APPENDIX D

GEOTECHNICAL INVESTIGATION

APPENDIX B: LABORATORY TEST PROGRAM

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

Moisture Content: The natural water content was determined (ASTM D2216) on 21 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 20 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on one sample of the subsurface soils to aid in the classification of these soils. The result of these test is shown on the boring log at the appropriate sample depth.

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

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

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

CORNERSTONE EARTH GROUP

TYPE OF SERVICES	Geotechnical Investigation
PROJECT NAME	North 27th Street Mixed-Use Development
LOCATION	70-80 North 27th Street San Jose, California
CLIENT	HC Investment Associates, LP
PROJECT NUMBER	1285-1-2
DATE	December 1, 2021

CORNERSTONE EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**North 27th Street Mixed-Use Development**
Location	**70-80 North 27th Street** **San Jose, California**
Client	**HC Investment Associates, LP**
Client Address	**1700 Embarcadero Road** **Palo Alto, California**
Project Number	**1285-1-2**
Date	**December 1, 2021**

Prepared by **Bryan Cervantes Guzman, E.I.T.**
Senior Staff Engineer

Nicholas S. Devlin, P.E.
Principal Engineer
Geotechnical Project Manager

REGISTERED PROFESSIONAL ENGINEER
NICHOLAS S. DEVLIN
No. 75613
Exp. 6-30-2022
CIVIL
STATE OF CALIFORNIA

CORNERSTONE EARTH GROUP

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Type of Services	**Geotechnical Investigation**
Project Name	**North 27th Street Mixed-Use Development**
Location	**70-80 North 27th Street**
	San Jose, California

SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of HC Investment Associates, LP for the North 27th Street Mixed-Use Development in San Jose, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- Preliminary sketches depicting the layout and elevation profile of the proposed structure prepared by Ruggeri Jensen Azar (RJA), undated.

1.1 PROJECT DESCRIPTION

We understand the project will consist of redeveloping the approximately 1.2-acre rectangular site with an at-grade, mixed-use building consisting of 5-stories of residential above a concrete podium commercial and parking floor. A portion of the podium parking level will be below-grade to accommodate a 3-level puzzle lift parking system in the central portion of the proposed building. The podium floor will also include 7,000 square feet for commercial use. The second floor will consist of residential units surrounding an outdoor podium courtyard. The third through sixth floors will consist of residential units. A total of 198 residential units consisting of studio to 1-bedroom apartments are currently planned. Appurtenant parking, utilities, landscaping and other improvements necessary for site development are also planned. We anticipate the podium level of commercial area and parking to be of concrete construction and the residential units above to be of wood construction.

Structural loads are not currently known for the proposed structure; however, structural loads are expected to be typical of similar type structures. Minor cuts and fills on the order of 1 to 3 feet are expected for site development. However, excavations of up to 8 feet are anticipated for the below-grade portions of the 3-level puzzle mechanical lifts.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated September 23, 2021 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 two borings drilled on October 19, 2021 with truck-mounted, hollow-stem auger drilling equipment and four Cone Penetration Tests (CPTs) advanced on October 14, 2021 and October 18, 2021. The borings were drilled to depths of 30 to 50 feet; the CPTs were advanced to depths of 50 to 90 feet in which CPT-1 encountered refusal at a depth of 90 feet. Seismic shear wave velocity measurements were performed at CPT-1. Two of the borings (Borings EB-1 and EB-2) were advanced adjacent to CPT-1 and CPT-4, respectively, for direct evaluation of physical samples to correlated soil behavior. We also performed two percolation tests using hand auger equipment to excavate the test holes to depths of 4 to 5 feet on October 20, 2021.

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

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

1.4 LABORATORY TESTING PROGRAM

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

1.5 ENVIRONMENTAL SERVICES

Cornerstone Earth Group also provided environmental services for this project which included a Phase 1 Site Assessment; environmental findings and conclusions are provided under separate covers.

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 thicknesses in the area are expected to be on the order of 500 feet or greater (Rogers & Williams, 1974).

2.2 REGIONAL SEISMICITY

While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated (in 2015) earlier estimates from their 2014 Uniform California Earthquake Rupture Forecast (Version 3; UCERF3) 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%), 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 Fault.

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

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
Hayward (Southeast Extension)	3.9	6.3
Calaveras	6.6	10.7
Hayward (Total Length)	7.6	12.2
Monte Vista-Shannon	9.0	14.5
San Andreas	13.6	21.9
Sargent	15.2	24.5

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

SECTION 3: SITE CONDITIONS

3.1 SITE BACKGROUND

We reviewed historical aerial imagery provided online by Historical Aerials (http://www.historicaerials.com). A summary of pertinent surface changes at and in the near vicinity of the site is as follows:

- 1948: The site appears to be part of a larger previous parcel spanning along North 27th Street. The site appears developed with warehouse buildings along the western edge and staging area on the eastern section of the site. The railroad tracks are observed east of the site. The church along Santa Clara Street appears built and in its current location. The adjacent residential neighborhood to the west appears mostly built with a few lots undeveloped.
- 1968: The project site appears to remain the same. The adjacent neighborhood appears to have been further developed and largely resembles current conditions.
- 1982: The warehouse buildings appear to have been demolished on the project site. Remainder of the site appears to remain the same.
- 1998: The project site appears to match the current property boundary. The current existing commercial building appears to be built, but the surrounding parking lot and landscape do not appear to be completed yet.
- 2002: The surrounding parking lot and landscaping appear to have been constructed. The adjacent building to the northwest appears to be built
- 2018: The site appears fully constructed with the building, asphalt parking lot, and landscaping in its current condition. No further changes are observed to the site.

3.2 SURFACE DESCRIPTION

The project site is set in a commercial/industrial zone near the intersection of Santa Clara Street and U.S. Highway 101. The area directly west of the site across North 27th Street consist of a residential neighborhood with a few commercial buildings. Currently, the site is developed and occupied by a two-story commercial building, asphalt parking lot, concrete sidewalks, flatwork, utility boxes/pads, and landscaping areas on the edges of the site. Site elevations range from 89 to 91 feet (Google Earth Pro, WGS84). The site is relatively level but graded to drain to storm drain facilities.

Surface pavements generally consisted of 2 to 4 inches of asphalt concrete over 5½ to 8 inches of aggregate base. Based on our observations, the existing pavements are in fair to poor condition with moderate alligator cracking.

3.3 SUBSURFACE CONDITIONS

Our explorations generally encountered existing undocumented fill underlain by interbedded native alluvial soil to the terminal depths explored during this investigation. A more detailed description of the subsurface conditions is presented in the following sections.

3.3.1 Undocumented Fills

Below the surface pavements, our borings encountered approximately 2¼ to 3 feet of undocumented fill; however, deeper localized fill may be encountered during demolition and/or site grading. The fills were highly variable in content and generally consisted of hard, sandy lean clay and medium dense, silty gravel with sand.

