Appendix D-2:

Preliminary Geotechnical Investigation

for Retail/Hotel Site

Type of Services	Preliminary Geotechnical Investigation
Project Name	North First Street Parcels
Location	North First Street San Jose, California
Client	Terra Hospitality
Client Address	461 S. Milpitas Boulevard, Suite 1 Milpitas, California 95035
Project Number	881-1-1
Date	April 20, 2016

DRAFT

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CORNERSTONE EARTH GROUP

	DRAFT
Type of Services	**Preliminary Geotechnical Investigation**
Project Name	**North First Street Mixed-Use Development**
Location	**North First Street** **San Jose, California**

SECTION 1: INTRODUCTION

This preliminary geotechnical investigation report was prepared for the sole use of Terra Hospitality for the North First Street Mixed-Use Development project in San Jose, California. The purpose of this study was to evaluate the existing subsurface conditions and develop an opinion regarding potential geotechnical concerns that could impact the proposed development. The preliminary geotechnical recommendations contained in this report are for your forward planning, cost estimating, and preliminary project design. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- A Conceptual Site Plan titled, "North 1st Street, Alviso, Conceptual Site Plan," prepared by the KTGY Group Inc. and HMH Engineers, dated February 8, 2016.
- A report titled, "Phase II Environmental Site Assessment, 4701 North First Street, San Jose, California," prepared by E_2C, Inc., dated September 20, 2004.

1.1 PROJECT DESCRIPTION

The planned 35-acre project site is located on the south side of North First Street near the intersection with Trinity Park Drive in San Jose, California. The site is currently occupied by an existing golf driving range, parking lot, maintenance facility, and open lots.

Based on our review of the revised conceptual development plans prepared by KTGY Inc. and HMH Engineers dated February 8, 2016, the planned mixed-use development will be constructed on a 21.6-acre portion of the site (Areas 1, 2 and 3). The proposed TopGolf facility on the remaining 13.4 acres (Area 4) is not included in this report as we previously prepared a draft geotechnical report dated February 10, 2016 for the TopGolf facility.

Area 1 will include a 4-story hotel with up to 200 rooms and approximately 17,000 square feet of commercial retail space (two additional buildings). Area 2 will include six retail/commercial buildings totaling 42,800 square feet. Area 3 will include two retail/commercial buildings totaling 56,600 square feet. Each development area will be constructed on an above-ground podium with parking beneath at the ground level. Appurtenant parking, retaining walls, utilities, landscaping and other improvements necessary for site development are also planned.

The site is bounded by Liberty Street and single family residential development to the west, North First Street, a school and residential development to the north, the Guadalupe River channel to the south, and the 237 & 1st commercial development to the east. The Guadalupe River in this area is a man-made channel bordered by flood control levees.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated February 9, 2016 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare preliminary recommendations for site work, building foundations, and pavements, and preparation of this preliminary report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 PREVIOUS EXPLORATION BY OTHERS

Previous field explorations in Areas 1 through 3 were performed by United Soil Engineering (USE), in 1996 and 2001. USE performed 11 borings on the site in total, with nine performed in 1997 (PB-1 through PB-7, PB-10 and PB-11) and two more performed in 2001 (B-1 and B-7). All borings were drilled with truck-mounted, hollow-stem auger drilling to a maximum depth of approximately 30 feet. Terrasearch Inc. performed a number of borings and test pits on the 35 acre site, but a complete site plan of the work could not be located. Hence these borings are not shown on the site plan. The approximate locations of the previous borings performed by others are shown on the Site Plan, Figure 2; the previous field and lab data is presented in Appendix E.

1.4 EXPLORATION PROGRAM

Our recent field exploration consisted of four borings drilled April 5 and 6, 2016 with track-mounted, limited-access hollow-stem auger drilling equipment and six Cone Penetration Tests (CPTs) advanced on March 21 and 22, 2016. The borings were drilled to depths ranging from 50 to 55 feet; the CPTs were advanced to depths of 50 to 75 feet. Three of the borings (Borings EB-1, EB-2, and EB-3) were advanced near CPT-1, CPT-3, and CPT-5, respectively, for direct evaluation of physical samples to correlated soil behavior.

The borings and CPTs were backfilled with cement grout in accordance with Santa Clara Valley Water District 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.

1.5 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, and Plasticity Index tests. Details regarding our laboratory program are included in Appendix B.

1.6 CORROSION EVALUATION

Two samples from our borings from depths from the upper 5 feet were tested for saturated resistivity, pH, and soluble sulfates and chlorides. JDH Corrosion Consultants prepared a brief corrosion evaluation based on the laboratory data, which is attached to this report in Appendix C. In general, the on-site soils can be characterized as corrosive to buried metal, and non-corrosive to buried concrete.

1.7 ENVIRONMENTAL SERVICES

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

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

The site is located within the Santa Clara Valley, which is a broad alluvial plane between the Santa Cruz Mountains to the southwest and west, and the Diablo Range to the northeast. The San Andreas Fault system, including the Monte Vista-Shannon Fault, exists within the Santa Cruz Mountains and the Hayward and Calaveras Fault systems exist within the Diablo Range. Alluvial soil thicknesses in this area of Santa Clara Valley and north San Jose range from about 500 to greater than 700 feet (Rogers & Williams, 1974).

2.2 REGIONAL SEISMICITY

The San Francisco Bay area is one of the most seismically active areas in the Country. While seismologists cannot predict earthquake events, the U.S. Geological Survey's Working Group on California Earthquake Probabilities 2015 revised earlier estimates from their 2008 publication (UCERF2, 2008). Compared to the previous assessment issued in 2008, the estimated rate of earthquakes around magnitude 6.7 (the size of the destructive 1994 Northridge earthquake) has decreased by about 30 percent. The expected frequency of such events statewide has dropped from an average of one per 4.8 years to about one per 6.3 years.

