Appendix D

Geotechnical Investigation

CORNERSTONE
EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**2305 Mission College Boulevard Data Center**
Location	**2305 Mission College Boulevard** **Santa Clara, California**
Client	**Aligned Data Centers**
Client Address	**980 Avenue of the Americas, Suite 406** **New York, New York**
Project Number	**930-1-1**
Date	**January 18, 2016**

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SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Aligned Data Centers for the 2305 Mission College Boulevard Data Center project in Santa Clara, 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 preliminary (draft) site plan titled "Aligned Data Centers Santa Clara," Sheet A – Site Plan, prepared by CAC Architects, dated September 21, 2016.
- A preliminary (draft) phasing and site plan titled "Aligned Data Centers Santa Clara," Sheet A – Site Plan, prepared by CAC Architects, dated September 21, 2016.
- A flood analysis letter titled "2305 Mission College Boulevard 500-year and 1000-year Flood Analysis Summary," prepared by Schaaf & Wheeler Consulting Civil Engineers, dated September 30, 2016.
- An ALTA survey titled "ALTA/ACSM Land Title Survey, For: 2305 MCB, LLC, 2305 Mission College Boulevard, Santa Clara, California," Sheet 2, prepared by Kier & Wright Civil Engineers & Surveyors, Inc., dated October 27, 2014.

1.1 PROJECT DESCRIPTION

The project will consist of demolishing the existing building and improvements at the site and constructing a new 2-level, steel-framed data center building with an approximate 201,000-square-foot footprint. Site improvements will also consist of a substation and associated data center structures/equipment including transformers, switchgear lineups, inverter modules, water tanks, and generators. Appurtenant parking, drive aisles, utilities, landscaping, and other improvements necessary for site development are also planned.

Based on the preliminary building loading you provided, dead plus live columns loads for the data hall and electric rooms with mezzanines are 516 kips and 427 kips, respectively. Based on the associated structure/equipment loads you provided, diesel generators with belly tanks are to be 98 kips, pad mount transformers are to be 15 kips, switchgear lineups are to be 27 to 48 kips, utility transformers are to be 50 kips, UPS sections are to be 47 kips, single and double stack inverter modules are to be 46 and 100 kips, and 20,000 gallon water tanks are to be 167 kips.

Based on the flood analysis letter provided and correspondence with you, we understand the overall site grades will be raised to Elevation 25 feet (NAVD88), which is two feet above the FEMA 100 year flood elevation and the building's finished floor elevation will be Elevation 27 feet (two feet above the highest grade) . The highest grades will be around the building perimeter and slope down to the street level along Mission College Boulevard and Agnew Road. An approximately 3- to 4-foot high retaining wall will be constructed along the eastern property line. At this time, we have not been provided a topographic survey of the existing site grades. However, based on the above information and elevations provided by Google Earth, it appears site grades will be raised about 0 to 5 feet above existing grades.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated June 21, 2016, and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of five borings drilled on December 9, 2016, with truck-mounted, hollow-stem auger drilling equipment and six Cone Penetration Tests (CPTs) advanced on November 14, 2016. The borings were drilled to depths of about 20 to 39½ feet; the CPTs were advanced to depths of approximately 40 to 101 feet. Seismic shear wave velocity measurements were collected from CPT-5. Borings EB-1, EB-3, EB-4, and EB-5 were advanced adjacent to CPT-1, CPT-3, CPT-4, and CPT-5, respectively, for direct evaluation of physical samples to correlated soil behavior.

The borings and CPTs were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions. The approximate locations of our exploratory borings and CPTs are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

1.4 LABORATORY TESTING PROGRAM

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

1.5 CORROSION EVALUATION

Three samples from our borings at depths of 1 to 4 feet were tested for saturated resistivity, pH, and soluble sulfates and chlorides. JDH Corrosion Consultants prepared a brief corrosion evaluation based on the laboratory data, which is attached to this report in Appendix C. In general, the on-site soils can be characterized as corrosive to buried metal, and non-corrosive to corrosive to buried concrete.

1.6 ENVIRONMENTAL SERVICES

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

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

The site is located within 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. Alluvial soil thickness in the area of the site is greater than 500 feet (Rogers & Williams, 1974).

2.2 REGIONAL SEISMICITY

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

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

Table 1: Approximate Fault Distances

Fault Name	Distance (miles)	Distance (kilometers)
Hayward (Southeast Extension)	6.3	10.1
Monte Vista-Shannon	7.8	12.6
Hayward (Total Length)	8.8	14.1
Calaveras	9.9	16.0
San Andreas (1906)	11.3	18.2

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

SECTION 3: SITE CONDITIONS

3.1 SURFACE DESCRIPTION

The site is bounded by San Thomas Aquino Creek to the west, Agnew Road to the north, one- and two-story technology buildings to the east, and Mission College Boulevard to the south. The site is currently developed with a two-story building and surrounding asphalt parking lots. Landscaping areas containing grass, shrubs, and mature trees are generally scattered throughout the parking lots, around the perimeter of the site, and along the south side of the existing building.

The site is relatively flat with Elevations of about 19 to 25 feet (Google Earth, 2016) and is graded slightly up to the existing building and to drain to storm drain facilities. The San Thomas Aquino Creek parallels the west side of the property, has a levee extending roughly 6½ to 7 feet above the adjacent site grades, has side slopes at roughly 2:1 (H:V) to 3:1 (H:V), and is about 12 to 14 feet deep below the adjacent site grades.