3.3.2 Alluvial Soil

Below the undocumented fills, our borings encountered native alluvial soil consisting of medium stiff to very stiff, lean clay and lean clay with sand to a depth of approximately 50 feet below existing site grades. Thin interbedded layers of loose, silty sand were encountered from approximately 8 to 10 feet within our Exploratory Boring EB-2 and 14 to 14½ feet within Boring EB-1. Below the terminal boring depth of 50 feet, our CPTs generally encountered additional fine-grained material consisting of stiff to very stiff, lean clay with varying amounts of silt and sand to depths of approximately 63½ to 64½ feet. Below the fine-grained materials, dense to very dense, sand with variable amounts of silt and clay fines were encountered to a depth of approximately 90 feet, the depth of refusal.

3.3.1 Plasticity/Expansion Potential

We performed two Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of surficial soils. The results of the surficial PI tests indicated PIs ranging from 11 to 16, indicating low to moderate expansion potential to wetting and drying cycles.

3.3.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from 5 to 31 percent moisture. In our opinion, we estimated this corresponds to about 2 percent below the estimate laboratory optimum moisture content of the sandy gravel fill encountered within EB-1. For the underlying clayey soil, we estimate this corresponds to about 3 to 18 percent above the estimated laboratory optimum moisture content.

3.4 GROUNDWATER

Groundwater was encountered in our Boring EB-1 at a depth of 13 feet below current site grades. Groundwater was inferred at our CPTs (CPT-1 to CPT-4) at depths of approximately 10 to 16 feet below current grades. The depths to groundwater inferred from the CPTs are based on pore pressure dissipation tests being performed at each CPT location. All measurements were taken at the time of drilling and may not represent the stabilized levels that can be higher than the initial levels encountered.

Maps published by the California Geological Survey (CGS, 2000) indicate historical high groundwater depth between 5 to 10 feet below the ground surface. We also reviewed groundwater data available online from the website GeoTracker, https://geotracker.waterboards.ca.gov/. Nearby monitoring well data indicates that groundwater has been measured at depths of approximately 9 to 11 feet at wells located approximately 600 feet east of the project site between 2007 to 2014.

Based on the above, we recommend a design groundwater depth of 8 feet. Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

3.5 IN-SITU WATER INFILTRATION

The design of subsurface drainage systems including detention basins and bio-swales typically requires adequate subsurface data and percolation characteristics of the underlying soil. Percolation characteristics of the underlying soil were requested in future potential retention basin or bioswales. To estimate the infiltration rate of the soils at locations and depths typically associated with these subsurface drainage systems, we performed two in-situ field infiltration tests using a Guelph permeameter by SoilMoisture Equipment Corp., Model #2800, in general accordance with ASTM D5126. Generally, the Guelph permeameter is a constant head device, which uses two water-filled chambers to measure infiltration rate in a shallow borehole. A constant head level is established in the borehole and the rate of water outflow into the surrounding soil is noted. The rate of flow when it reaches a steady state, or constant rate, is used to determine an approximate infiltration rate for that location and depth.

The approximate location of the field infiltration tests (P-1 and P-2) are shown on the Site Plan, Figure 2. The infiltration tests were performed at approximate depths of 4 and 5 feet below existing site grades, respectively. The test results are summarized in Table 2.

Table 2: In-Situ Field Guelph Permeameter Test Results

Location	Depth Below Existing Grade (ft)	Infiltration Rate (in/hr)
P-1	4.0	1.2
P-2	5.0	1.2

3.5.1 Reliability of Field Test Data

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

3.5.2 Findings and Recommendations

Based on our findings, the soil at the locations tested and at depths of about 4 and 5 feet below existing grade have an infiltration rate of about 1.2 inches per hour. Based on our test results, the in-situ field tests generally indicated a moderate infiltration rate at the depths and locations tested.

We recommend the above estimate be confirmed in the field at the time of construction, as required. In addition, the project civil engineer should review the above information and provide additional recommendations as deemed necessary.

3.5.3 General Comments and Design Considerations

As discussed, the tests were performed at discrete locations and depths. In addition, some disturbance in preparing the test also can occur. Therefore, the above results can vary significantly and may not be representative over the entire site. Localized areas/depths with higher or lower permeable materials can increase or decrease the actual infiltration rates. Therefore, we recommend the potential for variations be considered when evaluating the soil infiltration capacity or performance.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT SURFACE 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 surface 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 following the ground motion hazard analysis procedure presented in Chapter 16 and 18 and Appendix J of the 2019 California Building Code (CBC) and Chapter 21, Section 21.2 of ASCE 7-16 and Supplement No. 1. For our analysis we used a PGA_M of 0.70g which was determined in accordance with Section 21.5 of ASCE 7-16.

4.3 LIQUEFACTION POTENTIAL

The site is within a State-designated Liquefaction Hazard Zone (CGS, San Jose East Quadrangle, 2001) as well as a Santa Clara County Liquefaction Hazard Zone (Santa Clara County, 2012). 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 groundwater depth of 8 feet. Following the liquefaction analysis framework in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008), incorporating updates in *CPT and SPT Based Liquefaction Triggering Procedures* (Boulanger and Idriss, 2014), and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008) for quantitative analysis, these layers were analyzed for liquefaction triggering and potential post-liquefaction settlement. These methods compare the ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil's estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.3 are considered to be potentially liquefiable and capable of post-liquefaction re-consolidation (i.e. settlement).

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

The soil's CRR is estimated from the in-situ measurements from CPTs and laboratory testing on samples retrieved from our borings. SPT "N" values obtained from hollow-stem auger borings were not used in our analyses, as the "N" values obtained are less reliable in sands below groundwater. The tip pressures are corrected for effective overburden stresses, taking into consideration both the groundwater level at the time of exploration and the design groundwater level, and stress reduction versus depth factors. The CPT method utilizes the soil behavior type index (I_C) to estimate the plasticity of the layers. Selected soil samples collected from advancing borings EB-1 and EB-2 adjacent to CPT-1 and CPT-4, respectively, were tested to evaluate grain size, as well as visually observed for confirmation of CPT soil behavior types.

The results of our CPT analyses (CPT-1 to CPT-4) are presented on Figures 4A to 4D of this report. Calculations for these CPTs are included in Appendix C.

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 ¼- to ½-inch based on the Yoshimine (2006) method. As discussed in SP 117A, differential movement for level ground sites over deep soil sites will be up to about two-thirds of the total settlement between independent foundation elements. In our opinion, differential settlements are anticipated to be on the order of ¼ inch or less between independent foundation elements or over a horizontal distance of 30 feet.

4.3.4 Ground Deformation and Surficial Cracking Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground deformation or sand boils. For ground deformation 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 8-foot-thick layer of non-liquefiable cap is sufficient to prevent ground deformation and significant surficial cracking; therefore, the above total settlement estimates are reasonable.

4.4 LATERAL SPREADING

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

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

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. As the soils encountered at the site were predominantly stiff to very stiff clays and medium dense sands, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low.

4.6 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone AH and AO, special flood hazard zones with a base flood elevation of 89 feet and a depth of flooding of 1 foot. We understand the project civil engineer confirmed this information.