However, in the new study, the estimate for the likelihood that California will experience a magnitude 8 or larger earthquake in the next 30 years has increased from about 4.7% for UCERF2 to about 7.0% for UCERF3.

UCERF3 estimates that each region of California will experience a magnitude 6.7 or larger earthquake in the next 30 years. Additionally, there is a 63 percent chance of at least one magnitude 6.7 or greater earthquake occurring in the Bay Area region between 2007 and 2036.

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)	4.9	7.9
Hayward (Total Length)	7.1	11.5
Calaveras	9.0	14.5
Monte Vista-Shannon	9.4	15.2
San Andreas (1906)	12.7	20.5

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 available historic aerial photographs at www.historicaerials.com dating back to 1948. Aerial photographs indicate the site had been used for agricultural purposes since at least 1948, when the majority of the site and surrounding area was used for row crops or orchards. The original meandering Guadalupe River channel is visible in the 1948 and 1956 photographs, but was re-routed, straightened and channelized sometime between 1960 and 1968. Levee were constructed on both sides of the realigned river channel. A remnant piece of the Guadalupe River channel still remains to the southeast of the site (southeast of Area 4).

The 1980 photograph shows signs of grading (fill) activity, and by the 1987 photograph, the crops had been completely removed and/or filled. The existing golf course to the south of the site and the previous RV storage lot were constructed sometime between 1993 and 1999. The site remained relatively unchanged after 1999. The RV storage lot was occupied until sometime between June 2015 and October 2015 based on Google Earth images.

3.2 SURFACE DESCRIPTION

The 21.6-acre site is currently comprised of three areas, including an undeveloped parcel on the west end of the site adjacent to Liberty Street that has one single-family residence at the northwest corner of the parcel, a vacant, asphalt-covered parcel formerly used as equipment and RV storage, and the northern portion of the existing Pin High Golf Center and associated parking lot. The northern portion of the existing Pin High driving range, a former golf fairway and the associated parking lot are to be included in this phase of the project. All of these parcels are bordered by chain link fencing.

The parcels are relatively level, but developed parcels appear to be graded to drain to storm water facilities. The driving range is lined with a tall net and associated timber net poles. Areas outside of the existing improvements are generally covered with grass or bare ground. Two sand traps are located on the practice hole that is located at the northeast corner of the site. The former RV storage parcel is covered with approximately 1 to 2 inches of asphalt concrete overlying 3 to 4 inches of granular base. The western undeveloped parcel was covered with low weeds at the time of our exploration.

The nearby Guadalupe River Trail (levee) is situated approximately 4 to 8 feet above existing driving range site grades. Based on topography provided by Google Earth, existing site grades generally range from about Elevation 6 to 12 feet on the Pin High parcel, about Elevation 4 to 6 feet on the former RV storage parcel, and about Elevation 3 to 5 feet on the western vacant parcel. A few localized mounded areas and the area around the existing clubhouse building extend to roughly Elevation 13 feet. The top of the nearby existing levee ranges from approximately Elevation 18 to 20 feet.

3.3 SUBSURFACE CONDITIONS

Below the surface grades, our explorations generally encountered several feet of undocumented fill across the site over interbedded native alluvial soil. A brief description of each of these soil types is presented below.

Undocumented Fills: Based on our review of the subsurface conditions noted in the prior geotechnical reports and in our recent investigation, the existing fills range from about 5 to 14 feet in thickness. Borings EB-1 and EB-2 drilled on the former RV storage parcel encountered approximately 5 to 6 feet of fill consisting of loose to medium dense clayey sand with gravel and very stiff to hard sandy lean clay with gravel. Borings EB-3 and EB-4 drilled on the Pin High parcel encountered approximately 6 to 7 ½ feet of fill consisting of stiff to hard lean and fat clay with varying percentages of sand and gravel. Some minor concrete and asphalt rubble was encountered in Boring EB-4 at a depth of about 5 to 6 feet. The fill depth was generally consistent with the conditions encountered in the previous investigations. Previous borings drilled on the western undeveloped parcel in 1996 did not encounter fill. Previous boring PB-11 (1997) and B-7 (2001) drilled in the Pin High Golf Center parking lot encountered approximately 5 and 14 feet of fill, respectively. The fill in previous boring B-7 reportedly encountered abundant debris and concrete fragments.

Alluvial Soils: Below the undocumented fills or where no fills exist, the native alluvial soils generally consist of interbedded layers of soft to very stiff, low to high plasticity lean and fat clay to depths of about 14 to 22 feet. Below these depths, the clay is interbedded with layers of loose to medium dense silty and clayey sand to a depth of approximately 28 to 34 feet. Below these depths, our recent exploration encountered medium dense to dense sands and gravelly sands with variable amounts of silt and clay fines to depths of 75 feet, the maximum depth explored. The exception was in CPT-6, where a stiff silty clay layer was encountered between depths of approximately 66 and 72 feet.

3.3.1 Plasticity/Expansion Potential

We performed four Plasticity Index (PI) tests on representative samples during our current investigation; additionally, five PI tests were performed on our previous investigation for the TopGolf facility and six PI tests were performed between the two previous investigations. Test results from the current investigation were used to evaluate the plasticity of the fines in potentially liquefiable layers, while results from the previous investigation focused on evaluating the expansion potential of the existing surficial soils. The results of the PI tests of the fines contents in potentially liquefiable layers indicated PIs of 11; the results of the surficial PI tests indicated PIs ranging from 10 to 31. For saturated soils below the ground water level with PIs of 7 or greater generally indicate the corresponding layer has fines that are unlikely to liquefy. Liquefaction potential is discussed further in the "Geologic Hazards" section of this report. The surficial PI's indicated a low to moderate expansion potential to wetting and drying cycles, with a few localized areas where high plasticity clays were encountered.

