras South | 10.3 | 16.6 |
| Monte Vista-Shannon | 15.2 | 24.5 |

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

## SECTION 3: SITE CONDITIONS

### 3.1 SITE BACKGROUND

We reviewed historic aerial photographs obtained from EDR dating from 1939 to 2012, as well as aerial images from Google Earth from 1993 to 2014. The aerial images indicate that the parcel was originally occupied by a residential home and several appurtenant structures on the northwest corner of the site from 1939 to the late 1960s. An unpaved access road or driveway crossed west to east along the southern one-third of the parcel that appears to have provided access to a parcel to the east. High power transmission lines were observed crossing the southern half of the site in all aerial photos dating back to 1939. The adjacent Interstate 680 Freeway had not been constructed in the 1968 photograph.

By 1974, a new rectangular building reported to be a riding and stable facility had been constructed near the northeast corner of the parcel and the previous small structures had been demolished. The building appeared to be accessed by a new gravel roadway and surrounding

equestrian ring and equipment storage areas. The stables and equestrian ring appear to have been demolished by 1998. Remnants of the original gravel road were observed in the 1998 photo. The site appears to have remained relatively unchanged since 1998.

## 3.1 SURFACE DESCRIPTION

The approximately 7.8-acre project site is located at 43510 Osgood Road and is currently designated as Assessor's Parcel Nos. APN 513-701-7-10. The site is currently vacant and covered with low grasses, weeds, bushes and a few small trees. The site is bounded by Osgood Road to the west, existing commercial development to the north and south, and Interstate I680 to the east. The perimeter of the site is bounded by chain link fencing or masonry sound walls.

Four sets of high-power transmission lines cross the southern half of the site in an east-west orientation, as shown on Figure 2. We observed several small soil stockpiles around the site, as well as a water supply well at the west end of the site. A natural gas pipeline reportedly runs east to west through the middle of the utility easement.

Based on topographic data from the project civil engineer, existing site grades range from approximately Elevation 30 feet near Osgood Road to Elevation 35 feet near the southeast corner of the site (NGVD, 1929).

## 3.2 SUBSURFACE CONDITIONS

Our explorations generally encountered alluvial soils consisting of stiff to very stiff clays and occasional interbedded medium dense to dense sands to the maximum depth explored at 50 feet. The sand layers generally ranged from approximately 1 to 2 feet thick, except in CPT-2, where a medium dense to dense sand layer was encountered from about 38 to 43 feet deep. Minor undocumented fills, likely associated with prior site development, were encountered in CPT-1 and Borings EB-1 and EB-3 to depths of approximately 1½ to 3 feet. The fill consisted of hard sandy lean clay in the borings.

### 3.2.1 Plasticity/Expansion Potential

We performed four Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of surficial soils and the plasticity of the fines in potentially liquefiable layers. The results of the surficial PI tests indicated PIs of 16, indicating low to moderate expansion potential to wetting and drying cycles. The results of tests performed at depths of 9, 15, and 20 feet indicated PIs of 17, 17, and 21 for samples at or below the ground water level.

### 3.2.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet ranged from approximately 3 percent below to 5 percent over the estimated laboratory optimum moisture.

### 3.3 GROUND WATER

Ground water was inferred from two pore pressure dissipation tests performed during CPT-1 and CPT-3, which indicated ground water at a depth of approximately 8 to 9 feet below current grades. Ground water was encountered in Borings EB-1 and EB-3 at depths ranging from approximately 9 to 15 feet. All measurements were taken at the time of exploration and may not represent the stabilized levels that can be higher than the initial levels encountered.
Historic high ground water maps prepared by the California Geologic Survey indicate ground water on the order of 8 feet (CGS, 2004). For our study, we assumed a design ground water level at 8 feet below current site grades.

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

## SECTION 4: GEOLOGIC HAZARDS

### 4.1 FAULT RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. The site is not located within a State-designated Alquist Priolo Earthquake Fault Zone, although the western edge of an A-P Zone is located immediately east of the eastern property boundary (see Figure 2 and Figure 4). As shown in Figures 3 and 4, no known surface expression of fault traces is thought to cross the site; therefore, fault rupture hazard is not a significant geologic hazard at the site.

### 4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration (PGA) was estimated for analysis using a value equal to $F_{PGA}$ x PGA, as allowed in the 2013 edition of the California Building Code. For our liquefaction analysis we used a PGA of 0.87g.

### 4.3 LIQUEFACTION POTENTIAL

As shown on Figure 5, the site is within a State-designated Liquefaction Hazard Zone (CGS, Niles Quadrangle, 2004) and a City of Fremont Liquefaction Hazard Area (City of Fremont, 2011). Our field and laboratory programs addressed this issue by sampling potentially liquefiable layers to depths of at least 50 feet, performing visual classification on sampled materials, evaluating CPT correlations, and performing various tests to further classify the soil properties.

#### 4.3.1 Background

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress

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

### 4.3.2 Analysis

As discussed in the "Subsurface" section above, several sand layers were encountered below the design ground water depth of 8 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. Soil samples collected from EB-1 that was drilled adjacent to CPT-4 were tested to evaluate grain size and Plasticity Index (PI), as well as visually observed for confirmation of CPT soil behavior types.

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-4) are presented graphically on Figures C-1 to C-4 in Appendix C; liquefaction calculations are also presented in Appendix C.

#### 4.3.3 Summary

Our analyses indicate that several thin layers could potentially experience liquefaction triggering that could result in post-liquefaction total settlement at the ground surface ranging up to about ½ inch based on the Yoshimine et. al. (2006) method. As discussed in SP 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 ¼ inch or less over a horizontal distance of approximately 50 feet.

#### 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 a 8-foot thick layer of non-liquefiable cap is sufficient to prevent ground rupture; therefore, the above total settlement estimates are considered 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 unsaturated sandy soils can settle during strong seismic shaking. As the soils encountered above the ground water level at the site were predominantly stiff to very stiff clays and medium dense to dense sands, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low.

### 4.6 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone X, an area determined to be outside the 0.2 percent annual chance floodplain. We recommend the project civil engineer be retained to confirm this information, 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 localized undocumented fills
- Shallow ground water

#### 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 to the ground surface through cracks in the surficial soils is low, our analysis indicates that liquefaction-induced settlement on the order of ½ inch or less could occur, resulting in differential settlement up to ¼ inch between adjacent foundation elements, which is assumed to be approximately 50 feet. Based on our understanding of potential warehouse foundation loads, it should be feasible to support the proposed building on shallow foundations; however, the building foundations will need to be designed to tolerate total and differential settlement due to static loads and liquefaction-induced settlement. Detailed foundation recommendations are presented in the “Foundations” section.

#### 5.1.2 Undocumented Fill

As discussed, localized undocumented fills were encountered in our some of our explorations and several soil stockpiles were observed at various locations across the site. In addition, previous site development, which has since been demolished, included a residential structure and associated barns prior to 1970 and an equestrian riding center in the 1970s and 1980s. We anticipate that shallow fills could also be encountered in these areas. Due to the potential variability of these shallow fills, all undocumented fills should be over-excavated and re-compacted prior to placing any new fill in building pad areas. Stockpiles should be removed from the site or possibly re-used as fill. Recommendations addressing this concern are presented in the “Earthwork” section of this report.

### 5.1.3 Shallow Ground Water

Shallow ground water was measured at depths ranging from approximately 9 to 15 feet below the existing ground surface, and historic high ground water is mapped at a depth of about 7 feet in the vicinity. Our experience with similar sites in the vicinity indicates that shallow ground water could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable pavement subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering and shoring of utility trenches may be required in some isolated areas of the site. Detailed recommendations addressing this concern are presented in the “Earthwork” section of this report.

## 5.2 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. Based on our site observations, surficial stripping should extend about 1 to 3 inches below existing grade in vegetated areas.

At the owner’s option the surface vegetation may be mowed, leaving no more than 1 inch of stubble, the area disked in two directions to a depth of 12 inches, moisture conditioned, and re-compacted.

### 6.1.2 Tree and Shrub Removal

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

### 6.1.3 Abandonment of Existing Utilities

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

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

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

## 6.2 REMOVAL OF EXISTING FILLS

As previously discussed, undocumented fill was encountered in Borings EB-1, EB-3 and CPT-2 to depths of approximately 1½ to 3 feet below the existing grades. We also observed several soil stockpiles ranging from about 2 to 8 feet high. The approximately location of the stockpiles, as interpreted from available topographic information, is shown on Figure 2. We understand that the proposed site grading will raise the elevation of the site up to 4 feet above current grades in the building area.

All undocumented fills and soil stockpiles should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Additional recommendations to address undocumented fill within areas of environmental excavations are provided below. Provided the existing fills meet the "Material for Fill" requirements below, the

fills may be reused when backfilling the excavations. Based on review of the samples collected from our borings, it appears that the fill may be reused. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below.

Fills extending into planned pavement and flatwork areas may be left in place provided they are determined to be a low risk for future differential settlement and that the upper 12 to 18 inches of fill below pavement subgrade is re-worked and compacted as discussed in the “Compaction” section below. In our opinion, the fills encountered at this site are not acceptable to be left in place.

## 6.3 TEMPORARY CUT AND FILL SLOPES

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

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

## 6.4 SUBGRADE PREPARATION

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

## 6.5 SUBGRADE STABILIZATION MEASURES

Soil subgrade and fill materials, especially soils with high fines contents such as clays and silty soils, can become unstable due to high moisture content, whether from high in-situ moisture contents or from winter rains. As the moisture content increases over the laboratory optimum, it becomes more likely the materials will be subject to softening and yielding (pumping) from construction loading or become unworkable during placement and compaction.
As discussed in the “Subsurface” section in this report, the in-situ moisture contents are up to about 5 percent over the estimated laboratory optimum in the upper 10 feet of the soil profile. The contractor should anticipate drying or moisture conditioning the soils prior to reusing them as fill following periods of heavy rainfall or for soils excavated below a depth of 3 feet. In addition, repetitive rubber-tire loading will likely de-stabilize the soils.

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

### 6.5.1 Scarification and Drying

The subgrade may be scarified to a depth of 6 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.

### 6.5.2 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.

### 6.5.3 Chemical Treatment

Where the unstable area exceeds about 5,000 to 10,000 square feet and/or site winterization is desired, chemical treatment with quicklime (CaO), kiln-dust, or cement may be more cost-effective than removal and replacement. Recommended chemical treatment depths will typically range from 12 to 18 inches depending on the magnitude of the instability.