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 Static and Seismic Settlement
- Mitigation of Undocumented Fill and Redevelopment Considerations
- Shallow Groundwater – Puzzle Lifts

5.1.1 Potential for Static and Seismic Settlement

5.1.2.1 Static Settlement

We understand the finished floor of the proposed building will approximately be at the existing site grade and that new fill is not anticipated at this time. As such, static settlement due to fill placement was not considered in our static settlement analysis discussed below. In addition, structural loads were not available; therefore, we estimated foundation loading (dead plus live loads) of 600 to 650 kips for interior columns beneath the residential floors, 250 kips for interior columns beneath the courtyard/deepened podium level and 18 to 22 kips per lineal foot for exterior walls. Our static settlement estimates are based on an at-grade building being supported on conventional shallow spread and strip footings with deepened footings under the mechanical puzzle lifts.

For shallow footings for the building anticipated to be about 2 to 4 feet below existing site grades, we estimate static settlement to be on the order of ¾ to 1 inch with differential settlement on the order of ⅓ to ½ inch. Based on our review of the provided plans, we anticipate deepened footings for the puzzle lift to be on the order of 8 to 10 feet below existing site grades. For deepened footings for the puzzle lifts, we estimate static settlement on the order of 1⅓ to 2 inches with differential settlement on the order of ⅔ to 1 inch. This range of settlement is preliminary and will depend on the final building loads and finished floor elevations for the building. Therefore, we recommend that we be retained to re-evaluate the static settlement estimates after final structural loads are known.

5.1.2.2 Seismic Settlement

As discussed above, 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 sand to vent to the ground surface through cracks in the surficial soil is low, our analysis indicates that liquefaction-induced settlement of ¼ to ½ inch could occur, resulting in differential settlement of up to ⅓ inch between adjacent independent foundation elements or over a horizontal distance of 30 feet.

5.1.2.3 Total (Static and Seismic) Settlement

Based on the estimated settlement discussed above, the total (static and seismic) settlement for shallow footings could be on the order of 1 to 1½ inches with differential settlement of about ½ to ¾ inch between independent adjacent foundation elements, or over a horizontal distance of about 30 feet. In addition, the total (static and seismic) settlement for the deeper puzzle lift footings could be on the order of 1½ to 2½ inches with differential settlement of about ¾ inch to 1⅓ inches between independent adjacent foundation elements, or over a horizontal distance of about 30 feet.

5.1.2.4 Foundation Recommendations

Based on our assumed structural loads and results of our settlement analysis, we anticipate the proposed building may be supported on shallow foundations consisting of conventional spread footings provided the estimated total and differential settlement is tolerable. If the settlement estimated above is not tolerable for spread footings, the proposed building may be supported on a rigid mat foundation designed to tolerate the total and differential settlement discussed above. Detailed recommendations are presented in the "Foundations" section.

5.1.2 Mitigation of Undocumented Fill and Redevelopment Considerations

As discussed above, approximately 2¼ to 3 feet of undocumented fill was encountered below current site grades in our borings during our field exploration. Based on our explorations and due to the previous development and site history, we anticipate fills are generally present across the site. Undocumented fills are expected to be variable in thickness, density, and consistency. Additionally, deeper localized fill may be encountered during demolition and/or site grading. We recommend all undocumented fill at the site be completely removed from within the building areas and replaced as engineered fill. Additional recommendations are provided in the "Earthwork" section.

5.1.3 Shallow Groundwater – Puzzle Lifts

Shallow groundwater was encountered within our borings and inferred at our CPTs at depths ranging from approximately 10 to 16 feet below the existing ground surface. As discussed above, we recommend a design groundwater depth of 8 feet. Anticipated grading for the proposed building pad and foundations are expected to be on the order of 2 to 3 feet to mitigate existing undocumented fill and approximately 8 to 10 feet for the puzzle lift parking system. Our experience with similar sites in the vicinity indicates that shallow groundwater could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering and shoring of excavations and utility trenches may be required in some isolated areas (e.g. puzzle lifts) of the site. Detailed recommendations addressing this concern are presented in the "Earthwork" section of this report.

5.2 PLANS AND SPECIFICATIONS REVIEW

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

5.3 CONSTRUCTION OBSERVATION AND TESTING

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

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION

All existing improvements not to be reused for the current development, including all foundations, flatwork, pavements, utilities, and other improvements should be demolished and removed from the site. Recommendations in this section apply to the removal of these improvements, which are currently present on the site, prior to the start of mass grading or the construction of new improvements for the project.

Cornerstone should be notified prior to the start of demolition and should be present on at least a part-time basis during all backfill and mass grading as a result of demolition. Occasionally, other types of buried structures (wells, cisterns, debris pits, etc.) can be found on sites with prior development. If encountered, Cornerstone should be contacted to address these types of structures on a case-by-case basis.

6.1.1 Demolition of Existing Slabs, Foundations and Pavements

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

Special care should be taken during the demolition and removal of existing floor slabs, foundations, utilities and pavements to minimize disturbance of the subgrade. Excessive disturbance of the subgrade, which includes either native or previously placed engineered fill, resulting from demolition activities can have serious detrimental effects on planned foundation and paving elements.

Existing foundations are typically mat-slabs, shallow footings, or piers/piles. If slab or shallow footings are encountered, they should be completely removed. If drilled piers are encountered, they should be cut off at an elevation at least 60-inches below proposed footings or the final subgrade elevation, whichever is deeper. The remainder of the drilled pier could remain in place. Foundation elements to remain in place should be surveyed and superimposed on the proposed development plans to determine the potential for conflicts or detrimental impacts to the planned construction. Following review, additional mitigation or planned foundation elements may need to be modified.

6.1.2 Abandonment of Existing Utilities

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

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

The risk for owners 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.

6.2 SITE CLEARING AND PREPARATION

6.2.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements to be removed within the proposed development area. Demolition of existing improvements is discussed in the prior paragraphs. A detailed discussion of removal of existing fills is provided later in this report. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Based on our site observations, surficial stripping should extend about 3 to 4 inches below existing grade in vegetated areas.

6.2.2 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.

6.3 MITIGATION OF UNDOCUMENTED FILL

As discussed above, approximately 2¼ to 3 feet of undocumented fill was encountered; however, deeper localized fills may be encountered during demolition and/or site grading and should be anticipated and planned for by the contractor. We anticipate that existing fill will be removed within the areas of the deeper excavations (e.g. puzzle lifts); however, all undocumented fills should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations. Based on review of the samples collected from our borings, it appears that the fill may be reused. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should be 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.

6.4 TEMPORARY CUT AND FILL SLOPES

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

Excavations performed during site demolition and fill removal should be sloped at 3:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Actual excavation inclinations should be reviewed in the field during construction, as needed. Excavations below building subgrade and excavations in pavement and flatwork areas should be sloped in accordance with OSHA soil classification requirements.

6.5 BELOW-GRADE EXCAVATIONS

The below-grade excavations for the proposed puzzle lifts may be constructed with temporary slopes in accordance with the “Temporary Cut and Fill Slopes” section above if space allows. Alternatively, temporary shoring may support the planned cuts of up to 8 to 10 feet. We have provided geotechnical parameters for shoring design in the section below. The choice of shoring method should be left to the contractor’s judgment based on experience, economic considerations and adjacent improvements such as utilities, pavements, and foundation loads. Temporary shoring should support adjacent improvements without distress and should be the contractor’s responsibility. A pre-condition survey including photographs and installation of monitoring points for existing site improvements should be included in the contractor’s scope. We should be provided the opportunity to review the geotechnical parameters of the shoring design prior to implementation; the project structural engineer should be consulted regarding support of adjacent structures.