3.3.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from approximately 0 to 10 percent over the estimated laboratory optimum moisture.

3.4 GROUND WATER

Ground water was previously encountered at depths ranging from approximately 5 to 16 feet in the Terrasearch investigation of 1987 and approximately 10 to 13 feet in the United Soils Engineering investigation of 1996. However, ground water was not reported in any of the borings to the maximum depth explored of 30 feet in the United Soils Engineering update report (2001). Ground water was estimated at depths of 7½ to 14 feet below current grades based on pore pressure dissipation testing performed during the Cone Penetration Tests. Ground water was not encountered in our recent exploratory borings due to the rotary wash drilling method used. Ground water was encountered during our investigation for the adjacent Area 4 Topgolf parcel (2016) at depths of approximately 17 to 22 feet.

All measurements were taken at the time of exploration and may not represent the stabilized levels that can be higher than the initial levels encountered. Historic high ground water maps of the vicinity indicate that water levels at the site are approximately 5 feet below site grades (CGS, 2004). For our analysis, we assumed a high ground water level to be at 5 feet below current grades.

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

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. The site is not located within a State-designated Alquist-Priolo Earthquake Fault Zone or a Santa Clara County Fault Hazard Zone. As shown in Figure 3, no known surface expression of fault traces is thought to cross the site; therefore, fault rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration (PGA) was estimated for analysis using a value equal to F_{PGA}xPGA, as allowed in the 2013 edition of the California Building Code. For our liquefaction analysis we used a PGA of 0.5g.

4.3 LIQUEFACTION POTENTIAL

The site is within a State-designated Liquefaction Hazard Zone (CGS, Milpitas Quadrangle, 2006) as well as a Santa Clara County Liquefaction Hazard Zone (Santa Clara County, 2003). Our field and laboratory programs addressed this issue by sampling potentially liquefiable layers to depths of at least 50 feet, performing visual classification on sampled materials, evaluating CPT correlations, and performing various tests to further classify the soil properties.

4.3.1 Background

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

4.3.2 Analysis

As discussed in the "Subsurface" section above, several sand layers were encountered below the design ground water depth of 5 feet. Following the liquefaction analysis framework in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008), incorporating updates in *CPT and SPT Based Liquefaction Triggering Procedures* (Boulanger and Idriss, 2014), and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008) for quantitative analysis, these layers were analyzed for liquefaction triggering and potential post-liquefaction settlement. These methods compare the ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil's estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.3 are considered to be potentially liquefiable and capable of post-liquefaction re-consolidation (i.e. settlement).

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

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

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

The results and calculations from our CPT analyses (CPT-1 through CPT-6) are presented on Figures D-1 through D-6 in Appendix D.

4.3.3 Summary

Our analyses indicate that several layers could potentially experience liquefaction triggering that could result in post-liquefaction total settlement at the ground surface ranging from about 1¼ to 2¼ inches 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 over a horizontal distance of 50 feet. In our opinion, differential settlements are anticipated to be on the order of 1 inch between independent foundation elements, assumed over a horizontal distance of about 30 feet.

4.3.4 Ground Rupture Potential

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

4.4 LATERAL SPREADING

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

The Guadalupe River runs approximately 400 to 600 feet south of the project site. The channel bottom is at approximately Elevation 0 feet. Therefore, the channel bottom is approximately 5 to 11 feet deep relative to existing site grades. As part of our liquefaction analyses, we calculated the Lateral Displacement Index (LDI) for potentially liquefiable layers based on methods presented in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008). LDI is a summation of the maximum shear strains versus depth, which is a measurement of the potential maximum displacement at that exploration location. Summations of the LDI values to a depth equal to twice the open face height were included. Estimated displacements in the area of CPT-1 through CPT-6 based on the LDI calculations are on the order of a 0 to 2 inches. Since the proposed mixed-use portion of the development addressed in this report is approximately 400 to 600 feet from the Guadalupe River levee and because the development will likely be supported over ground improvement areas or deep foundations, in our opinion, the potential for lateral spreading to affect the project is low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. The soils above the ground water table are relatively clayey, however, localized zones of existing undocumented fill may be susceptible to seismic densification. Provided the surficial fills are mitigated as recommended in this report and/or buildings are supported on ground improvement or deep foundations, in our opinion, the potential for significant differential seismic settlement affecting the proposed development is low.

4.6 TSUNAMI/SEICHE

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

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

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the study of tsunami inundation potential for the San Francisco Bay Area (Ritter and Dupre, 1972), areas most likely to be inundated are marshlands, tidal flats, and former bay margin lands that are now artificially filled, but are still at or below sea level, and are generally within 1½ miles of the shoreline. The site is approximately 4 miles inland from the San Francisco Bay shoreline, and is approximately 5 to 13 feet above mean sea level. Therefore, the potential for inundation due to tsunami or seiche is considered low.

4.7 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone AE, an area where the base flood elevation is shown as Elevation 12 feet. We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

From a geotechnical viewpoint, the project is feasible provided the concerns listed below are addressed in the project design. The preliminary recommendations that follow are intended for conceptual planning and preliminary design. A design-level geotechnical investigation should be performed once site development plans are finalized indicating where proposed structures finished floor elevations are planned. The design-level investigation findings will be used to confirm the preliminary recommendations and develop detailed recommendations for design and construction. Descriptions of each geotechnical concern with brief outlines of our preliminary recommendations follow the listed concerns.