## 6.6 MATERIAL FOR FILL

### 6.6.1 Re-Use of On-site Soils

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

### 6.6.2 Potential Import Sources

Imported material should be inorganic with a Plasticity Index (PI) of 15 or less, and not contain recycled asphalt concrete where it will be used within the habitable building areas. To prevent significant caving during trenching or foundation construction, imported material should have sufficient fines. Samples of potential import sources should be delivered to our office at least 10 days prior to the desired import start date. Information regarding the import source should be provided, such as any site geotechnical reports. If the material will be derived from an excavation rather than a stockpile, potholes will likely be required to collect samples from

throughout the depth of the planned cut that will be imported. At a minimum, laboratory testing will include PI tests. Material data sheets for select fill materials (Class 2 aggregate base, ¾-inch crushed rock, quarry fines, etc.) listing current laboratory testing data (not older than 6 months from the import date) may be provided for our review without providing a sample. If current data is not available, specification testing will need to be completed prior to approval.

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

## 6.7 COMPACTION REQUIREMENTS

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

**Table 2: Compaction Requirements**

| Description | Material Description | Minimum Relative[1] Compaction (percent) | Moisture[2] Content (percent) |
|---|---|---|---|
| General Fill (within upper 5 feet) | On-Site Soils | 90 | >2 |
| General Fill (below a depth of 5 feet) | On-Site Soils | 93 | >2 |
| Retaining Wall Backfill | With Surface Improvements | 95[4] | >2 |
| Trench Backfill | On-Site Soils | 90 | >2 |
| Trench Backfill (upper 6 inches of subgrade) | On-Site Soils | 95 | >2 |
| Crushed Rock Fill | ¾-inch Clean Crushed Rock | Consolidate In-Place | NA |
| Non-Expansive Fill | Imported Non-Expansive Fill | 90 | Optimum |
| Flatwork Subgrade | On-Site Soils | 90 | >2 |
| Flatwork Aggregate Base | Class 2 Aggregate Base[3] | 90 | Optimum |
| Pavement Subgrade | On-Site Soils | 95 | >2 |
| Pavement Aggregate Base | Class 2 Aggregate Base[3] | 95 | Optimum |
| Asphalt Concrete | Asphalt Concrete | 95 | NA |

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

## 6.8 TRENCH BACKFILL

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

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

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

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated

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

## 6.9 SITE DRAINAGE

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

## 6.10 LOW-IMPACT DEVELOPMENT (LID) IMPROVEMENTS

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

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

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

- Locally, seasonal high ground water is mapped at a depth of about 7 feet and was encountered in our subsurface explorations as shallow as 9 feet, and therefore seasonally could be within 10 feet of the base of the infiltration of stormwater.

- 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.

### 6.10.1 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.

#### 6.10.1.1 General Bioswale Design Guidelines

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

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

#### 6.10.1.2 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.

#### 6.10.1.3 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.

## 6.11 LANDSCAPE CONSIDERATIONS

If applicable and depending on the finished site grade, 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, and
- Selecting landscaping that requires little or no watering, especially near foundations.

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

## SECTION 7: FOUNDATIONS

### 7.1 SUMMARY OF RECOMMENDATIONS

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

### 7.2 SEISMIC DESIGN CRITERIA

We understand that the project structural design will be based on the 2013 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The "Seismic Coefficients" used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Based on our site exploration to depths of 50 feet and review of available geologic mapping for the vicinity, the site is underlain by deep alluvial soils. 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 web-based program (http://earthquake.usgs.gov/designmaps/us/application.php) *U.S. Seismic Design maps* 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 3: CBC Site Categorization and Site Coefficients**

| Classification/Coefficient | Design Value |
|---|---|
| Site Class | D |
| Site Latitude | 37.51787° |
| Site Longitude | -121.94418° |
| 0.2-second Period Mapped Spectral Acceleration[1], $S_S$ | 2.267g |
| 1-second Period Mapped Spectral Acceleration[1], $S_1$ | 0.940g |
| Short-Period Site Coefficient – Fa | 1.0 |
| Long-Period Site Coefficient – Fv | 1.5 |
| 0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - $S_{MS}$ | 2.267g |
| 1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – $S_{M1}$ | 1.410g |
| 0.2-second Period, Design Earthquake Spectral Response Acceleration – $S_{DS}$ | 1.511g |
| 1-second Period, Design Earthquake Spectral Response Acceleration – $S_{D1}$ | 0.940g |

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

## 7.3 SHALLOW FOUNDATIONS

### 7.3.1 Spread Footings

Spread footings should bear entirely on natural, undisturbed soil or engineered fill, be at least 18 inches wide, and extend at least 18 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 3,000 psf for dead loads, 4,500 psf for combined dead plus live loads, and 6,000 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 assumed the typical loading in the following table.

**Table 4: Assumed Structural Loading**

| Foundation Area | Range of Assumed Loads |
|---|---|
| Interior Isolated Column Footing | 50 to 75 kips |
| Exterior Isolated Column Footing | 50 to 75 kips |
| Perimeter Strip Footing | 4 to 6 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 about ¼-inch of post-construction differential settlement between adjacent foundation elements. In addition we estimate that differential seismic movement will be on the order of ¼ inch or less over a horizontal distance of 50 feet, resulting in a total estimated differential footing movement of ½ inch between foundation elements, assumed to be on the order of 50 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 0.40 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

### 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 WAREHOUSE SLABS-ON-GRADE

We recommend that the proposed warehouse slab-on-grade be at least 6 inches thick and reinforced with minimum temperature and shrinkage steel. If heavy forklift, rack or pallet storage is anticipated, we recommend that the structural engineer evaluate the required stiffness and thickness of the warehouse slab to accommodate the flexural stresses associated with these various loading conditions.

The warehouse slab should be supported on at least 8 inches of crushed granular base having an R-value of at least 50, no more than 10 percent passing the No. 200 sieve and no particles larger than 2½ inches, such as Class 2 aggregate subbase. A modulus of subgrade reaction of 100 pounds per cubic inch (pci) may be used for the initial slab-on-grade design. As an alternative, the building pad subgrade can be chemically treated with lime to a depth of least 12 inches and the granular base section reduced to 4 inches thick. If this option is considered, a modulus of subgrade reaction of 200 pci may be used for the warehouse slab analysis.

We should review and approve the planned select fill materials prior to importing. The slab section should be constructed over subgrade prepared in accordance with the recommendations in the “Earthwork” section of this report. All subgrade and sub-base materials should be placed and compacted in accordance with the “Compaction” section of this report. If significant time elapses between initial subgrade preparation and slab-on-grade construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to at least 2 percent over the optimum moisture content.

If there will be areas within the warehouse that are moisture sensitive, such as offices or equipment rooms, a vapor retarder may be placed over the upper granular base prior to slab construction. Please refer to the recommendations in the “Interior Slabs Moisture Protection Considerations” section for vapor retarder construction. 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. The 4- to 8-inch-thick granular base material in the warehouse may be considered as the capillary break. In office areas, at least 4-nches of gravel consisting of ½- to ¾-inch crushed rock with less than 5 percent passing the No. 200 sieve, should be placed below the vapor retarder and consolidated in place with vibratory equipment.

- The concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement.
- Water should not be added after initial batching unless the slump is less than specified and/or the resulting water:cement ratio will not exceed 0.45.
- Where floor coverings are planned, all concrete surfaces should be properly cured.
- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869-98 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 and/or occasional light pick up loading should be at least 4 inches thick and supported on at least 4 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

## SECTION 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 Plasticity Index of the near-surface soils and the variable surface conditions. We have also included pavement structural section alternatives for lime-treated subgrade soil with a minimum estimated design R-value of 50 for your consideration. If it desired to lime-treat the proposed auto parking and truck parking/loading areas, we recommend that the upper 12 inches of subgrade be treated, as discussed in the ""Earthwork" section of this report.

**Table 5: Asphalt Concrete Pavement Recommendations, Untreated Subgrade**

| 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 | 12.5 | 16.0 |
| 6.5 | 4.0 | 14.0 | 18.0 |
| 7.0 | 4.0 | 15.5 | 19.5 |
| 7.5 | 4.5 | 17.0 | 21.5 |
| 8.0 | 5.0 | 17.5 | 22.5 |
| 8.5 | 5.0 | 19.0 | 24.0 |
| 9.0 | 5.5 | 20.5 | 26.0 |

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

**Table 6: Asphalt Concrete Pavement Recommendations, Lime-Treated Subgrade**

| Design Traffic Index (TI) | Asphalt Concrete (inches) | Class 2 Aggregate Base* (inches) | Total Pavement Section Thickness (inches) |
|---|---|---|---|
| 4.0 | 2.5 | 4.0 | 6.5 |
| 4.5 | 2.5 | 4.0 | 6.5 |
| 5.0 | 3.0 | 4.0 | 7.0 |
| 5.5 | 3.0 | 4.0 | 7.0 |
| 6.0 | 3.5 | 4.0 | 7.5 |
| 6.5 | 3.5 | 4.5 | 8.0 |
| 7.0 | 3.5 | 5.5 | 9.0 |
| 7.5 | 4.5 | 5.5 | 10.0 |
| 8.0 | 4.5 | 6.0 | 10.5 |
| 8.5 | 5.0 | 6.0 | 11.0 |
| 9.0 | 5.0 | 7.0 | 12.0 |

*Caltrans Class 2 aggregate base; minimum R-value of 78, minimum subgrade R-value assumed to be 50

Frequently, the full asphalt concrete section is not constructed prior to construction traffic loading. This can result in significant loss of asphalt concrete layer life, rutting, or other pavement failures. To improve the pavement life and reduce the potential for pavement distress

through construction, we recommend the full design asphalt concrete section be constructed prior to construction traffic loading. Alternatively, a higher traffic index may be chosen for the areas where construction traffic will be use the pavements.

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

## 9.2 PORTLAND CEMENT CONCRETE

The exterior Portland Cement Concrete (PCC) pavement recommendations tabulated below are based on methods presented in the Portland Cement Association (PCA) design manual (PCA, 1984). We have provided a few pavement alternatives as an anticipated Average Daily Truck Traffic (ADTT) was not provided. An allowable ADTT should be chosen that is greater than what is expected for the development. PCC pavement section alternatives for lime-treated subgrade are provided in Table 10 below.