Surface pavements generally consisted of 1 to 3 inches of asphalt concrete over 2 to 4 inches of aggregate base. Based on visual observations, the existing pavements range from generally good to poor condition, with areas of significant alligator cracking.

3.2 SUBSURFACE CONDITIONS

Below the surface pavement sections, Boring EB-4 encountered undocumented fill consisting of clayey sand to a depth of 2 feet below the surface. Below the undocumented fill at Boring EB-4 and the surface pavements at our other explorations, our explorations generally encountered stiff to hard lean clays with variable amounts of sand. The lean clays were interbedded with some loose to dense layers of silty, clayey, and poorly graded sands with variable amounts of silt, clay, and gravel. Some larger, about 8 to 12 foot thick layers of sand were encountered at depths ranging from about 12 to 24 feet in Borings EB-1 and EB-4 and the paired CPT-1 and CPT-4. An approximate 5-foot thick sandy silt layer was encountered at a depth of about 9 feet in Boring EB-2 and our deeper CPT exploration generally inferred a clayey silt to silty clay profile below a depth of about 50 feet.

3.2.1 Plasticity/Expansion Potential

We performed one Plasticity Index (PI) test on a representative sample. The test result was used to evaluate the expansion potential of the surficial soils. The result of the PI test indicated a PI of 31, indicating high expansion potential to wetting and drying cycles.

3.2.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from near optimum to about 8 to 10 percent over the estimated laboratory optimum moisture.

3.3 GROUND WATER

Ground water was encountered in our borings at depths of 8 to 11 feet below existing grades. Ground water was inferred at depths of approximately 13, 3, 13½, and 10 feet below current grades in CPT-1, CPT-3, CPT-4, and CPT-5, respectively, based on pore pressure dissipation tests. Historic high ground water levels are mapped at a depth of approximately 6 feet below current grades (CGS, Milpitas 7.5 Minute Quadrangle, 2001). In general, fluctuations in ground water levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors. Based on the above information, we anticipate a high ground water level of 6 feet below existing grades and recommend a ground water level of 6 feet be used for design.

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 or a Santa Clara County Fault Hazard Zone. As shown in Figure 3, no known surface expression of fault traces is thought to cross the site; therefore, fault rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration (PGA_M) was estimated for analysis using a value equal to F_{PGA} x PGA, as allowed in the 2016 edition of the California Building Code. For our liquefaction analysis we used a PGA of 0.500g.

4.3 LIQUEFACTION POTENTIAL

The site is within a State-designated Liquefaction Hazard Zone (CGS, Milpitas Quadrangle, 2004) as well as a Santa Clara County Liquefaction Hazard Zone (Santa Clara County, 2004). Our field and laboratory programs addressed this issue by testing and sampling potentially liquefiable layers to depths of at least 50 feet, performing visual classification on sampled materials, evaluating CPT data, and performing various tests to further classify soil properties.

4.3.1 Background

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

4.3.2 Analysis

As discussed in the "Subsurface" section above, several sand layers were encountered below the design ground water depth of 6 feet. Following the liquefaction analysis framework in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008), incorporating updates in *CPT and SPT Based Liquefaction Triggering Procedures* (Boulanger and Idriss, 2014), and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008) for quantitative analysis, these layers were analyzed for liquefaction triggering and potential post-liquefaction settlement. These methods compare the ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil's estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.3 are considered to be potentially liquefiable and capable of post-liquefaction re-consolidation (i.e. settlement).

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

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

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

The results of our CPT analyses (CPT-1 to CPT-6) are presented on Figures 4A to 4F of this report. Calculations for these CPTs are attached as Appendix D.

4.3.3 Summary

Our analyses indicate that several layers could potentially experience liquefaction triggering that could result in post-liquefaction total settlement at the ground surface ranging from less than ¼ inch to 1 inch based on the Yoshimine (2006) method. At locations within the proposed building area, our CPT analyses indicate post-liquefaction total settlement at the ground surface ranging from less than ¼ inch to ⅔ inch. As discussed in Special Publication 117A, differential movement for level ground sites over deep soil sites will be up to about two-thirds of the total settlement between independent foundation elements. In our opinion, differential settlements are anticipated to be on the order of ½-inch between independent foundation elements for the proposed building and on the order of ⅔-inch between independent foundation elements for the supplemental structures/equipment areas.

4.3.4 Ground Rupture Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground rupture or sand boils. For ground rupture to occur, the pore water pressure within the liquefiable soil layer will need to be great enough to break through the overlying non-liquefiable layer, which could cause significant ground deformation and settlement. The work of Youd and Garris (1995) indicates that the 9-foot and greater thick layer of non-liquefiable cap is sufficient to prevent ground rupture; therefore, the above total settlement estimates are reasonable.

4.4 LATERAL SPREADING

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

The top of the eastern bank of the San Thomas Aquino Creek is located as close as about 30 feet west of the project site boundary, and has an estimated bank height of about 10 to 14 feet, based on site observations and elevations provided by Google Earth®. In general, lateral spreading is considered when an open face (Height = D) is within about 40D of a site. Since the project site is within this criteria, we analyzed the site for lateral spreading using analytical methods outlined in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008) and *CPT and SPT Based Liquefaction Triggering Procedures* (Boulanger and Idriss, 2014) by calculating Lateral Displacement Index (LDI) values at each CPT location. The LDI is calculated by integrating maximum shear strains versus depth, representing a measure of the potential maximum displacement (Zhang et al., 2004).