- Potential for significant static and seismic settlements
- Presence of undocumented fills
- Presence of existing expansive surficial soils
- Construction difficulties due to shallow, soft soils

- Shallow ground water
- Soil corrosion potential

5.1.1 Potential for Significant Static and Seismic Settlements

Static Settlement from New Fill and Building Loads: We assume that new fill on the order of 2 to 6 feet thick may be required to achieve the proposed building pad and parking lot grades. We anticipate settlements on the order of 1 to 2 inches could occur in the underlying existing fills and alluvial soils, mainly due to consolidation of the alluvial soils from the fill placement. In addition, we estimate that additional static settlement on the order of ½ to 1½ inches could occur due to the weight of new buildings. This range of settlement is very preliminary and will depend on the final building loads and finished floor elevations for each building area. Roughly one-half of the combined static settlement will likely occur during construction.

Seismic Settlements: In addition to the potential for significant static fill and foundation settlement discussed above, our analysis indicates the potential for significant total and differential settlements during strong seismic shaking is high in the proposed building areas. Based on our analyses, total building settlement resulting from seismic loading conditions is estimated to be on the order of 1¼ to 2¼ inches, with differential settlement within a typical proposed building footprint estimated to be up to roughly 1 inch between independent foundation elements, assumed across a horizontal distance of about 30 feet. To mitigate the potential for significant differential movement, the proposed structure may be supported on spread footings overlying ground improvement or on deep foundations with surrounding non-structural, ground improvement piles to address down drag forces. Buildings with relatively light building loads, such as the shops buildings, can likely be supported on conventional shallow footings or a mat foundation provided they can be designed to tolerate anticipated seismic settlement. A discussion of potential mitigation options is presented in the "Foundations" section of this report.

5.1.2 Lateral Spreading Due to Liquefaction and Guadalupe River Bank Stability

As discussed in the "Geologic Hazards" section of this report, our analyses indicate a potential of localized lateral spreading on the order of 2 inches or less is theoretically possible due to the presence of the nearby river bank, the variable subsurface conditions and following strong ground shaking. However, because proposed buildings will likely be underlain by ground improvement elements or be supported on a conventional deep foundation system, the potential impact to the buildings is considered low. Since the remainder of the site outside the building footprints will primarily consist of the surface parking, the potential for minor ground cracks due to localized lateral spreading could theoretically be possible. Surface improvements, including sidewalks and asphalt concrete pavements located between the buildings and the river channel may experience minor movement following strong ground shaking.

5.1.3 Undocumented Fill

As discussed, with the exception of the western undeveloped parcel, undocumented (man-made) fills blanket the site that ranged from approximately 5 to 14 feet thick at previous and current exploration locations. The fill encountered in our borings and the previous explorations performed by Terrasearch (1987) and U.S.E. (1997, 2001) contained minor debris (such as concrete and asphalt fragments) mixed within the matrix of soil fill within the Pin High Golf Center parcels. Based on our review of the existing fill stiffness and consistency, the potential for future total and differential settlement of the overlying fill layer is considered low to moderate. Preliminary recommendations addressing this concern for building and parking areas are discussed below.

Building Area: Since it may not be practical to remove and re-compact the undocumented fill within the building area, and because settlement mitigation will likely be required for static and seismic settlement beneath many of the buildings, it is our opinion that only a partial re-compaction of existing fills would be needed in the building area. Therefore, on a preliminary basis and prior to placing new fills within the building areas, the upper 2 to 3 feet should be over-excavated and re-compacted as engineered fill. The actual lateral extent of the fill over-excavation should be further evaluated during the design-level investigation and future construction.

Parking Areas: Within the proposed parking area, on a preliminary basis, the upper 12 inches of fill should be re-compacted as engineered fill prior to placement of the new fills or pavement sections. Differential settlement in parking areas could result in increased maintenance for pavements and flatwork, as well as localized ponding of surface water. The civil engineer should consider this when designing surface drainage and storm drain facilities. Future maintenance across the parking area may include sealing pavement cracks, repairing cracked or offset curbs and sidewalks, or re-leveling localized depressed pavement areas. For planning purposes, localized pavement settlement, if it occurs, will likely be gradual and occur over a few to several years.

5.1.4 Shallow Ground Water

As discussed, ground water was measured at depths ranging from about 5 to 14 feet below the existing ground surface during our recent investigation and previous investigations. Our experience with similar sites in the vicinity indicates that shallow ground water could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable pavement subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering and shoring of utility trenches deeper than 5 feet may be required.

5.1.5 Presence of Existing Expansive Surficial Soils

As previously discussed, expansive surficial soils generally blanket most of the site. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. To reduce the potential for damage to the planned structures, slabs-on-grade should have sufficient reinforcement and be supported on a layer of non-expansive fill; footings should extend below the zone of seasonal moisture fluctuation. In addition, it is important to limit moisture changes in the surficial soils by using positive drainage away from buildings as well as limiting landscaping watering. Potential import sources for the site should be limited to a Plasticity Index of 20 or less. Preliminary grading and foundation recommendations addressing this concern are presented in the following sections.

5.1.6 Soil Corrosion

A preliminary soil corrosion screening was performed by JDH Corrosion Consultants based on the results of analytical tests on samples of the near-surface soil. In general, the JDH report concludes that the corrosion potential for buried concrete does not warrant the use of sulfate resistant concrete. In addition, the corrosion potential for buried metallic structures, such as metal pipes, is considered corrosive. JDH recommends that special requirements for corrosion control be made to protect metal pipes. A discussion of the site corrosion evaluation is presented in Appendix C. As the preliminary soil corrosion screening was based on the results of limited sampling, consideration may be given to collecting and testing additional samples from the upper 5 to 8 feet during the design-level investigation for the site.