**Table 7: PCC Pavement Recommendations, Untreated Subgrade**

| Allowable ADTT | Minimum PCC Thickness (inches) |
|---|---|
| 13 | 5.5 |
| 150 | 6.0 |

**Table 8: PCC Pavement Recommendations, Lime-Treated Subgrade**

| Allowable ADTT | Minimum PCC Thickness (inches) |
|---|---|
| 13 | 5.0 |
| 150 | 5.5 |

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. The designer should consider dowelling all construction and expansion joints where constructed over untreated clayey subgrade soils.

### 9.3 PAVEMENT CUTOFF

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

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

## SECTION 10: RETAINING WALLS

### 10.1 STATIC LATERAL EARTH PRESSURES

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

**Table 9: Recommended Lateral Earth Pressures**

| Wall Condition | Lateral Earth Pressure* | Additional Surcharge Loads |
|---|---|---|
| Unrestrained – Cantilever Wall | 40 pcf | ⅓ of vertical loads at top of wall |
| Restrained – Braced Wall | 40 pcf + 8H** psf | ½ of vertical loads at top of wall |

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

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

### 10.2 SEISMIC LATERAL EARTH PRESSURES

The 2013 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. At this time, we are not aware of any retaining walls for the project except truck-high loading dock walls (i.e. walls 4 feet or less in height). In our opinion, design of these walls for seismic lateral earth pressures in addition to static earth pressures is not warranted.

## 10.3 WALL DRAINAGE

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

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

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

## 10.4 BACKFILL

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 95 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

## 10.5 FOUNDATIONS

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

## SECTION 11: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Seefried Industrial Properties, Inc. specifically to support the design of the Osgood Road Warehouse project in Fremont, 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.

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

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

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

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

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

Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

## SECTION 11: REFERENCES

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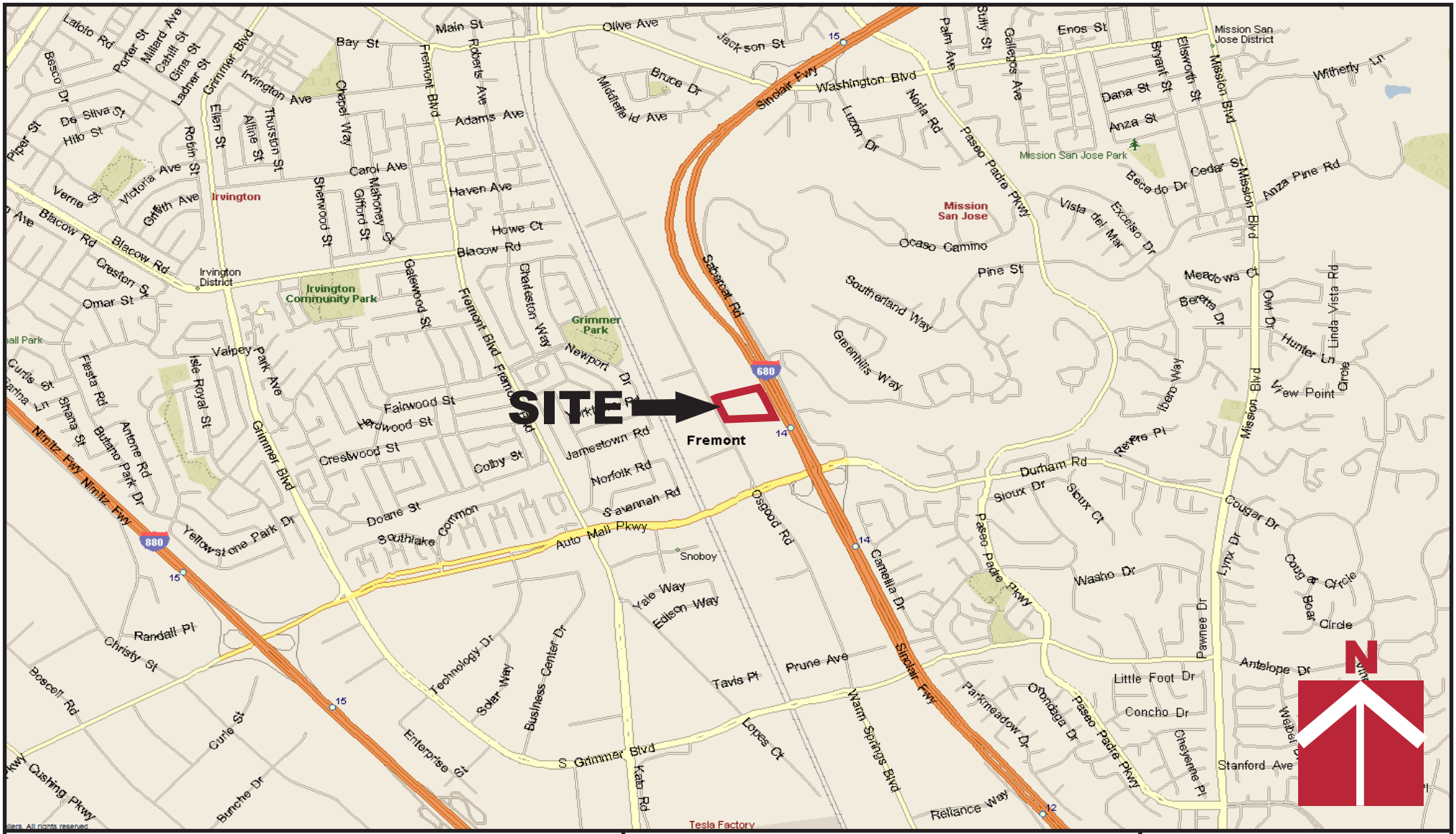
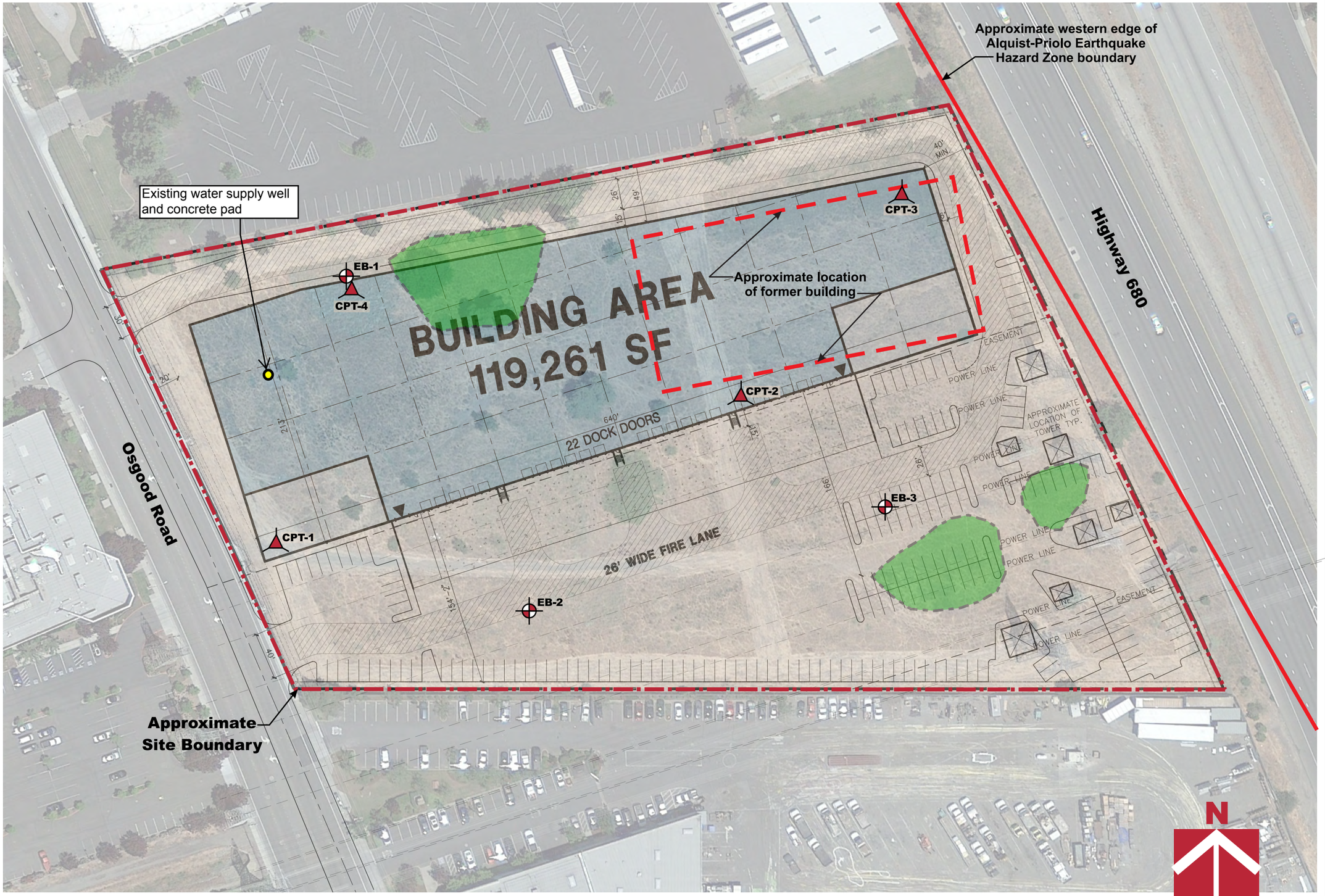



CORNERSTONE EARTH GROUP

**Vicinity Map**

**43510 Osgood Road**
**Fremont, CA**

| | |
|---|---|
| Project Number | 791-3-2 |
| Figure Number | Figure 1 |
| Date | October 2015 |
| Drawn By | RRN |




**Legend**

Approximate lateral extent of existing soil stockpile

Approximate location of exploratory boring (EB)

Approximate location of cone penetration test (CPT)

0 80 160

APPROXIMATE SCALE (FEET)