At our exploration locations closest to and adjacent to San Thomas Aquino Creek (CPT-1 and CPT-4) our analyses indicates potential for lateral displacement with LDI values of 0.81 and 0.79, respectively, and potential lateral displacements ranging from 0.4 to 1.6 feet. At our other exploration locations to the east of CPT-1 and CPT-4 and generally in the location of the proposed data center building, our analyses indicate LDI values of 0.0 to 0.02 and potential lateral displacement of 0.0 feet.

Based on the above, the potential for lateral displacement affecting the proposed data center building appears low. However, the potential for lateral spreading appears possible to affect the proposed substation and associated data center structures/equipment located between the creek and the west side of the proposed data center building. To protect these improvements, a shear key should be constructed between the creek and the western border of improvements. If desired, to further evaluate the horizontal distance into the site at which the potential for lateral spreading appears possible, further CPT exploration should be performed between CPT-1 and CPT-4 and the western side of the proposed data center building.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. As the soils encountered at the site above the design ground water depth of 6 feet below the existing ground surface were predominantly stiff to hard clays, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low.

4.6 TSUNAMI/SEICHE

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

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

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the study of tsunami inundation potential for the San Francisco Bay Area (Ritter and Dupre, 1972), areas most likely to be inundated are marshlands, tidal flats, and former bay margin lands that are now artificially filled, but are still at or below sea level, and are generally within 1½ miles of the shoreline. The site is approximately 5½ miles inland from the San Francisco Bay shoreline, and is approximately 19 to 25 feet above mean sea level according to Google Earth®. Therefore, the potential for inundation due to tsunami or seiche is considered low.

4.7 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone X and Zone AH. Zone X is described as "Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage area less than 1 square mile; and areas protected by levees from 1% annual chance flood." Zone AH is described as "Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevation determined to be Elevation 23 feet." We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

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

- Potential for significant static and seismic settlements
- Potential for lateral spreading
- Shallow ground water
- Highly expansive soils
- Undocumented fill

- Potential for fill settlement
- Soil corrosion potential

5.1.1 Potential for Significant Static and Seismic 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 differential seismic movement from liquefaction could be on the order of ½-inch between independent foundation elements for the proposed data center building and on the order of ⅔-inch between independent foundation elements for the supplemental structures/equipment areas outside the building along San Thomas Aquino Creek.

In addition to seismic settlement, we have analyzed static settlements due to the static dead plus live column loads provided for the proposed data center building. We estimate total static settlement for conventional shallow footings would be up to about 1¾ inches, resulting in approximately 1 inch of post-construction differential settlement between independent foundation elements for the data center building.

The building foundations will need to be designed to tolerate total and differential settlements due to static loads and liquefaction-induced settlement. Detailed foundation recommendations are presented in the “Foundations” section.

5.1.2 Potential for Lateral Spreading

As previously discussed, there is a potential for lateral spreading towards the adjacent San Thomas Aquino Creek. Lateral spreading appears possible for the substation and associated data center structures/equipment located to the west of the proposed data center building. However, the potential for lateral spreading does not appear to extend to the proposed data center building and therefore appears to be low at the location of the proposed building. If desired to protect the substation and associated data center structures/equipment to the west of the proposed building, the site can be mitigated to reduce the potential for lateral spreading. Typical techniques to mitigate the potential for lateral spreading include ground improvement to construct a shear key or the installation of shear (pin) piles to effectively create a shear key. If mitigation recommendations are desired, we should be retained to provide design recommendations. Additionally, to further evaluate the horizontal distance into the site at which lateral spreading does not appear possible, further CPT exploration should be performed.

5.1.3 Shallow Ground Water

Shallow ground water was measured at depths ranging from approximately 8 to 11 feet below the existing ground surface in our borings. We anticipate ground water may be present at depths as shallow as 6 feet below the existing ground surface, and can be perched in granular layers above ground water levels. 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.

5.1.4 Highly Expansive Soils

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

5.1.5 Undocumented Fill

As mentioned, undocumented fill consisting of clayey sand was encountered in Boring EB-4 to a depth of 2 feet below the surface. While fill was not encountered in our other borings, undocumented fill can be variable in thickness, density, and consistency across the site. We recommend any fill be completely removed from within the building and supplemental structure/equipment areas. Please refer to Section 6.2 below for further recommendations.

5.1.6 Potential for Fill Settlement

As discussed, we understand site grades will be raised to Elevation 25 feet. As a result, it appears site grades will be raised from 0 to about 5 feet above existing grades across the site. This additional fill would cause settlement of the existing soils in addition to settlement due to foundation loads or seismic settlement. We estimate maximum settlement of up to 1 inch due to new fills.

5.1.7 Soil Corrosion Potential

A preliminary soil corrosion screening was performed by JDH Corrosion Consultants based on the results of analytical tests on samples of the near-surface soil. The JDH report concludes that the corrosion potential for buried concrete warrants the use of Type II cement, the water/cement ratio should not exceed 0.45, and there should be minimum depth of 3 inches over reinforcing steel. The JDH report also concludes the corrosion potential for buried metallic structures, such as ductile/cast iron, steel, and dielectric coated steel, is considered corrosive. JDH recommends that special requirements for corrosion control be made to protect metal pipes. A more detailed discussion of the site corrosion evaluation is presented in Appendix C.