5.2 DESIGN-LEVEL GEOTECHNICAL INVESTIGATION

The preliminary recommendations contained in this feasibility study were based on limited site development information and limited exploration. As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that we be retained to 1) perform a design-level geotechnical investigation, once detailed site development plans are available; 2) 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; and 3) be present to provide geotechnical observation and testing during earthwork and foundation construction.

SECTION 6: EARTHWORK

6.1 ANTICIPATED EARTHWORK MEASURES

We recommend that any existing foundations, slabs and/or abandoned underground utilities be removed. On a preliminary basis, existing undocumented fill encountered during grading should be over-excavated to a depth on the order of 2 to 3 feet below existing site grades and re-compacted within the proposed building areas and to a lateral distance of about 5 feet beyond the building perimeter. Due to the variability of the existing fills, it may be necessary to re-compact fills to deeper depths in localized areas depending on the consistency of these fill materials. The actual lateral extent and depth of over-excavation should be confirmed during the design-level investigation.

Imported fill material for use as general fill should be predominantly granular, but have sufficient fines and a Plasticity Index of 20 or less. All fill as well as scarified surface soils in those areas to receive fill or new foundations should be compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D-1557, latest edition; and be at least 2 to 3 percent above optimum. The upper 6 inches of subgrade in pavement areas and all aggregate base materials should also be compacted to at least 95 percent relative compaction

(ASTM D-1557, latest edition). Utility trench backfill should be compacted to at least 90 percent relative compaction (ASTM D-1557, latest edition) by mechanical means only.

Surface water runoff should not be allowed to pond adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 1 to 2 percent towards suitable discharge facilities. Due to the sandy soils encountered at the site, aggressive erosion control will likely be required during construction, such as covering disturbed areas with jute netting or straw matting to reduce erosion of exposed sandy materials.

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

On a preliminary basis, ground improvement should be implemented to mitigate the potential for liquefaction-induced settlement and potential consolidation settlement of the proposed buildings. We believe ground improvement can be used to mitigate the settlement to tolerable levels and, provided the recommendations in the "Earthwork" section are followed, the proposed structures can likely be supported on shallow foundations. A design-build ground improvement contractor will be responsible for the mitigation design using an appropriate ground improvement technique to meet the project requirements. The proposed buildings could also be supported on deep foundations, such as augercast piles; however, downdrag forces on the piles due to liquefiable layers as deep as 40 feet would result in lower allowable capacities. We have also included preliminary capacities for augercast piles that are installed in conjunction with ground improvement consisting of densification piles surrounding the load-bearing piles for your consideration.

Smaller, lightly loaded retail buildings, such as the one-story shop buildings, could potentially be supported on shallow foundations, with or without ground improvement, provided the buildings can be designed to tolerate higher levels of differential settlement. Further evaluation of potential foundation alternatives should be performed during the design-level geotechnical investigation.

7.2 SEISMIC DESIGN CRITERIA

We assume that the project structural design will be based on the 2013 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The "Seismic Coefficients" used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Based on our borings, interpreted SPT "N" values from the CPTs, and review of local geology, the site is underlain by deep alluvial soils with typical SPT "N" values between 15 and 50 blows per foot. Therefore, we have classified the site as Soil Classification D. The mapped spectral acceleration parameters S_S and S_1 were calculated using the USGS computer program *Earthquake Ground Motion Parameters*, Version 5.1.0, revision date February 10, 2011, based on the site coordinates presented below and the site classification.

The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 2: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.42449°
Site Longitude	-121.97091°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.500g
1-second Period Mapped Spectral Acceleration[1], S_1	0.600g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv	1.5
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.500g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	0.900g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.000g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.600g

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

Because the potential for liquefaction and the potential for affects to the structure appear high, based on Table 1613.5.2, Site Class Definitions, of the 2013 California Building Code (CBC), the site should be classified as Site Class F. Site Coefficients F_a and F_v are determined using Tables 1613.5.3(1) and 1613.5.3(2). Site Class F of those tables refers the determination of Site Coefficients F_a and F_v to Section 11.4.7 of ASCE 7-10. ASCE 7-10 generally indicates that sites classified as Site Class F shall have a site response analysis performed in accordance with Section 21.1 of ASCE 7-10, unless the proposed structure meets the following exception.

> **EXCEPTION:** For structures having fundamental periods of vibration equal to or less than 0.5s, site-response analysis is not required to determine spectral accelerations for liquefiable soils. Rather, a site class is permitted to be determined in accordance with Section 20.3 and the corresponding values of F_a and F_v determined from Tables 11.4-1 and 11.4-2.

For these reasons, in our opinion, the above Site Classification D and the presented seismic coefficients appear valid due to the above exception, as the structures likely have a fundamental period equal to or less than 0.5 seconds. The Project Structural Engineer should verify this assumption. If the structure will have a fundamental period of greater than 0.5 seconds, and meets the requirements for a Site Class designation of F, the requirement for a site response analysis may be triggered, and additional geotechnical analysis would need to be performed.

7.3 SHALLOW FOUNDATIONS

Provided ground improvement is performed for the proposed buildings, on a preliminary basis the structures can likely be supported on spread footings, which should bear on the ground improvement elements discussed in the later sections, be at least 18 inches wide, and extend at least 18 to 24 inches below the lowest adjacent grade. Bottom of footing is based on lowest adjacent grade, defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil.

Bearing pressures will be dependent on the final ground improvement technique and spacing; however, substantial improvement in bearing capacity would be expected. On a preliminary basis, we expect allowable bearing pressures of at least 5,000 psf for combined dead plus live loads would be feasible.

As discussed in the "Conclusions" section of this report, total and differential settlement due to combined static and seismic loading will likely be on the order of 2 to 3½ inches and 1 to 2 inches, respectively. Ground improvement should be designed to reduce total settlement due to static and seismic conditions to tolerable levels, as described below.