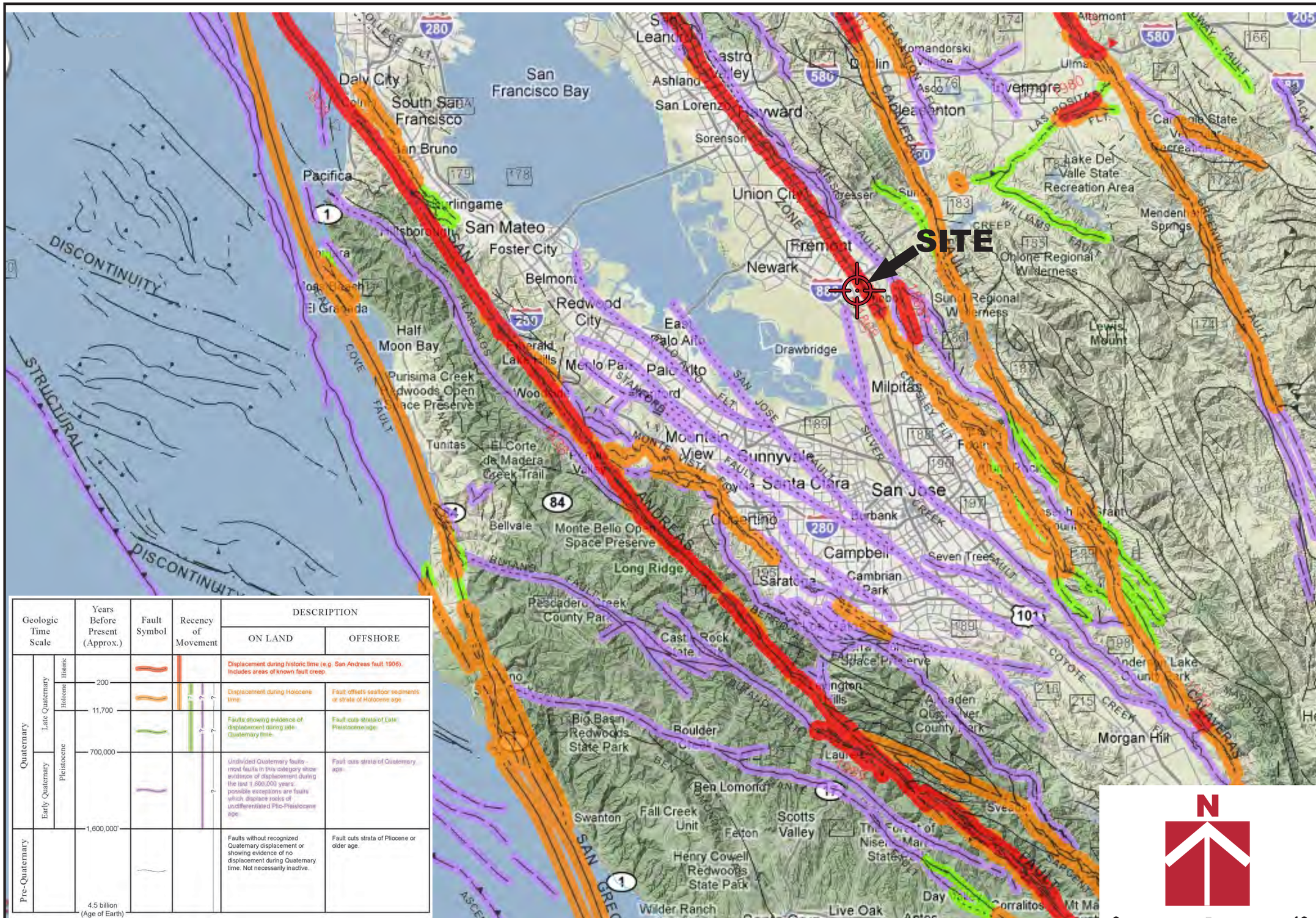
Base by Google Earth, dated 6/9/2014
Overlay by Seefried Industrial Properties, Inc., "Conceptual Site Plan - Scheme 8", dated 8/13/2015

| | |
|---|---|
| Project Number | 791-3-2 |
| Figure Number | Figure 2 |
| Date | October 2015 |
| Drawn By | RRN |

Site Plan

43510 Osgood Road
Fremont, CA

CORNERSTONE EARTH GROUP




| Geologic Time Scale | | | Years Before Present (Approx.) | Fault Symbol | Recency of Movement | DESCRIPTION: ON LAND | DESCRIPTION: OFFSHORE |
|---|---|---|---|---|---|---|---|
| Quaternary | Late Quaternary | Historic | | | | Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep. | |
| | | Holocene | 200 | | ? ? ? | Displacement during Holocene time. | Fault offsets seafloor sediments or strata of Holocene age. |
| | | Pleistocene | 11,700 | | ? ? ? | Faults showing evidence of displacement during late Quaternary time. | Fault cuts strata of Late Pleistocene age. |
| | Early Quaternary | | 700,000 | | ? | 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 | | | 1,600,000 | | | 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. |
| | | | 4.5 billion (Age of Earth) | | | | |

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

Project Number: 791-3-2

Figure Number: Figure 3

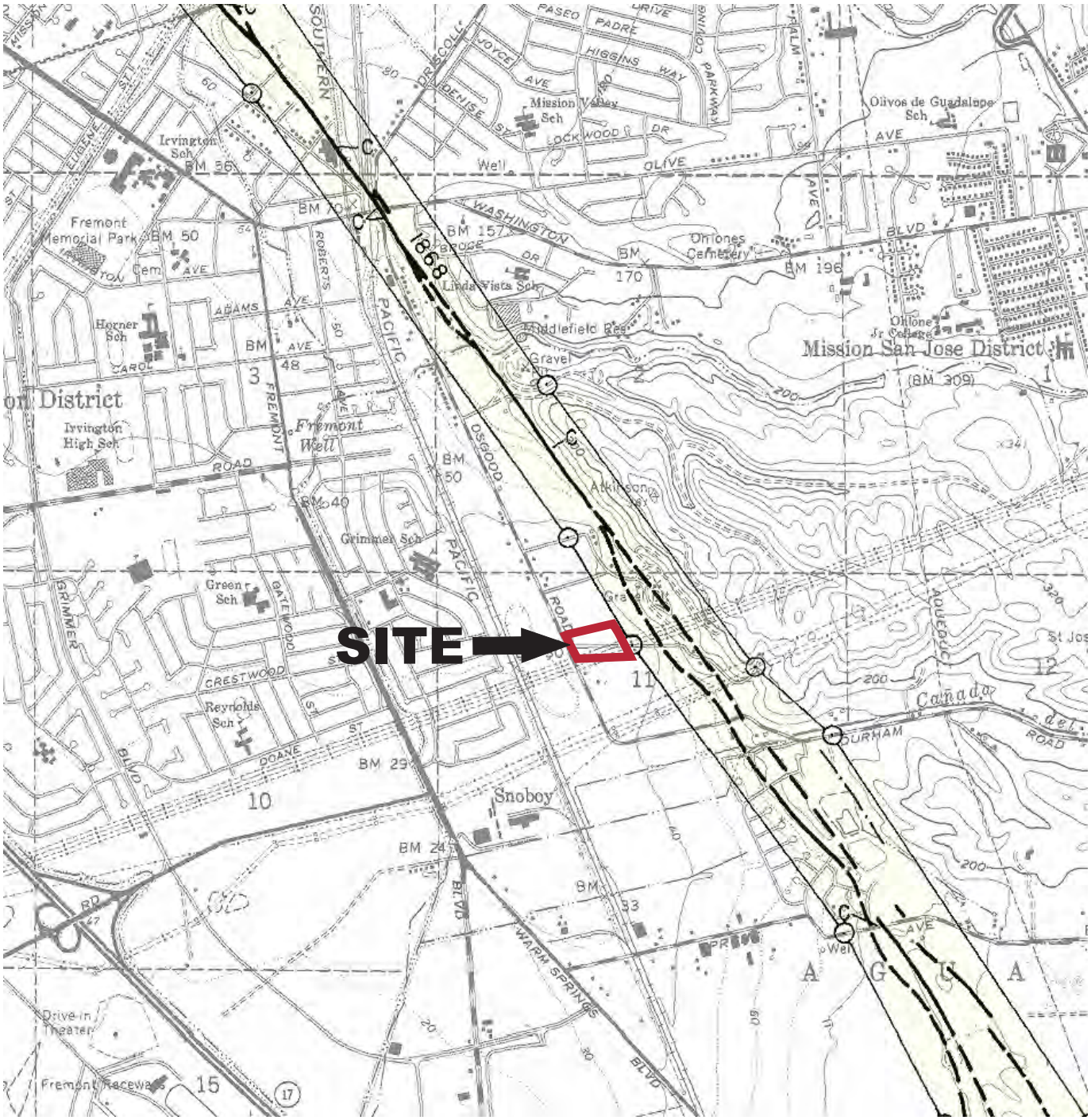
Drawn By: RRN

Date: October 2015

Regional Fault Map

43510 Osgood Road
Fremont, CA

CORNERSTONE EARTH GROUP




**MAP EXPLANATION**

**Potentially Active Faults**

1906 C — Faults considered to have been active during Quaternary time; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by creep or possible creep.

Aerial photo lineaments (not field checked); based on youthful geomorphic and other features believed to be the results of Quaternary faulting.

**Special Studies Zone Boundaries**

These are delineated as straight-line segments that connect encircled turning points so as to define special studies zone segments.

Seaward projection of zone boundary.