5.2 PLANS AND SPECIFICATIONS REVIEW

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

5.3 CONSTRUCTION OBSERVATION AND TESTING

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

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION, CLEARING AND PREPARATION

6.1.1 Site Stripping

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

6.1.2 Tree and Shrub Removal

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

6.1.3 Demolition of Existing Slabs, Foundations and Pavements

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

6.1.4 Abandonment of Existing Utilities

All utilities should be completely removed from within planned building and supplemental structure/equipment pad areas. For any utility line to be considered acceptable to remain within building and supplemental structure/equipment pad 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 and supplemental structure/equipment pad areas unless provided written confirmation from both the owner and the geotechnical engineer.

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

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

6.2 REMOVAL OF EXISTING FILLS

While undocumented fill was only encountered in Boring EB-4, any fills encountered during site grading should be completely removed from within the building areas and supplemental structure/equipment pad areas. Fill should be removed to a lateral distance of at least 5 feet beyond the building footprint and supplemental structure/equipment pad areas or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the "Material for Fill" requirements below, the fills may be reused when backfilling the excavations. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the "Compaction" section below.

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

6.3 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. On a preliminary basis, the upper 5 feet at the site may be classified as OSHA Soil Type C materials. A competent person should determine the actual soil classification during construction and be responsible for implementing and maintaining safe excavation slope inclination and/or shoring at the site during construction.

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 1.5:1 inclination unless the OSHA soil classification indicates differently.

6.4 GROUND WATER

As previously stated, ground water was encountered at approximately 8 to 11 feet below existing grade in our borings. We recommend that contractors anticipate dewatering to control water seeping into deeper excavations close to or below the ground water. Ground water conditions can be difficult to handle, and if the ground water is in a relatively widespread, continuous layer, it may be hard to dewater, requiring continuous dewatering during excavations.

6.5 SUBGRADE PREPARATION

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

6.6 SUBGRADE STABILIZATION MEASURES

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

As discussed in the "Subsurface" section in this report, the in-situ moisture contents range from near optimum to about 8 to 10 percent over the estimated laboratory optimum in the upper 10 feet of the soil profile. The contractor should anticipate needing to dry some of soils prior to reusing them as fill. In addition, repetitive rubber-tire loading may de-stabilize the soils.

There are several methods to address potentially unstable soil conditions and facilitate fill placement and trench backfill. Some of the methods are briefly discussed below.

Implementation of the appropriate stabilization measures should be evaluated on a case-by-case basis according to the project construction goals and the particular site conditions.

6.6.1 Scarification and Drying

The subgrade may be scarified to a depth of 8 to 12 inches and allowed to dry to near optimum conditions, if sufficient dry weather is anticipated to allow sufficient drying. More than one round of scarification may be needed to break up the soil clods.

6.6.2 Removal and Replacement

As an alternative to scarification, the contractor may choose to over-excavate the unstable soils and replace them with dry on-site or import materials. A Cornerstone representative should be present to provide recommendations regarding the appropriate depth of over-excavation, whether a geosynthethic (stabilization fabric or geogrid) is recommended, and what materials are recommended for backfill.

6.6.3 Chemical Treatment

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

6.7 MATERIAL FOR FILL

6.7.1 Re-Use of On-site Soils

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

6.7.2 Re-Use of On-Site Site Improvements

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

If the site area allows for on-site pulverization of PCC and provided the PCC is pulverized to meet the "Material for Fill" requirements of this report, it may be used as select fill within the building areas, excluding the capillary break layer; as typically pulverized PCC comes close to or meets Class 2 AB specifications, the recycled PCC may likely be used within the pavement structural sections. PCC grindings also make good winter construction access roads, similar to a cement-treated base (CTB) section.

6.7.3 Potential Import Sources

Imported and non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less, and not contain recycled asphalt concrete where it will be used within habitable 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.

6.7.4 Non-Expansive Fill Using Lime Treatment

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

6.8 COMPACTION REQUIREMENTS

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

Table 2: Compaction Requirements

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

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

6.8.1 Construction Moisture Conditioning

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

6.9 TRENCH BACKFILL

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

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

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

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

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

6.10 SITE DRAINAGE

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

6.11 LOW-IMPACT DEVELOPMENT (LID) IMPROVEMENTS

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

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

- The near-surface soils at the site are clayey, and categorized as Hydrologic Soil Group D, and is expected to have infiltration rates of less than 0.2 inches per hour. In our opinion, these clayey soils will significantly limit the infiltration of stormwater.
- Locally, seasonal high ground water is mapped at a depth of about 6 feet, and therefore is expected to be within 10 feet of the base of the infiltration measure.
- In our opinion, infiltration locations within 10 feet of the buildings would create a geotechnical hazard.
- Infiltration measures, devices, or facilities may conflict with the location of existing or proposed underground utilities or easements. Infiltration measures, devices, or facilities should not be placed on top of or very near to underground utilities such that they discharge to the utility trench, restrict access, or cause stability concerns.

6.11.1 Storm Water Treatment Design Considerations

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

6.11.1.1 General Bioswale Design Guidelines

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

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

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

6.11.1.2 Bioswale Infiltration Material

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

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

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

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

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

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

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

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

6.11.1.3 Bioswale Construction Adjacent to Pavements

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

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

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

6.12 LANDSCAPE CONSIDERATIONS

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

- Using drip irrigation

- Avoiding open planting within 3 feet of the building perimeter or near the top of 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 data center and associated structures/equipment may be supported on shallow foundations provided the recommendations in the "Earthwork" section and the sections below are followed.