7.4 GROUND IMPROVEMENT

As discussed above, shallow spread footings supporting the proposed buildings may be used in combination with ground improvement. Ground improvement can be used to improve the subsurface soils such that the total combined static and seismic settlements are reduced to less than 1½ inches, and no more than 1-inch for either the static or seismic component, enabling the structures to be supported on spread footings. Ground improvement should provide adequate confining improvement around all foundations. We anticipate the use of a conventional slab-on-grade floor is desired; therefore, the spacing of the improved areas should be adjusted to accommodate this requirement. Ground improvement options should also include an increase in allowable bearing pressures as discussed above.

7.4.1 Ground Improvement Options

Due to the presence of the existing site fills that may not be feasible to rework, as well as the thickness and shallow ground water concerns, any ground improvement method will need to extend from bottom of footing and slab-on-grade subgrade (unless an engineered fill layer will be used within the slab-on-grade improvement area to increase the improvement column spacing slightly due to bridging effects) to the design improvement depth; therefore, methods such as compaction grouting and jet grouting have not been considered. In addition, if the site soils are environmentally impacted, we anticipate that a ground improvement method that results in a low permeability improved zone will be desired by the regulatory agency to reduce the possibility of cross-contamination of water between aquifers in water bearing zones. This excludes compacted gravel column methods such as stone columns and rammed aggregate piers. Based on our experience and discussions with local ground improvement contractors, we suggest that Drilled Displacement Columns (DDC) be considered for ground improvement.

The DDC methodology involves drilling a hole with a displacement augercast pile rig and during auger extraction filling the hole under low pressure with low-permeability, controlled low strength material to displace and densify the surrounding weak soil, as well as construct stiffer sand-cement columns. Limited soil cuttings are generated with the displacement auger; therefore, limiting handling and disposal of additional soil.

7.4.2 General

The intent of the ground improvement design beneath the proposed buildings would be to increase the density of the potentially liquefiable sands generally at the depths of about 35 to 45 feet by laterally displacing and/or densifying the existing in-place soils as well as create a stiffer overall soil profile to reduce the static settlement due to the medium stiff clays present between depths of about 10 to 20 feet.

The degree to which the soil density is increased will depend on the spacing. Even though the above methods are designed to mitigate different existing soil conditions, ground improvement should provide an additional increase in bearing capacity and soil stiffness at the individual improvement locations, which could be taken into consideration during evaluation of the post-construction consolidation settlements.

In general, a ground improvement contractor's design should include, but not be limited to: 1) drawings showing the ground improvement layout, spacing and diameter, 2) the foundation layout plan, 3) proposed ground improvement length, and 4) top and bottom elevations. We should be retained to review the ground improvement contractor's plan and settlement estimates and analysis methods prior to construction.

Ground improvement would generally be constructed as follows: 1) clear the site, 2) grade site to rough grades, 3) install the ground improvement on the approved layout (potential pre-drilling may be required in areas where buried debris may be encountered), and 4) in slab areas, excavate the upper two feet, or as needed, and replace as engineered fill to repair disturbance to the near-surface soils resulting from ground improvement installation.

7.4.3 Ground Improvement Performance Testing

Performance testing typically consists of a pre-construction test section with post-installation CPT testing in both footing and slab areas to confirm that the necessary composite soil strength increases were achieved to meet the settlement criteria. Post-installation CPT testing is also required during production installation. We should observe and monitor installation of the ground improvement on a full-time basis and review the post-installation settlement analyses provided by the contractor.

7.5 DEEP FOUNDATIONS

7.5.1 Augercast Piles

As an alternative to using ground improvement with shallow foundations, on a preliminary basis, drilled, cast-in-place, conventional or partial-displacement augercast piles (ACIP piles) may be used to support the proposed buildings. To mitigate potential downdrag on the structural, load-bearing piles resulting from potential liquefaction settlement, rings of non-structural densification piles can be constructed around each pile cap/group to provide ground improvement zones around the piles. We also considered, full displacement ACIP piles; however, due to the presence of sand layers up to 30 feet in thickness or greater, displacement piles may not be feasible.

ACIP piles are concrete piles that are cast in place using a hollow-stem auger that drills to the design depth and then sand-cement grout (4,000 to 6,000 psi grout) is pumped through the hollow-stem as the drill stem is extracted. Conventional ACIP piles remove the soil column, similar to a drilled pier, as the drill stem is advanced, and partial displacement ACIP piles partially displace the soil column as the drill stem is advanced (resulting in a smaller volume of drill spoils), prior to pumping the grout. ACIP piles are a low noise and vibration installation compared to driven piles. Various types of steel reinforcing, including rebar cages or H-piles may be installed into the still-wet grout after drilling to satisfy bending moment requirements. Partial-displacement ACIP piles would generate a smaller volume of drill spoils compared to conventional augercast piles. ACIP piles are generally limited to a depth of about 85 feet due to equipment limitations.

If the structures are to be supported on piles, the first floor should consist of a structural slab supported by grade beams.

7.5.2 Vertical Capacity and Estimated Settlement

If the proposed structural loads are supported on augercast piles, adjacent pile centers should be spaced at least three diameters apart; otherwise, a reduction for vertical group effects may be required. Grade beams should span between piles and/or pile caps in accordance with structural requirements.

Vertical capacity is based solely on frictional resistance. We evaluated the allowable vertical capacity for 16-inch diameter ACIP piles and estimate that a 55- to 60-foot deep pile could potentially derive 100 kips of capacity for dead plus live loading. This assumes piles are installed in conjunction with ground improvement, consisting of displacement piles surrounding the load-bearing piles.