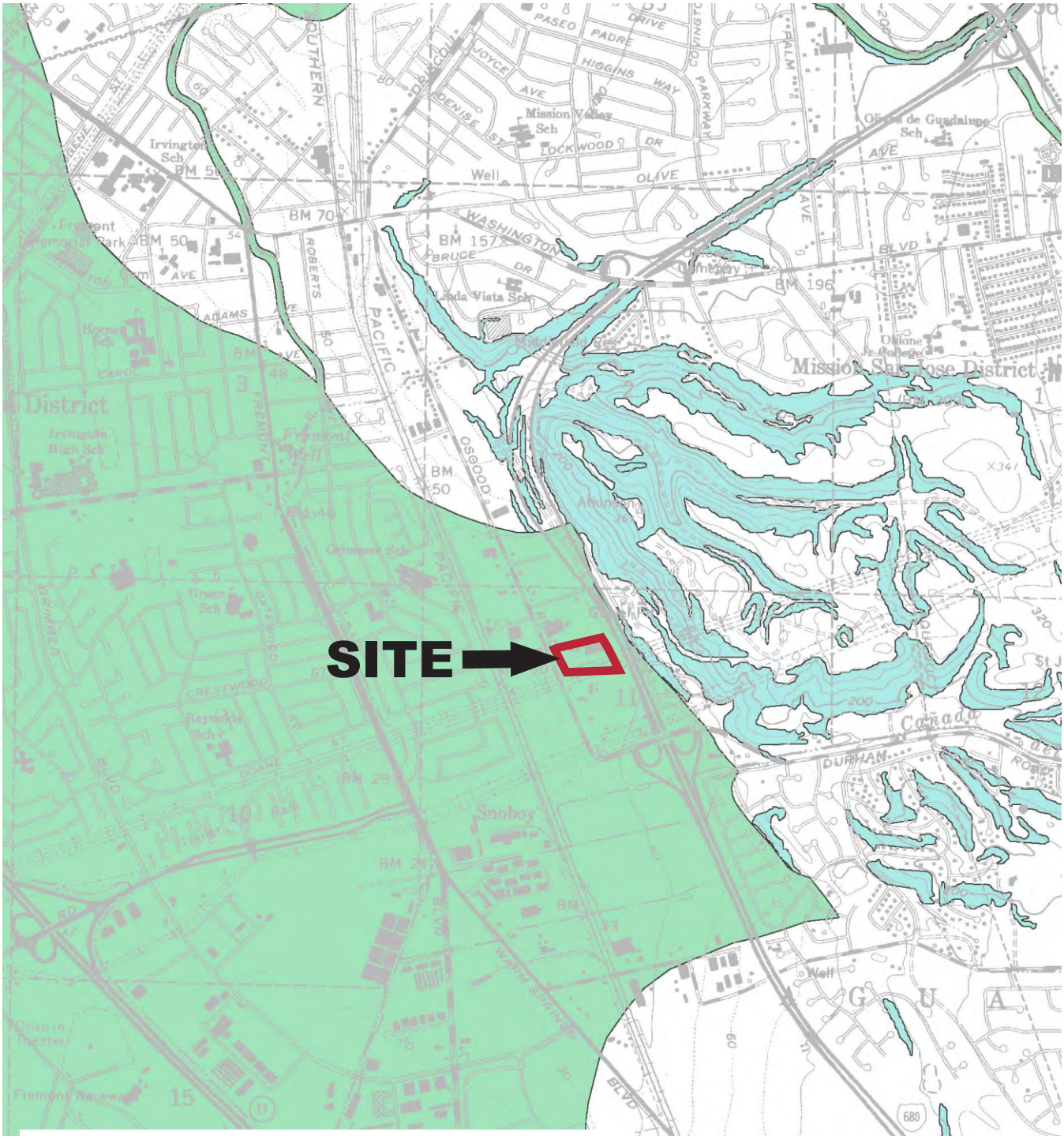
Base: CA Division of Mines and Geology, 1982, Revised official map of Alquist-Priolo Earthquake Fault Hazard Zones, Niles Quadrangle

CORNERSTONE EARTH GROUP

**Alquist-Priolo Earthquake Hazard Zone Map**

**43510 Osgood Road**
**Fremont, CA**

| | |
|---|---|
| Project Number | 791-3-2 |
| Figure Number | Figure 4 |
| Date | October 2015 |
| Drawn By | RRN |




## Explanation

**Liquefaction**

Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

**Earthquake-Induced Landslides**

Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

N

0 2000 4000

APPROXIMATE SCALE (FEET)

Base: CA Geological Survey, 2004, State of CA Seismic Hazard Zone, Niles 7.5-Minute Quadrangle, CA

CORNERSTONE EARTH GROUP

**Liquefaction Hazard Zone Map**

**43510 Osgood Road**
**Fremont, CA**

| | |
|---|---|
| Project Number | 791-3-2 |
| Figure Number | Figure 5 |
| Date | October 2015 |
| Drawn By | RRN |

## 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 October 7, 2015, to depths of 10 to 50 feet. Four CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on October 1, 2015, to depths of approximately 50 feet each. 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.

Exploration locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. Exploration elevations were not determined. The locations of the explorations should be considered accurate only to the degree implied by the method used.

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

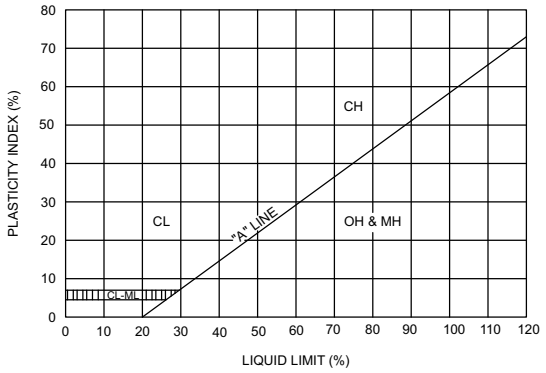
## SAMPLER TYPES

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

## ADDITIONAL TESTS

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

**PLASTICITY CHART**




PENETRATION RESISTANCE
(RECORDED AS BLOWS / FOOT)

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

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

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

CORNERSTONE EARTH GROUP

**LEGEND TO SOIL DESCRIPTIONS**

Figure Number A-1

# BORING NUMBER EB-1



CORNERSTONE EARTH GROUP

**PROJECT NAME** 43510 Osgood Road
**PROJECT NUMBER** 791-3-2
**PROJECT LOCATION** Fremont, CA

**DATE STARTED** 10/7/15 **DATE COMPLETED** 10/7/15
**GROUND ELEVATION** **BORING DEPTH** 50 ft.
**DRILLING CONTRACTOR** Exploration Geoservices, Inc.
**LATITUDE** 37.51787° **LONGITUDE** -121.94418°
**DRILLING METHOD** Mobile B-61, 8 inch Hollow-Stem Auger
**GROUND WATER LEVELS:**
**LOGGED BY** MFR
**AT TIME OF DRILLING** 15 ft.
**NOTES**
**AT END OF DRILLING** 17.3 ft.

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

| ELEVATION (ft) | DEPTH (ft) | SYMBOL | DESCRIPTION | N-Value (uncorrected) blows per foot | SAMPLES TYPE AND NUMBER | DRY UNIT WEIGHT PCF | NATURAL MOISTURE CONTENT, % | PLASTICITY INDEX, % | PERCENT PASSING No. 200 SIEVE | UNDRAINED SHEAR STRENGTH, ksf (○ HAND PENETROMETER, △ TORVANE, ● UNCONFINED COMPRESSION, ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL; 1.0 2.0 3.0 4.0) |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | | **Sandy Lean Clay (CL) [Fill]** hard, moist, brown, fine to coarse sand, some fine subangular gravel, low plasticity Liquid Limit = 29, Plastic Limit = 13 | 54 | MC-1B | 114 | 10 | 16 | | >4.5 |
| | | | **Lean Clay with Sand (CL)** hard, moist, dark brown to brown, fine to medium sand, moderate plasticity | 37 | MC-2B | 112 | 11 | | | >> |
| | 5 | | **Lean Clay (CL)** very stiff, moist, gray brown, some fine to medium sand, moderate plasticity | 28 | MC-3B | 111 | 18 | | | |
| | | | Liquid Limit = 36, Plastic Limit = 15 | 28 | MC-4B | 110 | 19 | 21 | 88 | |
| | 10 | | | 33 | MC | | | | | |
| | | | **Sandy Lean Clay (CL)** medium stiff, moist, brown, fine to medium sand, low plasticity Liquid Limit = 31, Plastic Limit = 14 | 37 | MC-6B | 112 | 16 | | 69 | |
| | 15 | | | 20 | MC-7B | 108 | 20 | 17 | 66 | |
| | | | **Lean Clay with Sand (CL)** very stiff, moist, brown, fine to medium sand, moderate plasticity | 42 | MC | | | | | |
| | 20 | | | 49 | MC-9A | 112 | 17 | | | |
| | | | **Sandy Lean Clay (CL)** very stiff, moist, brown, fine to medium sand, low plasticity Liquid Limit = 31, Plastic Limit = 14 | 58 | MC-10B | 114 | 17 | 17 | 64 | |
| | 25 | | | 68 | MC | | | | | |
| | | | | 51 | MC | | | | | |
| | 30 | | **Lean Clay with Sand (CL)** very stiff to hard, moist, brown, fine to medium sand, moderate plasticity | 76 | MC-13A | 111 | 18 | | | |
| | | | | 50 / 6" | MC-14B | 111 | 18 | | | >4.5 |
| | 35 | | | | | | | | | |

*Continued Next Page*

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 10/23/15 10:11 - P:\DRAFTING\GINT FILES\791-3-2 OSGOOD RD.GPJ

# BORING NUMBER EB-1



CORNERSTONE EARTH GROUP

**PROJECT NAME** 43510 Osgood Road

**PROJECT NUMBER** 791-3-2

**PROJECT LOCATION** Fremont, CA

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

| ELEVATION (ft) | DEPTH (ft) | SYMBOL | DESCRIPTION | N-Value (uncorrected) blows per foot | SAMPLES TYPE AND NUMBER | DRY UNIT WEIGHT PCF | NATURAL MOISTURE CONTENT, % | PLASTICITY INDEX, % | PERCENT PASSING No. 200 SIEVE | UNDRAINED SHEAR STRENGTH, ksf (○ HAND PENETROMETER, △ TORVANE, ● UNCONFINED COMPRESSION, ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL) 1.0 2.0 3.0 4.0 |
|---|---|---|---|---|---|---|---|---|---|---|
| | 35 | | | | | | | | | |
| | | | **Sandy Lean Clay (CL)** medium stiff, moist, brown, fine to medium sand, low plasticity | 26 | MC-15B | 106 | 20 | | 52 | ○ |
| | 40 | | | | | | | | | |
| | | | **Silty Sand (SM)** medium dense, moist, brown, fine sand | | | | | | | |
| | | | **Sandy Lean Clay (CL)** medium stiff, moist, brown, fine to medium sand, low plasticity | 27 | MC-16B | 102 | 23 | | | ○ |
| | 45 | | | | | | | | | |
| | | | **Lean Clay with Sand (CL)** very stiff, moist, brown, fine to medium sand, moderate plasticity | 47 | MC-17A | 108 | 19 | | | ○ |
| | 50 | | Bottom of Boring at 50.0 feet. | | | | | | | |
| | 55 | | | | | | | | | |
| | 60 | | | | | | | | | |
| | 65 | | | | | | | | | |
| | 70 | | | | | | | | | |
| | 75 | | | | | | | | | |