6.5.1 Temporary Shoring – Puzzle Lifts

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

Table 3: Suggested Temporary Shoring Design Parameters

Design Parameter	Design Value
Minimum Lateral Wall Surcharge (upper 5 feet)	120 psf
Cantilever Wall – Triangular Earth Pressure	40 pcf
Restrained Wall – Trapezoidal Earth Pressure	Increase from 0 to 25H* psf
Passive Pressure – Starting at 2 feet below the bottom of the excavation	400 pcf up to 2,000 psf maximum uniform pressure

* H equals the height of the excavation; passive pressures are assumed to act over twice the soldier pile diameter

The restrained earth pressure may also be distributed as described in Section 5.2.4 Figure 23(b) of the *FHWA Circular No. 4 – Ground Anchors and Anchored Systems* (with the hinge points at ¼H and ¾H) provided the total pressure is established from the uniform pressure above.

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

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

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

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

6.6 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.

6.7 WET SOIL STABILIZATION GUIDELINES

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

As discussed in the “Subsurface” section in this report, the in-situ moisture contents are about 3 to 18 percent over the estimated laboratory optimum in the upper 10 feet of the soil profile. The contractor should anticipate drying the soils prior to reusing them as fill. In addition, repetitive rubber-tire loading will likely de-stabilize the soil.

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

6.7.1 Scarification and Drying

The subgrade may be scarified to a depth of 6 to 12 inches and allowed to dry to near optimum conditions, if sufficient dry weather is anticipated to allow sufficient drying. More than one round of scarification may be needed to break up the soil clods.

6.7.2 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 geosynthetic (stabilization fabric or geogrid) is recommended, and what materials are recommended for backfill.

6.7.3 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.

6.8 MATERIAL FOR FILL

6.8.1 Re-Use of On-site Soils

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

6.8.2 Re-Use of On-Site Site Improvements

If asphalt concrete (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 building 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 proposed 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. Laboratory testing will be required to confirm the material meets project specifications. PCC grindings also make good winter construction access roads, similar to a cement-treated base (CTB) section.

6.8.3 Potential Import Sources

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

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

6.9 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 "Wet Soil Stabilization Guidelines" section of this report. Where the soil's PI is 20 or greater, the expansive soil criteria should be used.

Table 4: Compaction Requirements

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

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

6.9.1 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.

6.10 TRENCH BACKFILL

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

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

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

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

6.11 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.

6.12 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.

- In general, from our percolation testing discussed above, the near surface clayey site soils would likely be categorized as Hydrological Soil Group C for USDA soil classification, which typically has a possible range of infiltration of approximately 0.2 up to 1.2 inches per hour. In our opinion, these clayey soils will limit the infiltration of stormwater.

- Locally, design high groundwater is designated at a depth of 8 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.

6.12.1 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.

6.12.1.1 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.

6.12.1.2 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 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.

- 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.

6.12.1.3 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.

6.13 LANDSCAPE CONSIDERATIONS

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

- Using drip irrigation

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

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

SECTION 7: 2019 CBC SEISMIC DESIGN CRITERIA

7.1 SEISMIC DESIGN CRITERIA

We developed site-specific seismic design parameters in accordance with Chapter 16, Chapter 18 and Appendix J of the 2019 California Building Code (CBC) and Chapters 11, 12, 20, and 21 and Supplement No. 1 of ASCE 7-16.

7.1.1 Site Location and Provided Data For 2019 CBC Seismic Design

The project is located at latitude 37.349730° and longitude -121.865795°, which is based on Google Earth (WGS84) coordinates at the approximate center of site in San Jose, California. We have assumed that a Seismic Importance Factor (I_e) of 1.00 has been assigned to the structure in accordance with Table 1.5-2 of ASCE 7-16 for structures classified as Risk Category II. The building period has not been provided by the project structural engineer.

7.2 2019 CBC SEISMIC DESIGN CRITERIA

As discussed in the “Subsurface” of our report, our CPT and exploratory borings encountered alluvial soils consisting of medium dense to dense sands and medium stiff to very stiff clay deposits to a depth of 90 feet, the maximum depth explored. Shear wave velocity (V_S) measurements were performed while advancing CPT-1, resulting in a time-averaged shear wave velocity for the top 30 meters (V_{S30}) of 246 meters per second (807 feet per second), for the upper 100 feet.

7.2.1 2019 CBC Seismic Design

As our borings encountered deep alluvial soils with shear wave velocity for the upper 30 meters between 600 and 1200 feet per second, per section 20.3.2 of ASCE 7-16, we have classified the site as Soil Classification D, which is described as a “stiff soil” profile. Because we used site specific data from our explorations and laboratory testing, the site class should be considered as “determined” for the purposes of estimating the seismic design parameters from the code. Our site-specific ground motion hazard analysis considered a V_{S30} of 246 m/s (807 ft/s).

In accordance with Section 11.4.8 of ASCE 7-16, we performed a ground motion hazard analysis following Chapter 21, Section 21.2 of ASCE 7-16. We evaluated both Probabilistic MCE_R Ground Motions in accordance with Method 1 and Deterministic MCE_R Ground Motions to generate our recommended design response spectrum for the project, see Figure 6. The recommended design spectral accelerations and associated periods are provided graphically on Figure 5.

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

As discussed in the "Conclusions" section, the building may be supported on shallow foundations consisting of conventional spread footings provided the recommendations in the "Earthwork" section and below are followed. As an alternative, the building may be supported on a rigid mat foundation. Foundation recommendations are presented in the following sections.

8.2 SHALLOW FOUNDATIONS

8.2.1 Conventional Spread Footings

Conventional shallow footings should bear on natural, undisturbed soil or engineered fill, be at least 15 inches wide, and extend at least 24 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil.

Footings constructed to the above dimensions and in accordance with the "Earthwork" recommendations of this report are capable of supporting maximum allowable bearing pressures of 1,500 psf for dead loads, 2,250 psf for combined dead plus live loads, and 3,000 psf for all loads including wind and seismic. These pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected for the portion of the footing extending below grade (typically, the full footing depth). Top and bottom mats of reinforcing steel should be included in continuous footings to help span irregularities and differential settlement.

8.2.2 Footing Settlement

Structural loads were not known at the time this report was prepared; therefore, we assumed the following typical loading. We estimated foundation loading (dead plus live loads) of 600 to 650 kips for interior columns beneath the residential floors, 250 kips for interior columns beneath the courtyard/deepened podium level and 18 to 22 kips per lineal foot for exterior walls.

8.2.2.1 Shallow Footings

Based on the above loading and the allowable bearing pressures presented above, we estimate the total static footing settlement will be on the order of ¾ to 1 inch, with about ⅓ to ½ inch of post-construction differential settlement between adjacent foundation elements. In addition, we estimate that differential seismic movement will be on the order of ¼ to ⅓ inch resulting in a total estimated differential footing movement of ½ to ¾ inch between foundation elements, assumed to be on the order of 30 feet.