7.2 SEISMIC DESIGN CRITERIA

The 2016 California Building Code (CBC) provides criteria for the seismic design of buildings in Chapter 16. The "Seismic Coefficients" used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Shear wave velocity measurements performed at CPT-5 to a depth of 100 feet resulted in an average shear wave velocity of 777 feet per second (or 237 meters per second). Therefore, we have classified the site as Soil Classification D. The mapped spectral acceleration parameters S_S and S_1 were calculated using the USGS computer program *U.S. Seismic Design Maps*, located at http://earthquake.usgs.gov/designmaps/us/application.php, based on the site coordinates presented below and the site classification. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 3: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.39006°
Site Longitude	-121.96654°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.500g
1-second Period Mapped Spectral Acceleration[1], S_1	0.600g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv	1.5
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.500g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	0.900g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.000g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.600g
Mapped MCE Geometric Mean Peak Ground Acceleration - PGA	0.500g
Site Coefficient Based on PGA and Site Class - F_{PGA}	1.0

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

7.3 SHALLOW FOUNDATIONS – DATA CENTER BUILDING

7.3.1 Spread Footings

Provided the structure can tolerate the anticipated static and seismic total and differential settlements, conventional shallow spread footings can be considered. Spread footings should bear entirely on natural, undisturbed soil or engineered fill, be at least 18 inches wide, and extend at least 24 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil. The deeper footing embedment is due to the presence of highly expansive soils, and is intended to embed the footing below the zone of significant seasonal moisture fluctuation, reducing the potential for differential movement.

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

7.3.2 Spread Footing Settlement

As previously mentioned, you indicated preliminary dead plus live column loads for the data hall and electric rooms with mezzanines are 516 kips and 427 kips, respectively. Based on this loading, the allowable bearing pressures presented above, and assuming site grades will be raised from 0 to about 5 feet, we estimate that the total static footing settlement will be on the order of 1⅓ to 1¾ inches, with about 1-inch of post-construction differential settlement between adjacent foundation elements. In addition we estimate that differential seismic movement will be on the order of ½-inch between independent foundation elements, resulting in a total estimated differential footing movement of about 1½-inch between independent foundation elements. We recommend we be retained to review the final footing layout and loading, and verify the settlement estimates above.

As mentioned, it appears site grades will be raised in locations from 0 up to about 5 feet. We should review the final grading plans to evaluate any impacts varying fill thickness may have on the foundation performance.

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 0.45 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 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.

7.3.5 Alternative Foundation

As an alternative to spread footings or if the estimated settlements exceed the structural requirements, the data center building can also be supported on a reinforced concrete mat foundation as recommended in the sections below. Due to the wide column spacing, a stiff grid foundation or spread footings overlying ground improvement could be additional alternatives to limit settlement.

7.3.6 Reinforced Concrete Mat Foundations

As an alternative to spread footings, the data center building may be supported on a reinforced concrete mat foundation. The mat foundation should bear entirely on undisturbed native soil or engineered fill prepared in accordance with the "Earthwork" section of this report, and designed in accordance with the recommendations below. A non-expansive fill (NEF) section, as discussed in Section 8.1 for interior slabs-on-grade, would not be required beneath a continuous mat foundation for the data center building.

The mat foundation may be designed for a maximum average areal bearing pressure of 1,000 pounds per square foot (psf) for dead plus live loads; at column or wall loading locations the maximum localized bearing pressure should not exceed 3,000 psf. When evaluating wind and seismic conditions, the allowable bearing pressures may be increased by one-third. These pressures are net values; the weight of the mat may be neglected for the portion of the mat extending below grade. Top and bottom mats of reinforcing steel should be included as required to help span irregularities and differential settlement.

7.3.7 Mat Foundation Settlement

For our settlement analysis, we estimated an average areal mat pressure (structural dead plus live load) of 500 psf based on the previously discussed column loading provided. Based on this estimated loading and assuming site grades will be raised from 0 to about 5 feet, we estimate static settlements would be on the order of ⅔ to 1¼ inches at the mat edges and corners and on the order of about 1¼ to 1¾ inches near the center of the mat. Differential settlement from the center of mat to the edges due to static loads is estimated to be up to approximately 1 inch. Accounting for both static and seismic settlement, a mat foundation may experience combined static and seismic differential settlements on the order of 1½ inches between the center of the mat to its edges.

Static settlement estimates were developed based on an estimated average areal mat pressure from the preliminary column loading provided. We recommend we be retained to review the final layout and loading, and verify the settlement estimates above.

7.3.8 Mat Modulus of Soil Subgrade Reaction

We recommend using a variable modulus of subgrade reaction to provide a more accurate soil response and prediction of shears and moments in the mat. This will require at least one iteration between our soil model and the structural SAFE (or similar) analysis for the mat. As discussed above, we estimated an average areal mat pressure of 500 psf within the structure. Based on this pressure, we calculated preliminary modulus of subgrade reaction values for the mat foundation.

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

7.3.9 Mat Lateral Loading

Lateral loads may be resisted by friction between the bottom of mat foundation and the supporting subgrade, and also by passive pressures generated against deepened mat edges. An ultimate frictional resistance of 0.45 applied to the mat dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety to the ultimate values above. The upper 12 inches of soil should be neglected when determining passive pressure capacity.

7.3.10 Mat Foundation Construction Considerations

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

7.3.11 Moisture Protection Considerations for Mat Foundations

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

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

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

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

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

- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869-98 and F710-98 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

7.4 REINFORCED CONCRETE MAT FOUNDATIONS – SUPPLEMENTAL STRUCTURES/EQUIPMENT

The supplemental structures/equipment may be supported on mat foundations bearing on natural, undisturbed soil or engineered fill prepared in accordance with the "Earthwork" section of this report, and designed in accordance with the recommendations below.