We have assumed that the top of pile/bottom of pile cap occurs about 3 to 4 feet below proposed finished site grades. The allowable capacities are for dead plus live loads; dead loads should not exceed two-thirds of the allowable capacities. Allowable capacities may be increased by one-third for wind and seismic loads. In general, uplift loads should not exceed 75 percent of the allowable downward vertical capacity under seismic loading. Gross capacity of the piles should be less than the structural capacity of the piles.

7.5.3 Lateral Capacity

Lateral load resistance is developed by the soil's resistance to pile bending. The magnitude of the shear and bending moment developed within the pile are dependent on the pile stiffness, embedment length, the fixity of the pile into the pile cap (free or fixed-head conditions), the surrounding soil properties, the tolerable lateral deflection, and yield moment capacity of the pile.

Based on our experience with similar sites, a maximum shear resistance on the order of 8 to 15 kips could potentially be achieved for ¼ to ½ inch pipe head deflection for free-head conditions. This assumes a cracked (assumed 30 percent reduction) pile stiffness (EI) of 8.1×10^9 lb-in^2 for 16-inch diameter ACIP piles and a concrete compressive strength of greater than 4,000 psi for the concrete modulus.

Passive resistance against pile caps and grade beams poured neat against native or engineered fill may also be considered; however, as the allowable lateral deflections of the piles are limited, full allowable passive will not be developed. We should be retained to work with the structural engineer during the design-level investigation to evaluate appropriate allowable passive pressures that maintain strain compatibility between the piles and pile caps, if additional passive resistance is required.

SECTION 8: VEHICULAR PAVEMENTS

8.1 ASPHALT CONCRETE

The following preliminary asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual (latest edition), estimated traffic indices for various pavement-loading conditions, and on a preliminary design R-value of 10. The preliminary R-value was chosen based on engineering judgment considering the variable surface conditions; however, as the site will be raised a few feet using imported fill, an R-value test should be performed on a sample collected from the top 18 inches of finished subgrade for final design pavement recommendations.

Table 3: Preliminary Asphalt Concrete Pavement Recommendations

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

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

As an alternative, the aggregate base sections above may be reduced to a thickness of 4 inches up to a Traffic Index of 6.0 if the top 12 inches of subgrade is lime-treated with Quicklime for permanent improvement. Based on our experience, a minimum R-value of 50 can be achieved in lime-treated subgrade soils.

SECTION 9: LIMITATIONS

This preliminary report, an instrument of professional service, has been prepared for the sole use of Terra Hospitality specifically to support the design of the property referred to as the North First Street parcels 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.

Preliminary recommendations in this report are based upon the soil and ground water conditions encountered during our subsurface exploration and based on prior site investigations by others. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

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

Cornerstone prepared this preliminary 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 10: REFERENCES

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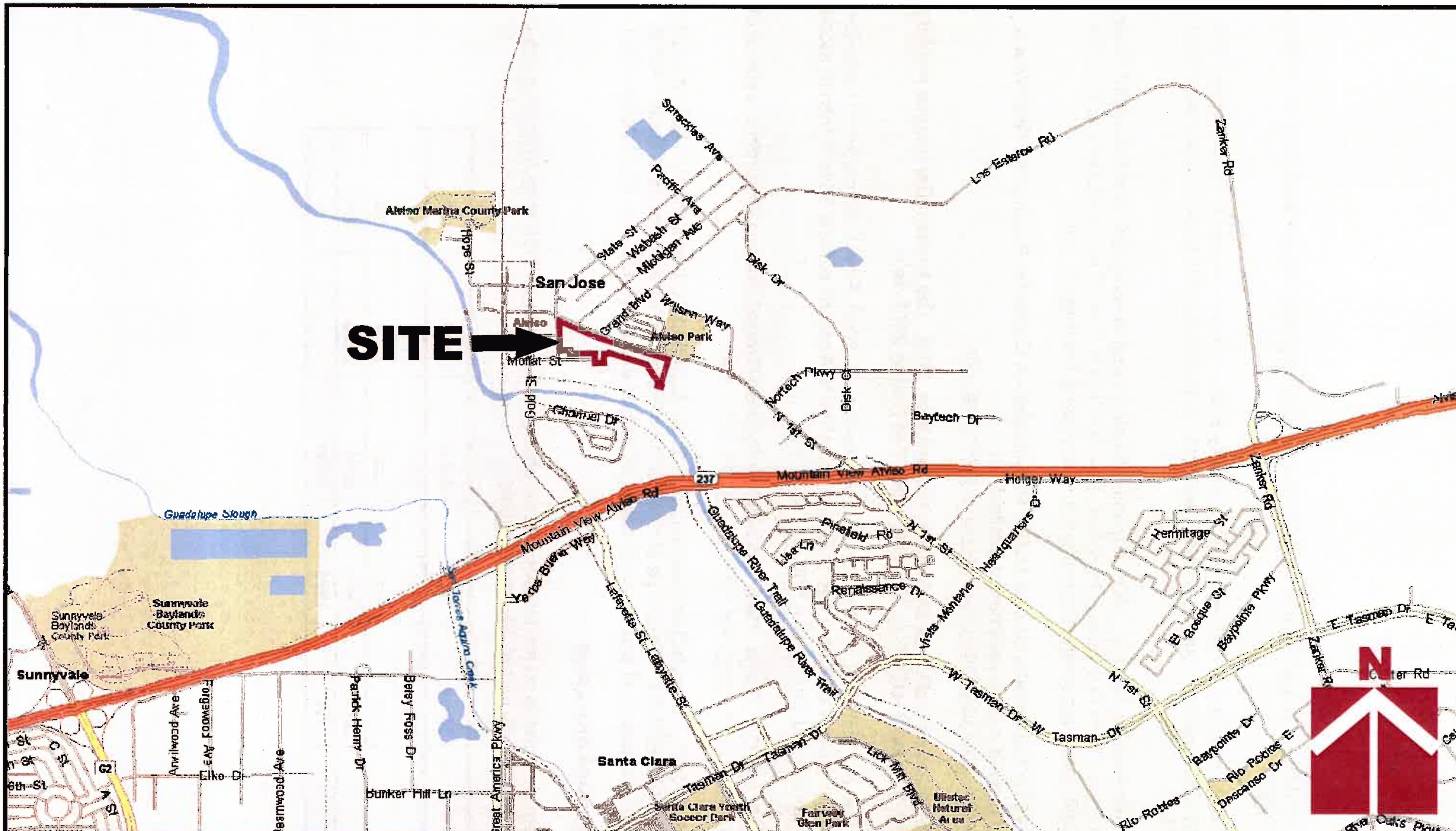
AERIAL PHOTOGRAPHS