8.2.2.2 Deep Footings – Puzzle Lifts

Based on the above loading and the allowable bearing pressures presented above, we estimate the total static footing settlement will be on the order of 1⅓ to 2 inches, with about ⅔ to 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 ¼ to ⅓ inch resulting in a total estimated differential footing movement of ¾ inch to 1⅓ inches between foundation elements, assumed to be on the order of 30 feet.

As our footing loads were assumed, we recommend we be retained to review the final footing layout and loading and verify the settlement estimates above.

8.2.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of footing and the supporting subgrade, and also by passive pressures generated against footing sidewalls. An ultimate frictional resistance of 0.40 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 400 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

8.2.4 Conventional Shallow Footing Construction Considerations

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

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

8.3.1 Reinforced Concrete Mat Foundations

As an alternative to conventional spread footings, the proposed structure may be supported on a mat foundation bearing on natural soil or engineered fill prepared in accordance with the "Earthwork" section of this report and designed in accordance with the recommendations below. Reinforced concrete mat foundations should be designed in accordance with the 2019 California Building Code.

To reduce potential differential movement, the mat should be designed for a maximum *average* allowable bearing pressure of 750 psf for dead plus live loads; at column or wall loading, the maximum localized bearing pressure should be limited to 1,500 psf. When evaluating wind and seismic conditions, allowable bearing pressures may be increased by one-third. These pressures are net values; the weight of the mat may be neglected for the portion of the mat extending below grade. Top and bottom mats of reinforcing steel should be included as required to help span irregularities and differential settlement. If the actual average areal bearing pressure is higher than presented above, or if there are other aspects of design not accounted for in this report, please notify us so that we may revise our recommendations.

8.3.2 Mat Foundation Settlement

Based on the assumed areal pressures above, we estimate a static settlement of ¾ to 1 inch with a differential static settlement of up to ½ inch between the center and edges of the mat. The static settlement estimates presented below are based on assumed loads of 125 psf per floor for the residential levels above the podium level and 150 psf per floor for the podium concrete level. Additionally, as discussed earlier, seismic settlements of up to ½ inch with a differential settlement of ¼ inch or less are anticipated across a horizontal distance of 30 feet. The total (static and seismic) settlement is estimated to be 1¼ to 1½ inches with an anticipated differential settlement of up to ¾ inch. As our structural loads are assumed, we recommend we be retained to review the final foundation plan and loading and to verify the settlement estimates above.

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

8.3.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of mat foundation and the supporting subgrade, and also by passive pressures generated against deepened mat edges. An ultimate frictional resistance of 0.40 applied to the mat dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 400 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. The upper 12 inches of soil should be neglected when determining passive pressure capacity.

8.3.4 Mat Modulus of Soil Subgrade Reaction

The modulus of soil subgrade reaction is a model element that represents the response to a specific loading condition, including the magnitude, rate, and shape of loading, given the subsurface conditions at that location. Based on the assumed average areal loading indicated above, we developed preliminary soil subgrade moduli for initial structural design. These values are preliminary and should be confirmed/finalized following initial analysis by the project structural engineer. Once contact pressures are available from the initial analysis (SAFE or equivalent), we should revise our model and provide contours of equal soil subgrade modulus values for design. Please forward contact pressures to scale and in color for our analysis.

For preliminary SAFE runs (or equivalent analysis), we recommend an initial modulus of soil subgrade reaction of 5 pounds per cubic inch (pci) for the below grade mat foundation. As discussed above, the modulus of soil subgrade reaction is intended for use in the first iteration of the structural SAFE analysis for the mat design. Once the initial structural analysis is complete, please forward a color plot of contact pressures for the mat (to scale) so that we can provide a revised plan with updated contours of equal modulus of soil subgrade reaction values.

8.3.5 Mat Foundation Construction Considerations

Due to the presence of moderately 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 2 percent over optimum in the upper 12 inches.

8.3.6 Hydrostatic Uplift and Waterproofing – Puzzle Lift

As discussed, we recommend a design high groundwater depth of 8 feet below existing grades at the site. In addition, we anticipate the deepened excavations for the puzzle lifts will be 8 to 10 feet below the existing grades. Therefore, where portions of the structure extend below the design groundwater level, including bottoms of slabs-on-grade and mat foundations, they should be designed to resist potential hydrostatic uplift pressures.

In addition, the portions of the structures extending below design groundwater should be waterproofed to limit moisture infiltration, including mat foundation/thickened slab areas, all construction joints, and any retaining walls. We recommend that a waterproof specialist design the waterproofing system.

8.4 GROUND IMPROVEMENT AND DEEP FOUNDATIONS

As alternatives to shallow spread footings and mat foundations, the building may also be supported on spread footings over ground improvement or a deep foundation system, such as augercast or driven piles. If these options are desired, we can provide additional recommendations upon request.

SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

9.1 INTERIOR SLABS-ON-GRADE

Due to the expansion potential of the surficial soils, the proposed slabs-on-grade should be at least 5 inches thick and be supported on at least 6 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. The NEF layer should be constructed over subgrade prepared in accordance with the recommendations in the "Earthwork" section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the "Interior Slabs Moisture Protection Considerations" section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and slab-on-grade 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 to at least 3 percent over the optimum moisture content.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying soils. For unreinforced concrete slabs, ACI 302.1R recommends limiting control joint spacing to 24 to 36 times the slab thickness in each direction, or a maximum of 18 feet.

9.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

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

- Place a minimum 15-mil vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab; the vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements. A 4-inch-thick capillary break, consisting of crushed rock should be placed below the vapor retarder and consolidated in place with vibratory equipment. The mineral aggregate shall be of such size that the percentage composition by dry weight as determined by laboratory sieves will conform to the following gradation:

Sieve Size	Percentage Passing Sieve
1”	100
¾”	90 – 100
No. 4	0 – 10
No. 200	0 – 5

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 F1869-98 and F710-98 requirements and evaluated against the floor covering manufacturer’s requirements prior to installation.

9.3 EXTERIOR FLATWORK

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

SECTION 10: VEHICULAR PAVEMENTS

10.1 ASPHALT CONCRETE

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

Table 5: 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	13.5	17.5

*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 use the pavements.

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

10.2 PORTLAND CEMENT CONCRETE

The Portland Cement Concrete (PCC) pavement recommendations outlined below are based on methods presented in American Concrete Pavement Association (ACPA, 2006). We have provided a few pavement alternatives as an anticipated Average Daily Truck Traffic (ADTT) was not provided. Recommendations for garage slabs-on-grade were provided in the "Concrete Slabs and Pedestrian Pavements" section above.

Table 6: PCC Pavement Recommendations, Design R-value = 5

Traffic Category	Minimum PCC Thickness (inches)	Class 2 Aggregate Base* (inches)
Maximum ADTT = 10	6.5	6.0
Maximum ADTT = 20	7.0	6.0

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

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi. Adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Due to the expansive surficial soils present, we recommend that the construction and expansion joints be dowelled.

10.2.2 Stress Pads for Trash Enclosures

Pads where trash containers will be stored, and where garbage trucks will park while emptying trash containers, should be constructed on Portland Cement Concrete. We recommend that the trash enclosure pads and stress (landing) pads where garbage trucks will store, pick up, and empty trash be increased to a minimum PCC thickness of 7 inches. The compressive strength, underlayment, and construction details should be consistent with the above recommendations for PCC pavements.