For design, we assume mat foundations with a maximum average bearing pressure of 350 pounds per square foot (psf) for dead plus live loads; maximum localized bearing pressure should not exceed 2,000 psf at heavily loaded portions of the mats. When evaluating wind and seismic conditions, the allowable bearing pressures may be increased by one-third. These pressures are net values; the weight of the mat may be neglected for the portion of the mat extending below grade. Top and bottom mats of reinforcing steel should be included as required to help span irregularities and differential settlement.

7.4.1 Mat Foundation Settlement

Based on the above bearing pressure and assuming site grades will be raised 0 to about 5 feet, we estimate static settlements would be on the order of ½ to ⅔ inch near the center of the mat and less than ½ inch at the mat edges and corners. Differential settlement from the center of mat to the edges due to static loads is estimated to be less than ½ inch. Accounting for both static and seismic settlement, a mat foundation may experience combined static and seismic differential settlements on the order of ¾ to 1 inch between the center of the mat to its edges. We recommend we be retained to review the final layout and loading, and verify the settlement estimates above.

7.4.2 Mat Lateral Loading

Lateral loads may be resisted by friction between the bottom of mat foundation and the supporting subgrade, and also by passive pressures generated against deepened mat edges. An ultimate frictional resistance of 0.45 applied to the mat dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety to the ultimate values above. The upper 12 inches of soil should be neglected when determining passive pressure capacity.

7.4.3 Mat Foundation Construction Considerations

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

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 INTERIOR SLABS-ON-GRADE

As the Plasticity Index (PI) of the surficial soils ranges up to 31, proposed slabs-on-grade should be supported on at least 18 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. If a continuous mat foundation is constructed for the data center building, NEF would not be required. The NEF layer should be constructed over subgrade prepared in accordance with the recommendations in the "Earthwork" section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the "Interior Slabs Moisture Protection Considerations" section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and NEF construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned in accordance with the recommendations in the "Compaction" section.

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

8.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

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

- Place a minimum 10-mil-thick vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab. The vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements.

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

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

- Water should not be added after initial batching unless the slump is less than specified and/or the resulting water:cement ratio will not exceed 0.45.
- Polishing the concrete surface with metal trowels is not recommended.
- Where floor coverings are planned, all concrete surfaces should be properly cured.
- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869 and F710 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

8.3 EXTERIOR FLATWORK

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

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 surface conditions.

Table 4: Asphalt Concrete Pavement Recommendations, Design R-value = 5

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

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

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

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

9.2 PORTLAND CEMENT CONCRETE

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

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

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

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

9.3 TRASH ENCLOSURES

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

9.4 PAVEMENT CUTOFF

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

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

SECTION 10: RETAINING WALLS

10.1 STATIC LATERAL EARTH PRESSURES

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

Table 6: Recommended Lateral Earth Pressures

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

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

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

10.2 SEISMIC LATERAL EARTH PRESSURES

The 2016 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. Based on our understanding, only 3- to 4-foot high retaining walls along the east property line are proposed. In our opinion, design of these walls (i.e. walls 6 feet or less in height) 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 with a PI less than 20 should be compacted to at least 95 percent relative compaction using light compaction equipment. If the soil's PI is 20 or greater, expansive soil criteria should be used as discussed in the "Compaction" section of this report. Where no surface improvements are planned, backfill should be compacted to at least 90 percent for soils with a PI less than 20. Expansive soil criteria should be followed for soils with a PI of 20 or greater. If heavy compaction equipment is used, the walls should be temporarily braced.

10.5 FOUNDATIONS

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 Aligned Data Centers specifically to support the design of the 2305 Mission College Boulevard Data Center project in Santa Clara, 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.