Geomorphic features on the following aerial photographs obtained from www.historicaerials.com were interpreted as part of this investigation:

Year	Type	Year	Type	Year	Type
1948	B/W	1980	B/W	2002	Color
1956	B/W	1987	Color	2004	Color
1968	B/W	1993	B/W	2005	Color
1965	B/W	1999	Color		



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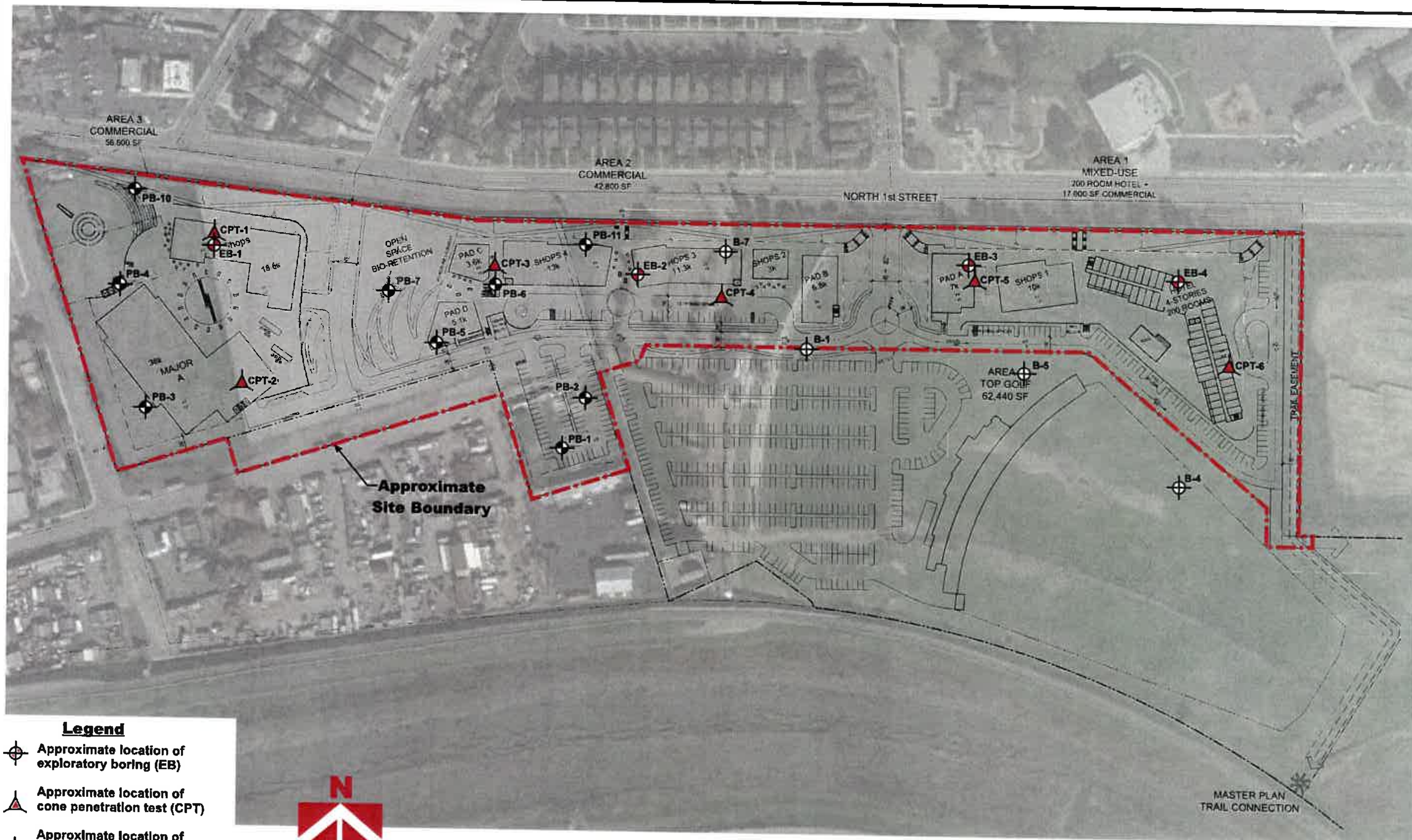
Vicinity Map	Project Number 881-1-1
North First Street Parcels San Jose, CA	Figure Number Figure 1
	Date April 2016 / Drawn By RRN



Legend

- Approximate location of exploratory boring (EB)
- Approximate location of cone penetration test (CPT)
- Approximate location of previous boring (PB) (United Soil Engineering, 1997)
- Approximate location of previous boring (PB) (United Soil Engineering, 2001)

0 160 320
APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 10/30/2015
Overlay by KTGY Group, Inc., Conceptual Site Plan, dated 2/4/2016

CORNERSTONE EARTH GROUP

Site Plan	Project Number	881-1-1
North First Street Parcels San Jose, CA	Figure Number	Figure 2
	Date April 2016	Drawn By RRN