SECTION 11: RETAINING WALLS

11.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall. Provided a drainage system is constructed behind the wall to prevent the build-up of hydrostatic pressures as discussed in the section below, we recommend that the walls with level backfill be designed for the pressures in the table below. Due to the presence of expansive native soils, cantilever retaining walls backfilled with the native clay soil should be designed as restrained. If granular backfill materials are used, then the unrestrained values in the table can be used.

Table 7: Recommended Lateral Earth Pressures

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

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

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

11.2 SEISMIC LATERAL EARTH PRESSURES

The 2019 California Building Code (CBC) states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. As we anticipate the walls for the puzzle lifts will be at least 8 feet tall, we checked seismic earth pressures for the anticipated proposed restrained walls in accordance with CBC 1803.5.12 and ASCE 7-16 Section 11.8.3 using the Design level earthquake. We developed seismic earth pressures for the proposed puzzle lift walls using interim recommendations generally based on refinement of the Mononobe-Okabe method (Lew et al., SEAOC 2010).

We anticipate the puzzle lift basement retaining wall will be approximately 8 feet tall. The peak ground accelerations at the site are greater than 0.40g so we checked the result of the total seismic increment when added to the recommended active earth pressure against the recommended fixed (restrained) wall earth pressures. Because the wall is restrained, or will act as a restrained wall, and will be designed for 45 pcf (equivalent fluid pressure) plus a uniform earth pressure of 8H psf, based on current recommendations for seismic earth pressures, it appears that active earth pressures plus a seismic increment do not exceed the fixed wall earth pressures. Therefore, an additional seismic increment above the design earth pressures is not required as long as the walls are designed for the restrained wall earth pressures recommended above in accordance with the CBC.

11.3 WALL DRAINAGE

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

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

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

11.4 BACKFILL

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.

11.5 FOUNDATIONS

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

SECTION 12: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of HC Investment Associates, LP specifically to support the design of the North 27th Street Mixed-Use Development project in San Jose, 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 groundwater 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.

HC Investment Associates, LP may have provided Cornerstone with plans, reports and other documents prepared by others. HC Investment Associates, LP 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 13: REFERENCES

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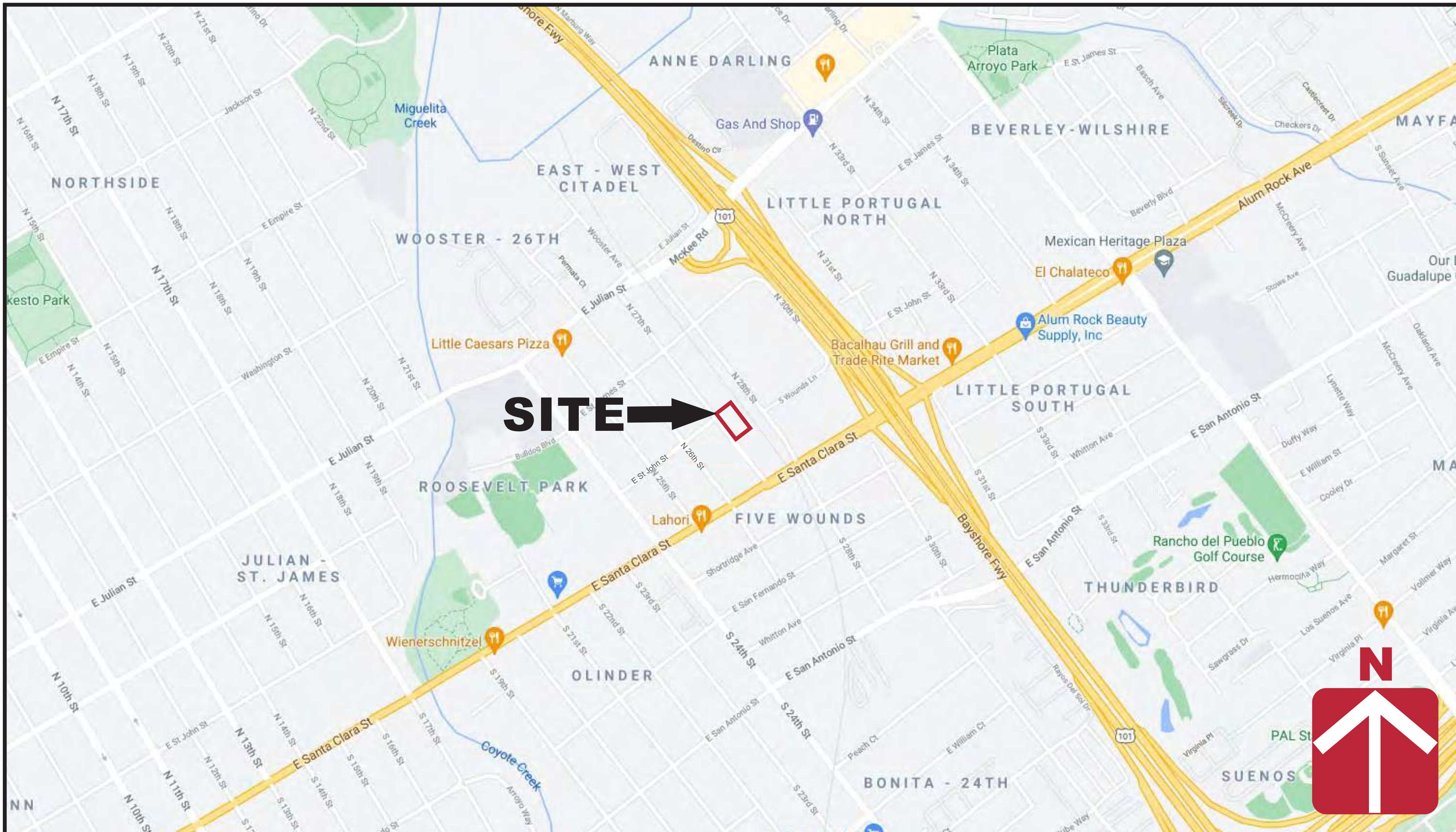
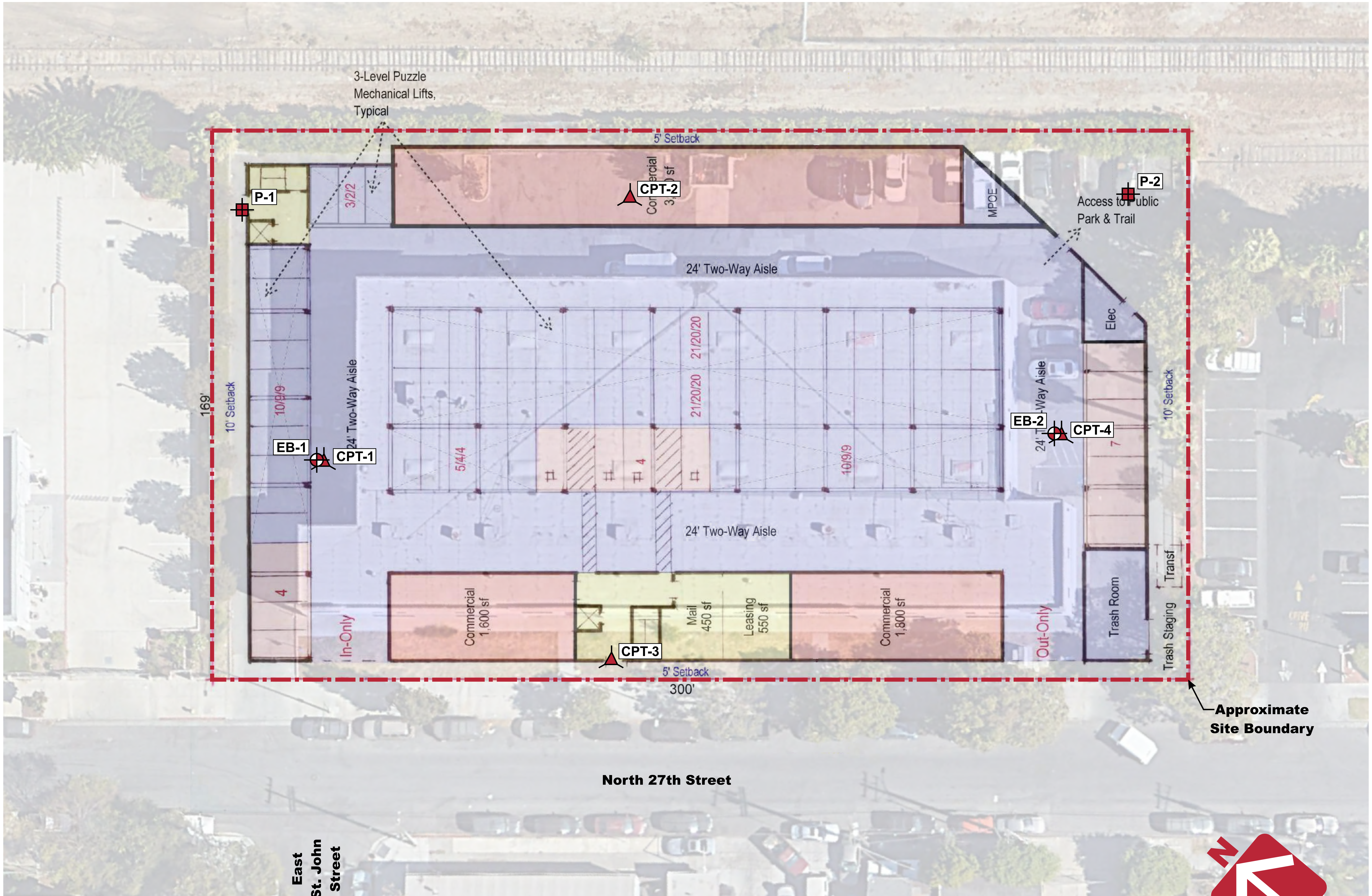