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

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

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

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

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

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

SECTION 12: REFERENCES

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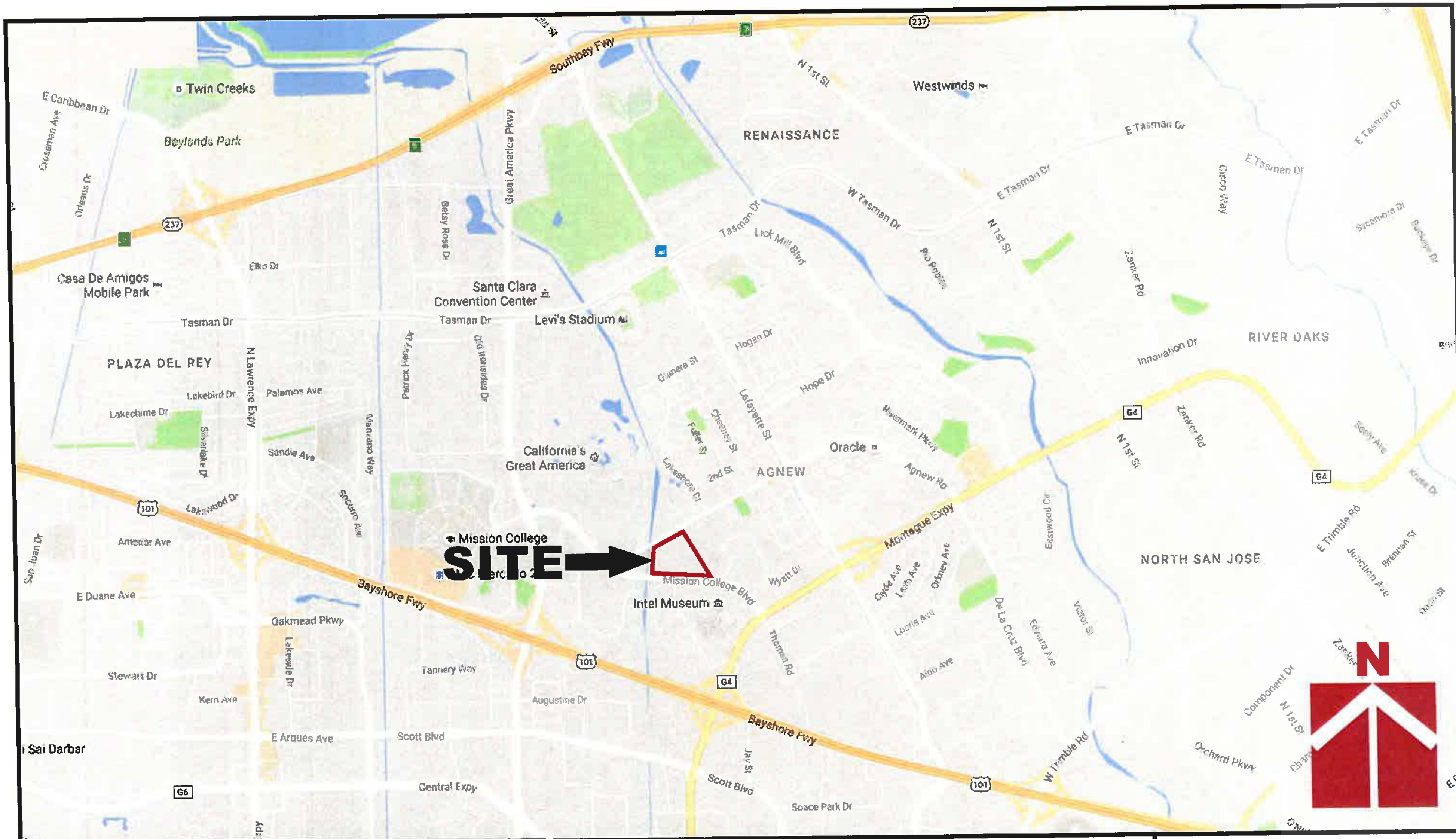

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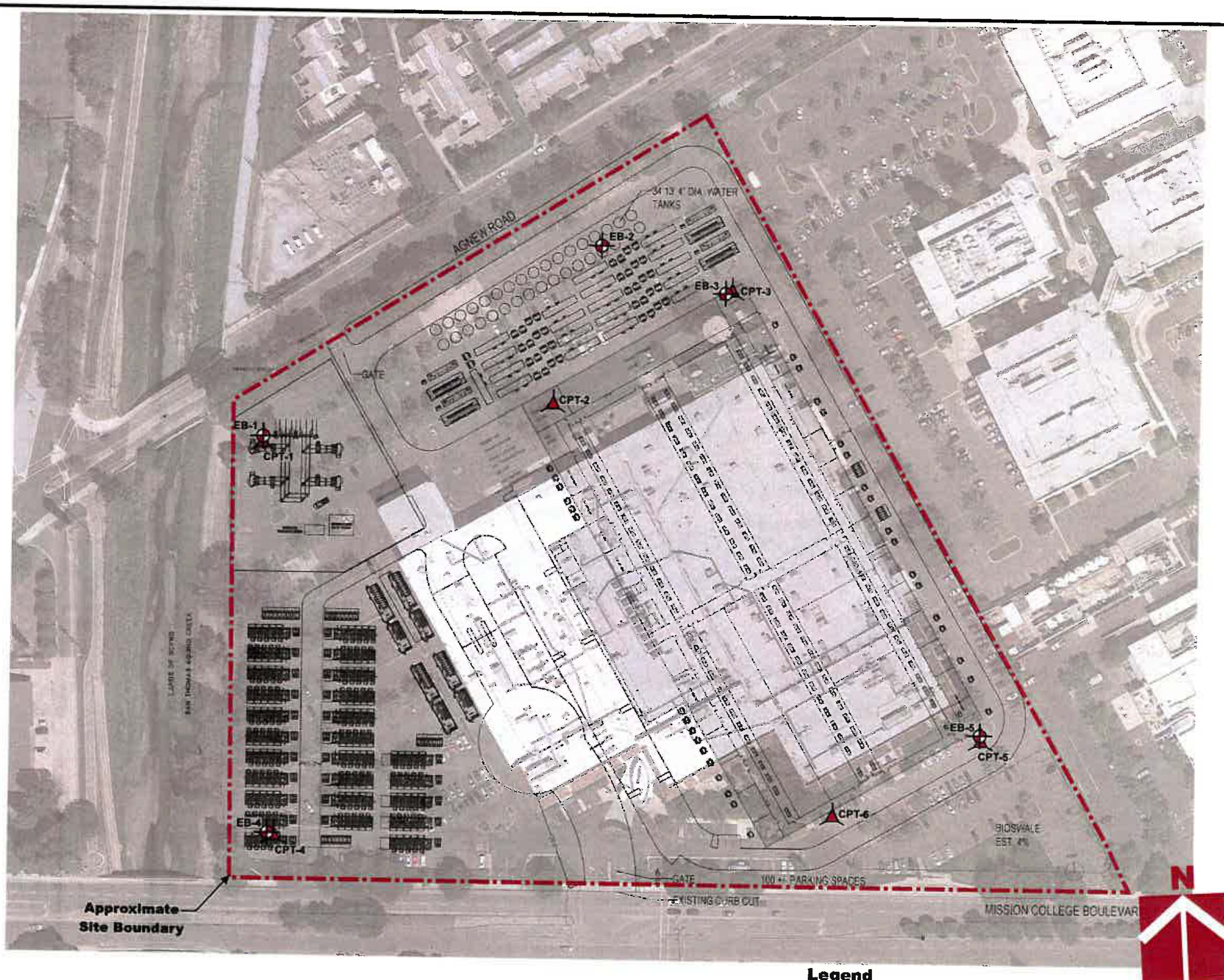
Vicinity Map	Project Number 930-1-1
2305 Mission College Boulevard Data Center Santa Clara, CA	Figure Number Figure 1
	Date November 2016 / Drawn By RRN