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 10/23/15 10:11 - P:\DRAFTING\GINT FILES\791-3-2 OSGOOD RD.GPJ

# BORING NUMBER EB-2



CORNERSTONE EARTH GROUP

**PROJECT NAME** 43510 Osgood Road
**PROJECT NUMBER** 791-3-2
**PROJECT LOCATION** Fremont, CA

**DATE STARTED** 10/7/15 **DATE COMPLETED** 10/7/15
**GROUND ELEVATION** **BORING DEPTH** 10 ft.
**DRILLING CONTRACTOR** Exploration Geoservices, Inc.
**LATITUDE** 37.51711° **LONGITUDE** -121.94361°
**DRILLING METHOD** Mobile B-61, 8 inch Hollow-Stem Auger
**GROUND WATER LEVELS:**
**LOGGED BY** MFR
**AT TIME OF DRILLING** Not Encountered
**NOTES**
**AT END OF DRILLING** Not Encountered

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

| ELEVATION (ft) | DEPTH (ft) | SYMBOL | DESCRIPTION | N-Value (uncorrected) blows per foot | SAMPLES TYPE AND NUMBER | DRY UNIT WEIGHT PCF | NATURAL MOISTURE CONTENT, % | PLASTICITY INDEX, % | PERCENT PASSING No. 200 SIEVE | UNDRAINED SHEAR STRENGTH, ksf (○ HAND PENETROMETER, △ TORVANE, ● UNCONFINED COMPRESSION, ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL) 1.0 2.0 3.0 4.0 |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | | **Lean Clay with Sand (CL)** hard, moist, dark brown to brown, fine to medium sand, moderate plasticity | 16 | MC-1B | 100 | 11 | | | >4.5 |
| | | | | 22 | MC | | | | | >4.5 |
| | 5 | | **Lean Clay (CL)** very stiff, moist, gray brown, some fine sand, moderate plasticity | 20 | MC-3A | 112 | 17 | | | |
| | | | **Sandy Lean Clay (CL)** stiff, moist, brown, fine to medium sand, low plasticity | 17 | MC-4B | 111 | 17 | | | |
| | 10 | | Bottom of Boring at 10.0 feet. | | | | | | | |
| | 15 | | | | | | | | | |
| | 20 | | | | | | | | | |
| | 25 | | | | | | | | | |
| | 30 | | | | | | | | | |
| | 35 | | | | | | | | | |

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 10/23/15 10:11 - P:\DRAFTING\GINT FILES\791-3-2 OSGOOD RD.GPJ

# BORING NUMBER EB-3



CORNERSTONE EARTH GROUP

**PROJECT NAME** 43510 Osgood Road
**PROJECT NUMBER** 791-3-2
**PROJECT LOCATION** Fremont, CA

**DATE STARTED** 10/7/15 **DATE COMPLETED** 10/7/15
**GROUND ELEVATION** **BORING DEPTH** 10 ft.
**DRILLING CONTRACTOR** Exploration Geoservices, Inc.
**LATITUDE** 37.51738° **LONGITUDE** -121.94258°
**DRILLING METHOD** Mobile B-61, 8 inch Hollow-Stem Auger
**GROUND WATER LEVELS:**
**LOGGED BY** MFR
**AT TIME OF DRILLING** 9 ft.
**NOTES**
**AT END OF DRILLING** Not Encountered

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

| ELEVATION (ft) | DEPTH (ft) | SYMBOL | DESCRIPTION | N-Value (uncorrected) blows per foot | SAMPLES TYPE AND NUMBER | DRY UNIT WEIGHT PCF | NATURAL MOISTURE CONTENT, % | PLASTICITY INDEX, % | PERCENT PASSING No. 200 SIEVE | UNDRAINED SHEAR STRENGTH, ksf (○ HAND PENETROMETER, △ TORVANE, ● UNCONFINED COMPRESSION, ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL) |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | | **Sandy Lean Clay (CL) [Fill]** hard, moist, brown, fine to coarse sand, some fine subangular gravel, low plasticity | 26 | MC-1A | 103 | 12 | | | >4.5 |
| | | | **Lean Clay with Sand (CL)** hard to very stiff, moist, dark brown to brown, fine to medium sand, moderate plasticity | 22 | MC-2A | 106 | 15 | | | |
| | 5 | | **Sandy Lean Clay (CL)** very stiff, moist, brown, fine to medium sand, low plasticity | 36 | MC | | | | | |
| | | | **Clayey Sand (SC)** medium dense, moist, brown, fine to medium sand, some fine subangular gravel | 28 | MC-4A | 109 | 18 | | | |
| | 10 | | Bottom of Boring at 10.0 feet. | | | | | | | |
| | 15 | | | | | | | | | |
| | 20 | | | | | | | | | |
| | 25 | | | | | | | | | |
| | 30 | | | | | | | | | |
| | 35 | | | | | | | | | |

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

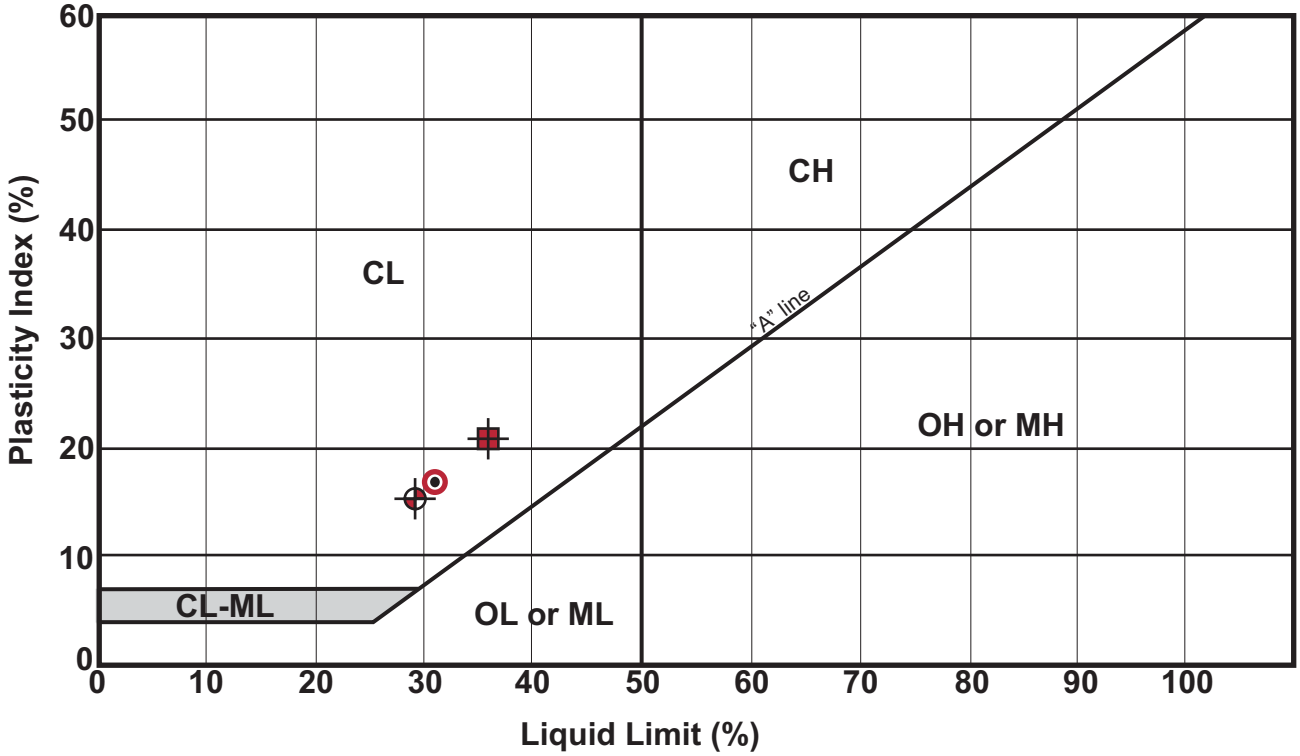
**Dry Densities:** In place dry density determinations (ASTM D2937) were performed on 19 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

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

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

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

## 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 | 10 | 29 | 13 | 16 | — | Sandy Lean Clay (CL) [Fill] |
| | EB-1 | 9.0 | 19 | 36 | 15 | 21 | 88 | Lean Clay (CL) |
| | EB-1 | 16.0 | 20 | 31 | 14 | 17 | 66 | Sandy Lean Clay (CL) |
| | EB-1 | 24.5 | 17 | 31 | 14 | 17 | 64 | Sandy Lean Clay (CL) |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |

CORNERSTONE EARTH GROUP

Plasticity Index Testing Summary

43510 Osgood Road
Fremont, CA

Project Number: 791-3-2

Figure Number: Figure B1

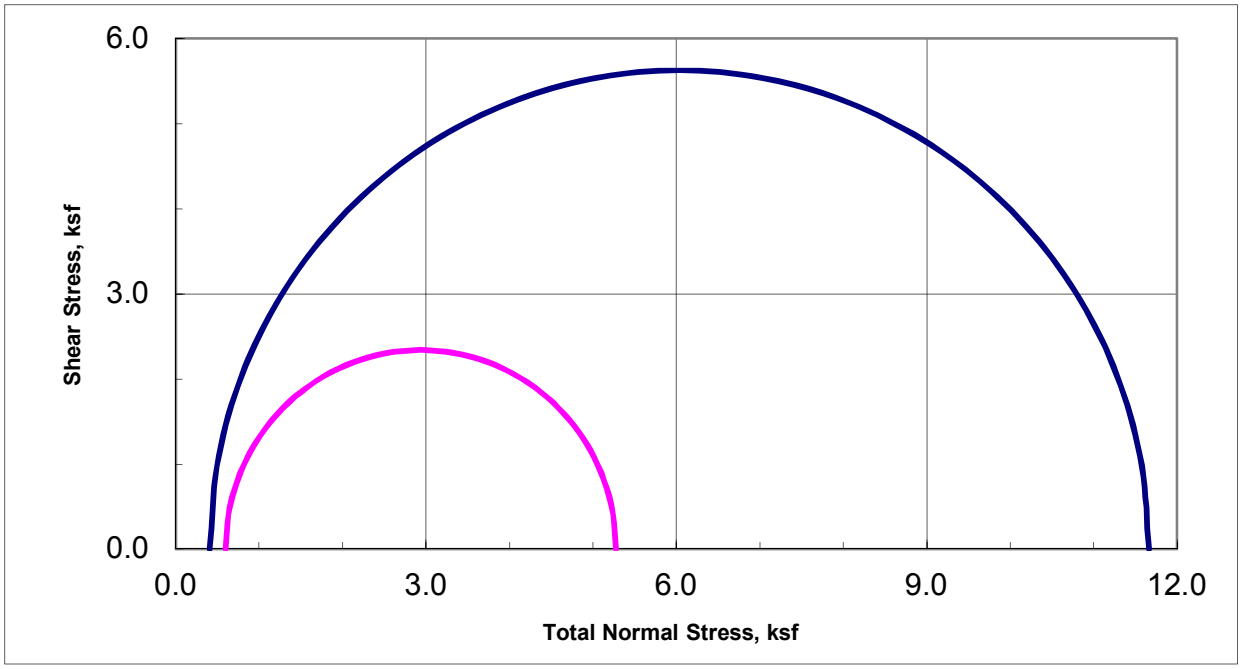
Date: October 2015

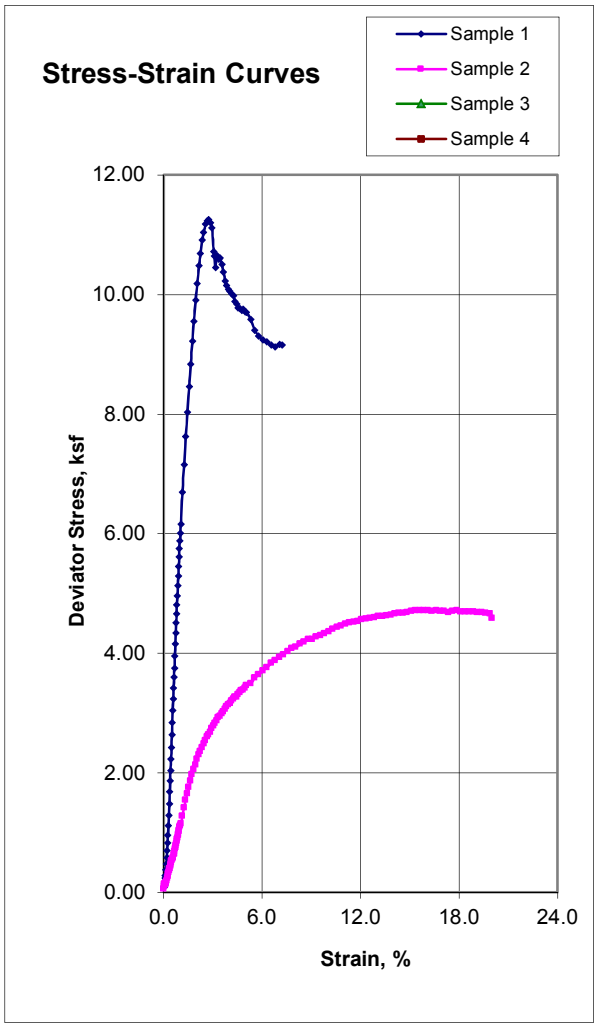
Drawn By: FLL

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

# Unconsolidated-Undrained Triaxial Test

ASTM D2850



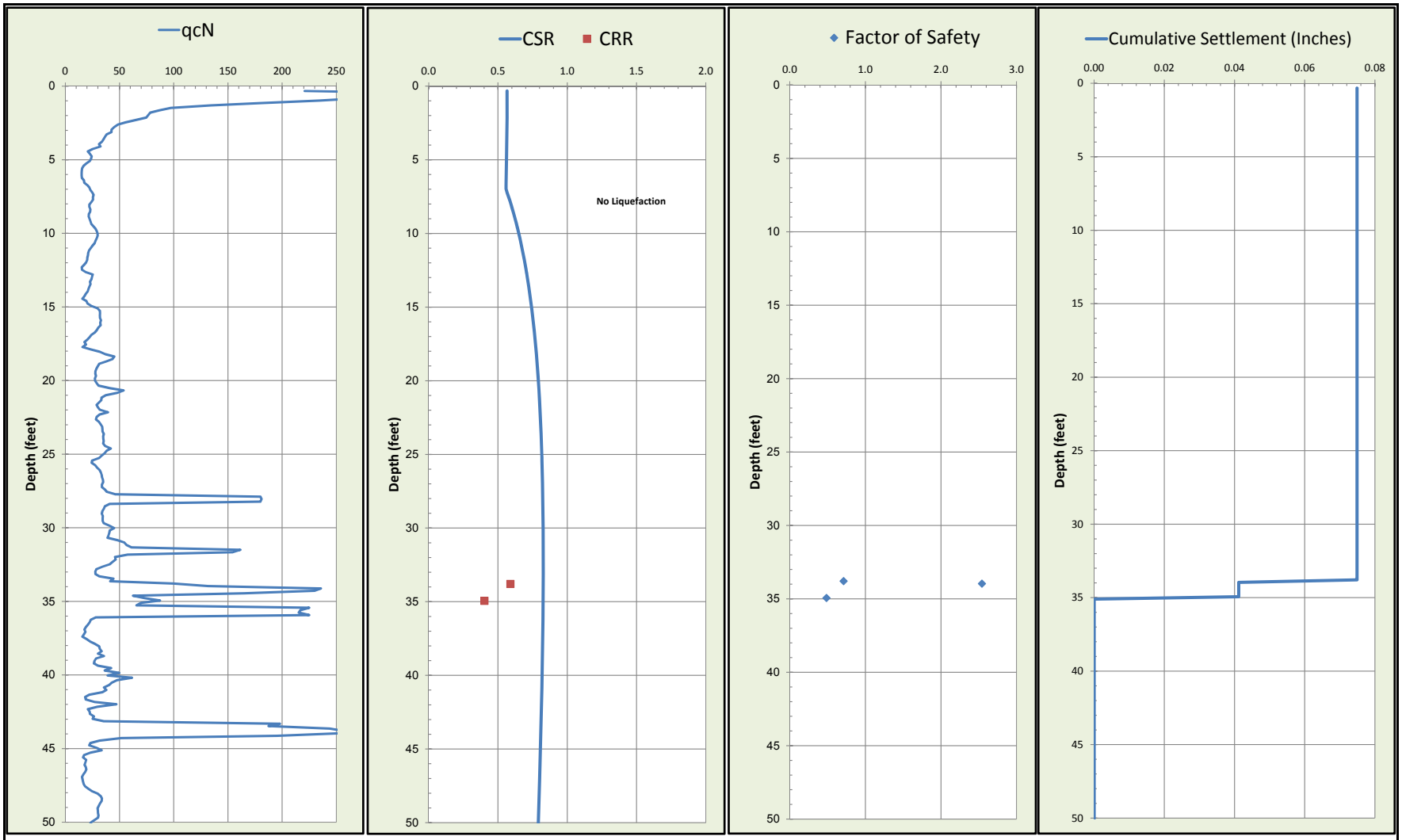

| Sample Data | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Moisture % | 10.5 | 18.2 | | |
| Dry Den,pcf | 112.1 | 111.3 | | |
| Void Ratio | 0.504 | 0.514 | | |
| Saturation % | 56.4 | 95.5 | | |
| Height in | 5.04 | 5.06 | | |
| Diameter in | 2.39 | 2.40 | | |
| Cell psi | 3.5 | 4.2 | | |
| Strain % | 2.74 | 15.00 | | |
| Deviator, ksf | 11.254 | 4.681 | | |
| Rate %/min | 1.00 | 1.00 | | |
| in/min | 0.050 | 0.050 | | |
| Job No.: | 640-896 | | | |
| Client: | Cornerstone Earth Group | | | |
| Project: | Osgood Rd Warehouse - P5672 | | | |
| Boring: | EB-1 | EB-1 | | |
| Sample: | 2B | 3B | | |
| Depth ft: | 4.0 | 6.0 | | |

| Visual Soil Description | |
|---|---|
| Sample # | |
| 1 | Dark Gray Sandy CLAY |
| 2 | Olive Brown CLAY w/ Sand |
| 3 | |
| 4 | |

Remarks:

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

# APPENDIX C: LIQUEFACTION ANALYSES CALCULATIONS




CORNERSTONE EARTH GROUP

**Liquefaction Analysis Summary**

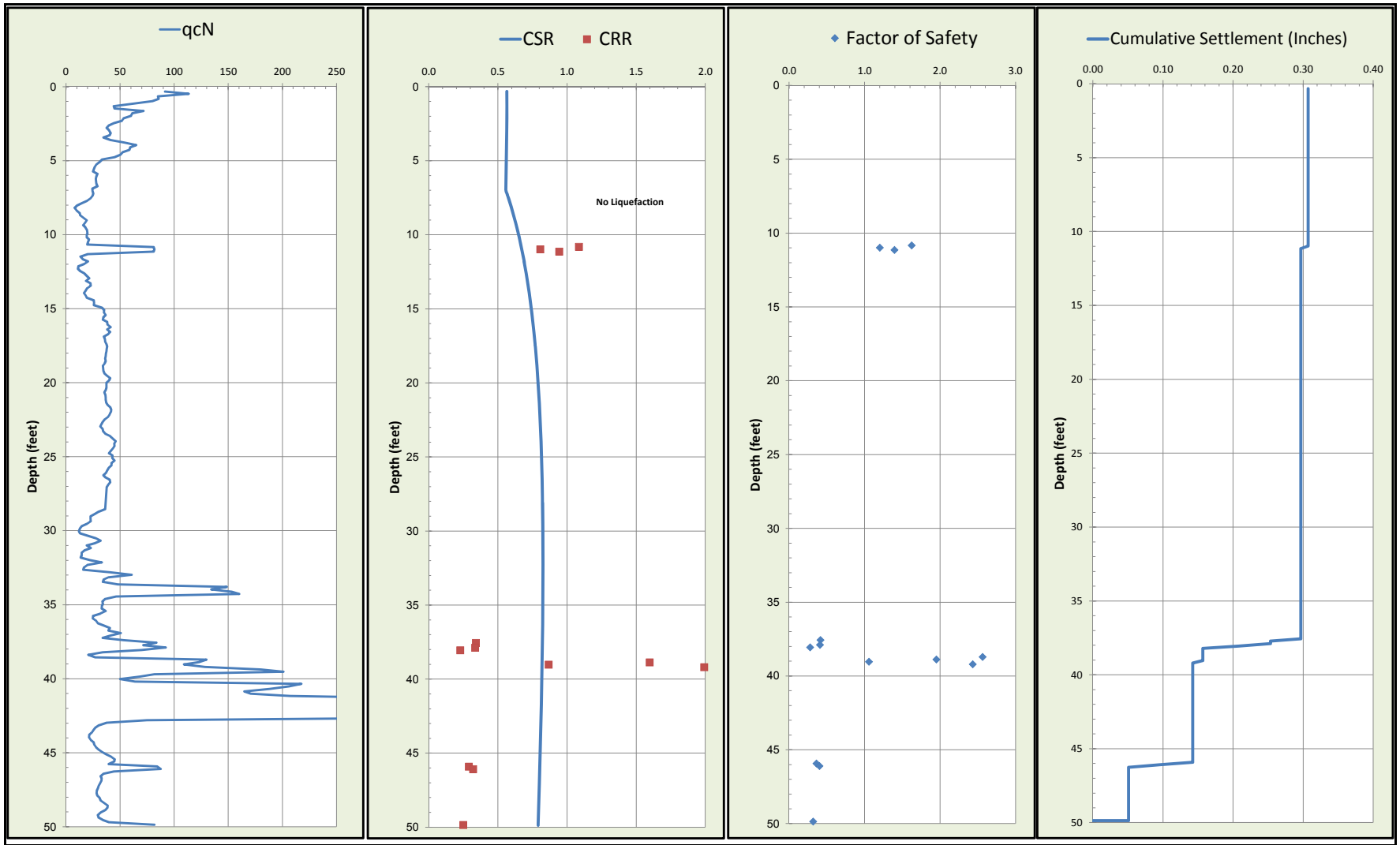
**43510 Osgood Road**
**Fremont, California**

Project Number
791-3-2

Figure Number
Figure C-1

10/27/2015

CPT No. 1




CORNERSTONE EARTH GROUP

**Liquefaction Analysis Summary**

**43510 Osgood Road**

**Fremont, California**

Project Number

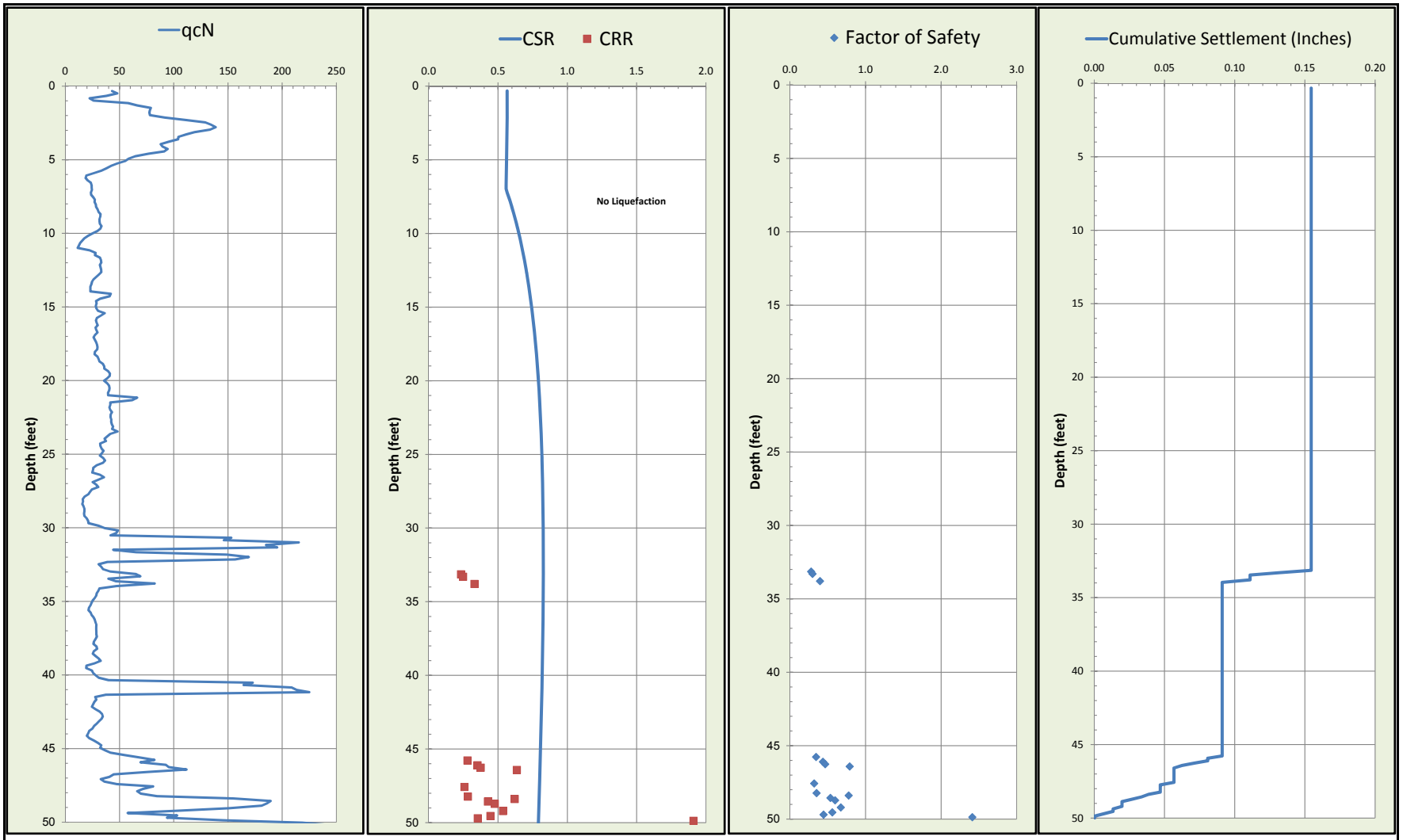
791-3-2

Figure Number

Figure C-2

10/27/2015

CPT No. 2




CORNERSTONE EARTH GROUP

**Liquefaction Analysis Summary**

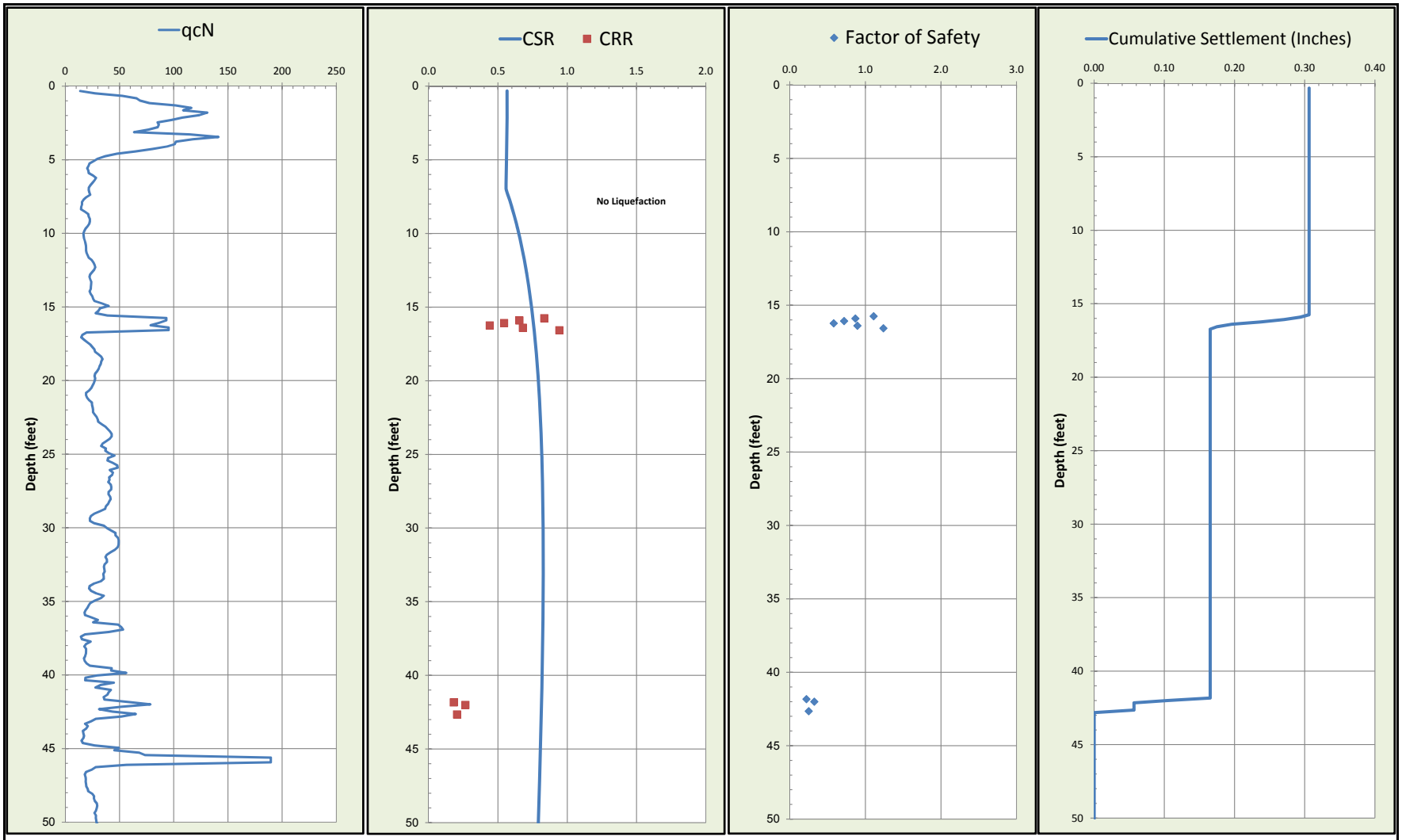
**43510 Osgood Road**
**Fremont, California**

Project Number
791-3-2

Figure Number
Figure C-3

10/27/2015

CPT No. 3




CORNERSTONE EARTH GROUP

**Liquefaction Analysis Summary**

**43510 Osgood Road**
**Fremont, California**

Project Number
791-3-2

Figure Number
Figure C-4

10/27/2015

CPT No. 4