CORNERSTONE EARTH GROUP	**Vicinity Map**	Project Number 1285-1-2	
	North 27th Mixed-Use Development **70-80 North 27th Street** **San Jose, CA**	Figure Number Figure 1	
		Date November 2021	Drawn By RRN



Legend

- Approximate location of exploratory boring (EB)
- Approximate location of cone penetration test (CPT)
- Approximate location of percolation test (P)

0 30 60
APPROXIMATE SCALE (FEET)

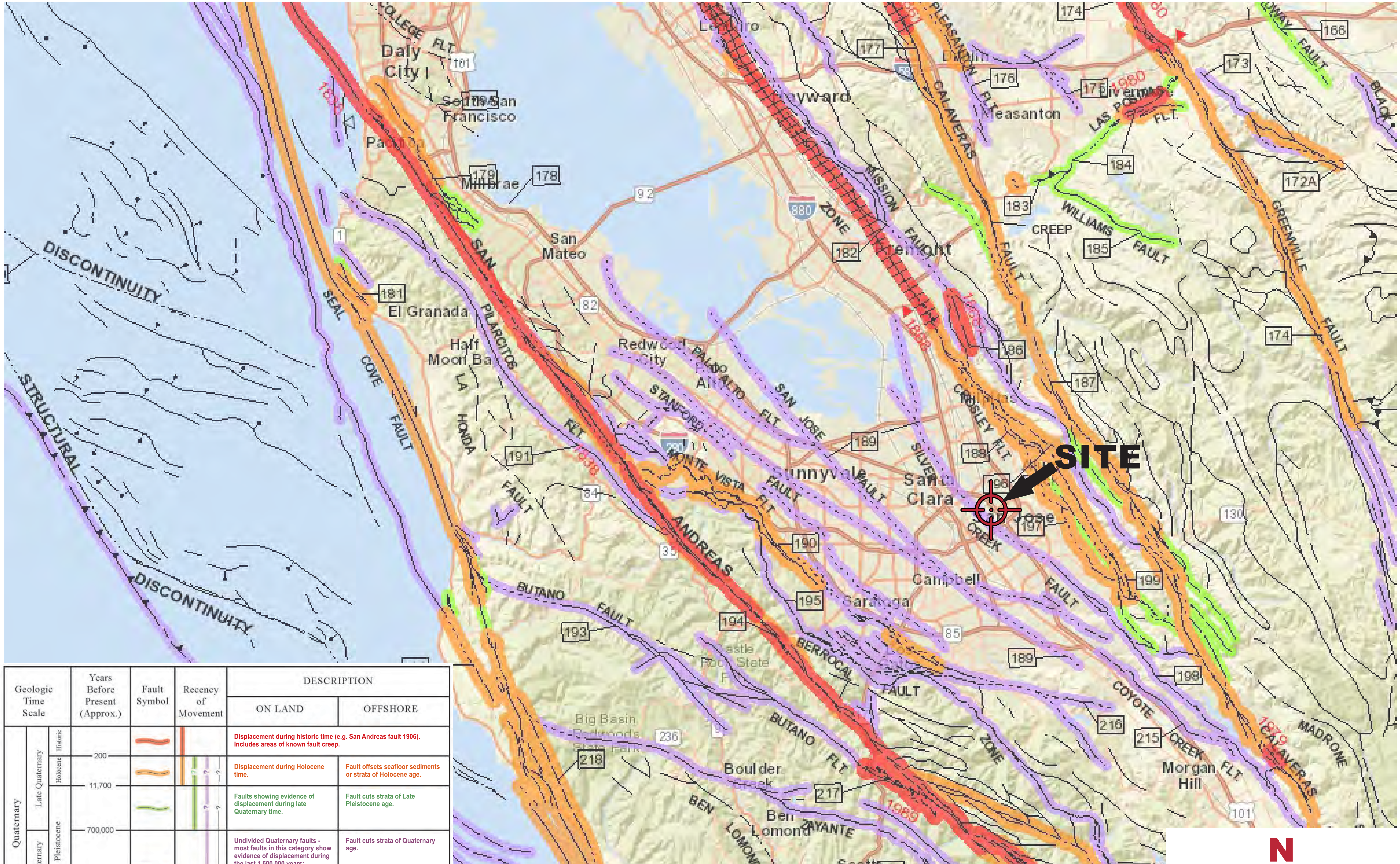
Base by Google Earth, dated 09/04/2020
Overlay: Sketch 09-10-21