Legend

Approximate location of exploratory boring (EB)

Approximate location of cone penetration test (CPT)

0 120 240

APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 4/5/2016
Overlay by CAC Architects, Site Plan - Sheet A, dated 9/21/2016

Site Plan

2305 Mission College Boulevard Data Center
Santa Clara, CA

Project Number: 930-1-1

Figure Number: Figure 2

Date: November 2016

Drawn By: RRN

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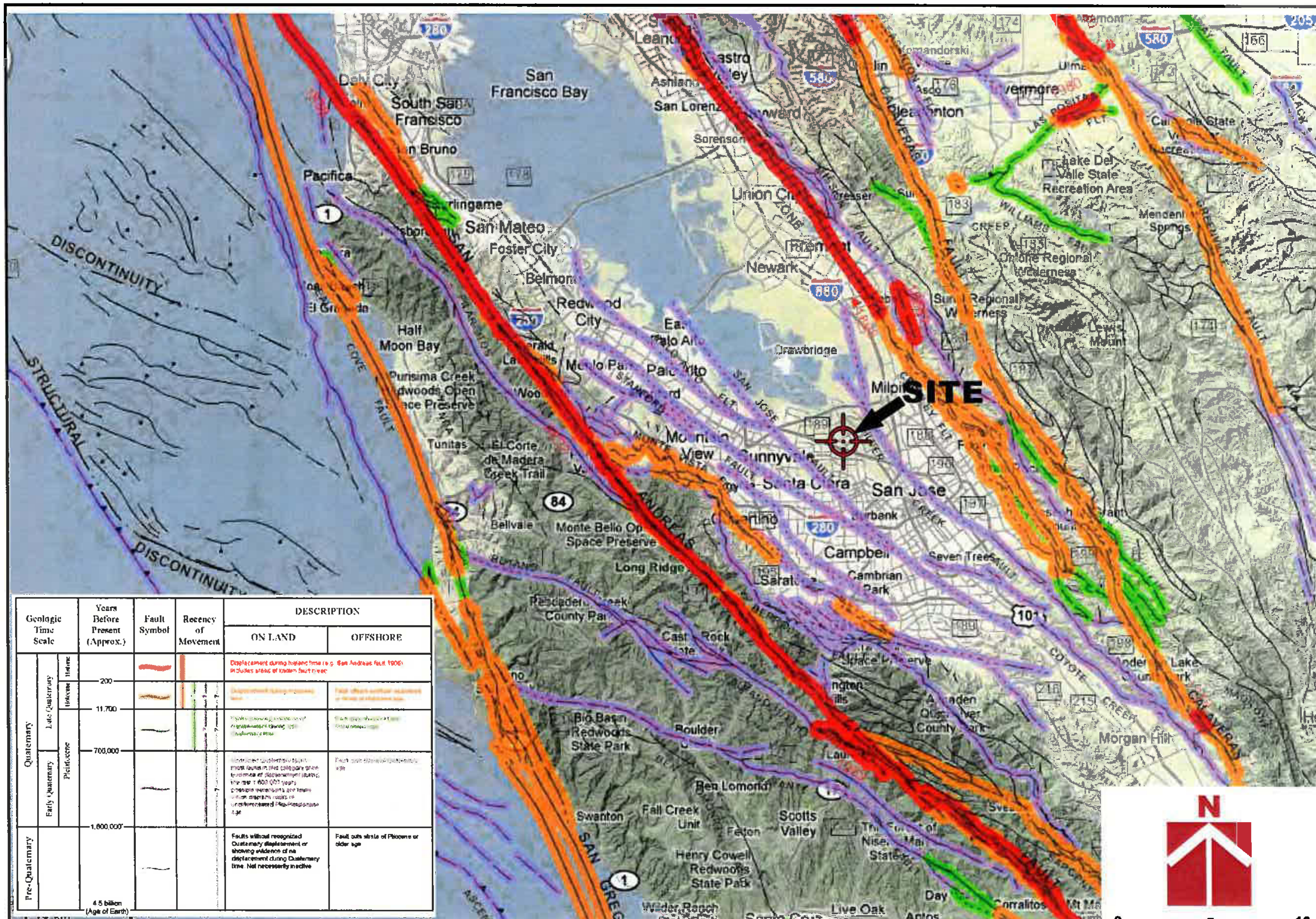

Base by California Geological Survey - 2010 Fault Activity Map of California (Jennings and Bryant, 2010)

N

0 5 10

APPROXIMATE SCALE (MILES)

Regional Fault Map

2305 Mission College Boulevard Data Center
Santa Clara, CA

Project Number	930-1-1
Figure Number	Figure 3
Date	November 2016
Drawn By	RRN

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