Site Plan

North 27th Mixed-Use Development
70-80 North 27th Street
San Jose, CA

Project Number	1285-1-2
Figure Number	Figure 2
Date	November 2021
Drawn By	RRN

CORNERSTONE EARTH GROUP



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

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

Project Number 1285-1-2

Figure Number Figure 3

Date November 2021

Drawn By RRN

Regional Fault Map

North 27th Mixed-Use Development
70-80 North 27th Street
San Jose, CA

CORNERSTONE EARTH GROUP

CORNERSTONE EARTH GROUP

FIGURE 4B

CPT NO. 2

PROJECT/CPT DATA

Project Title	**North 27th Street Mixed-Use**
Project No.	**1285-1-2**
Project Manager	**NSD**

SEISMIC PARAMETERS

Controlling Fault	**Hayward**	
Earthquake Magnitude (Mw)	**7.08**	
PGA (Amax)	**0.7**	(g)

SITE SPECIFIC PARAMETERS

Ground Water Depth at Time of Drilling (feet)	**16.3**
Design Water Depth (feet)	**8**
Ave. Unit Weight Above GW (pcf)	**120**
Ave. Unit Weight Below GW (pcf)	**122**

CPT ANALYSIS RESULTS

DRY SAND SETTLEMENT FROM 8 **FEET**

0.01 **(Inches)**

LIQUEFACTION SETTLEMENT FROM 50 **FEET**

0.18 **(Inches)**

TOTAL SEISMIC SETTLEMENT 0.2 **INCHES**

POTENTIAL LATERAL DISPLACEMENT

LDI[2] 0.13 **L/H** 118.4

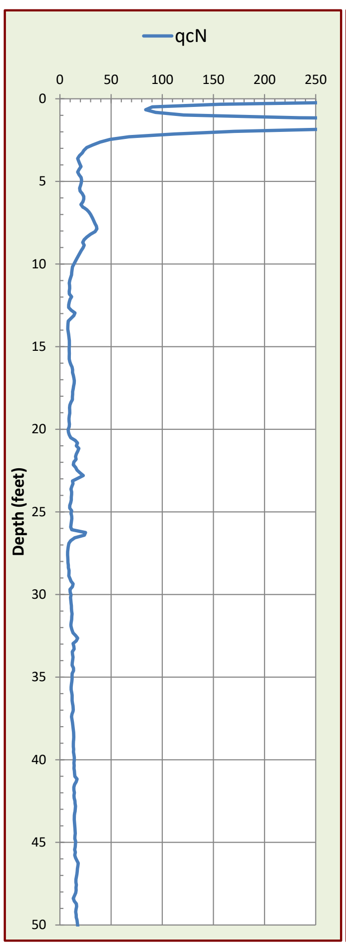
LDI[1] Corrected for Distance 0.02 (4 < L/H < 40)

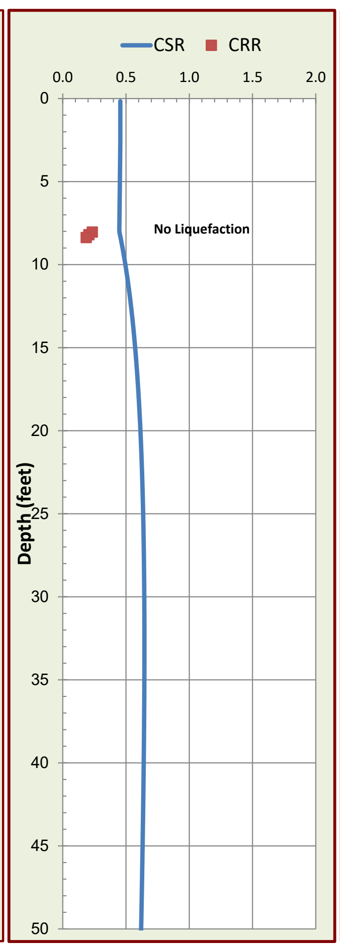
EXPECTED RANGE OF DISPLACEMENT

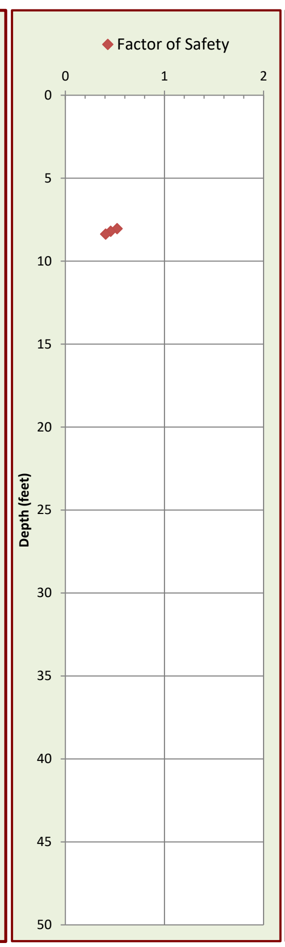
0.0 **to** **0.0** **feet**

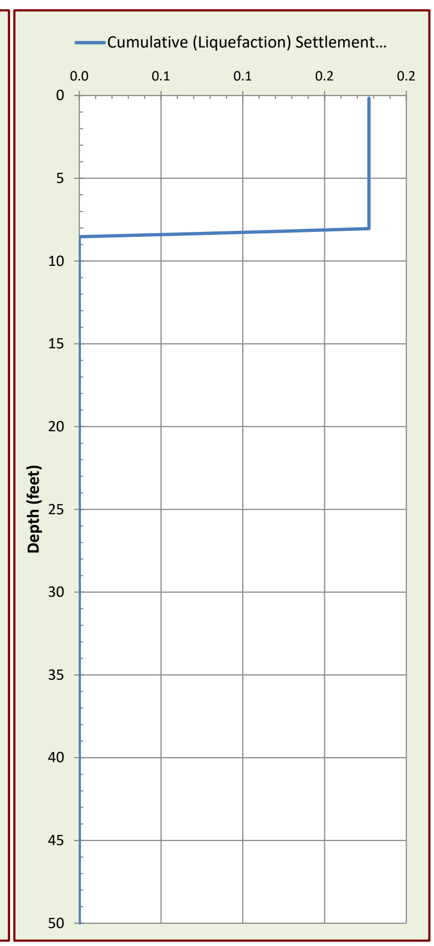
[1]Not Valid for L/H Values < 4 and > 40.

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









APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment and 25-ton truck-mounted Cone Penetration Test equipment. Two 8-inch-diameter exploratory borings were drilled on October 19, 2021 to depths of approximately 30 to 50 feet. Four CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on October 14, 2021 and October 18, 2021, to depths ranging from approximately 50 to 90 feet. The approximate locations of exploratory borings and CPTs are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

Boring and CPT locations were approximated using existing site boundaries and other site features as references. Boring and CPT elevations were based on interpolation of plan contours were not determined. The locations of the borings and CPTs should be considered accurate only to the degree implied by the method used.

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

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

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

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

APPENDIX B: LABORATORY TEST PROGRAM

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

Moisture Content: The natural water content was determined (ASTM D2216) on 21 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 20 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on one sample of the subsurface soils to aid in the classification of these soils. The result of these test is shown on the boring log at the appropriate sample depth.

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

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

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