Appendix D-1:

Preliminary Geotechnical Investigation

for Topgolf Facility Site

CORNERSTONE
EARTH GROUP

Type of Services	Geotechnical Investigation
Project Name	TopGolf San Jose
Location	North First Street San Jose, California
Client	Arco Murray
Client Address	3110 Woodcreek Drive Downers Grove, IL
Project Number	854-1-1
Date	February 10, 2016

DRAFT

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SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Arco Murray for the TopGolf San Jose project 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:

- A preliminary grading plan titled, "TopGolf – San Jose, Preliminary Grading Plan," prepared by Manhard Consulting LTD., dated November 6, 2015.
- A report titled, "Updated Geotechnical Investigation and Pavement Design, Land of Saint Claire Properties, Southeast Corner of North First Street and Liberty Street, San Jose, California," prepared by United Soil Engineering, Inc., dated February 7, 2001.
- A report titled, "Geotechnical Feasibility Evaluation on Proposed Mobile Home Park, St. Claire Corporation Property, Alviso, California," prepared by Terrasearch, Inc., dated March 25, 1987.
- 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 TopGolf facility will be constructed on an approximately 13½-acre portion of the site; while the remaining Parcels A through D are not included as part of the development area at this time. The project will include the construction of a three-story golf and entertainment building that includes 120 hitting bays, restaurants and appurtenant entertainment space. The hitting bays will face a driving range (outfield) equipped with electronic target areas covered with artificial turf. Large boundary nets will border the sides and end of the outfield. We assumed the facility will be of steel-frame or concrete construction with a slab-on-grade first floor. Appurtenant parking, utilities, landscaping and other improvements necessary for site development are also planned.

The site is bounded by commercial 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 & First Street Commercial Development to the east. The Guadalupe River in this area is a man-made channel bordered by flood control levees.

After reviewing the preliminary grading plans prepared by Manhard Consulting Ltd., it appears the building finished floors will be raised up approximately 0 to 3 feet from existing grades, to Elevation 13.00 feet for the golf and entertainment building. The driving range/outfield will include fills ranging from 1 to 7 feet at the highest point. In general, the high points will be located along the edges of the driving range.

Based on the information contained in the Geotechnical Investigation Scope of Work memo, maximum dead plus live column and wall loads of approximately 315 kips and 5.6 kips per foot, respectively, are expected for the three-story facility.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated September 4, 2015 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 PREVIOUS EXPLORATION PROGRAM BY OTHERS

Previous field exploration was performed by Terrasearch (1987) and United Soil Engineering (2001). The Terrasearch explorations on the site included seven borings (Borings 1 through 7) and 10 test pits (TP-1 through TP-10). The United Soil Engineer investigation consisted of 10 borings on the project site (B-1 through B-7, performed in 2001 and B-1, B-2, and B-11 performed in 1996) drilled with truck-mounted, hollow-stem auger drilling equipment. The previous borings were drilled to a maximum depth of approximately 30 feet. The approximate locations of the previous borings performed by others are shown on the Site Plan, Figure 2; the field and lab programs are available in Appendix E. A complete site plan for the Terrasearch investigation was not available; therefore, several of the borings and test pits are not shown on Figure 2.

1.4 EXPLORATION PROGRAM

Field exploration consisted of 17 borings drilled from January 18 through January 20, 2015 with track-mounted, limited-access hollow-stem auger drilling equipment and three Cone Penetration Tests (CPTs) advanced on January 20, 2015. Due to additional time required to extend through the concrete and asphalt debris, we were unable to complete a fourth CPT, which was planned in the scope of our proposal. The borings were drilled to depths ranging from 10 to 50 feet; the CPTs were advanced to depths of 48 to 100 feet. Seismic shear wave velocity measurements were collected from CPT-1. Two of the borings (Borings EB-1 and EB-3) were advanced near CPT-1 and CPT-3, respectively, for direct evaluation of physical samples to correlated soil behavior.

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

The approximate locations of our exploratory borings 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, grain size analyses, washed sieve analyses, Plasticity Index tests, an R-value test, and triaxial compression tests. Details regarding our laboratory program are included in Appendix B.

1.6 ENVIRONMENTAL SERVICES

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

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

The site is located within the Santa Clara Valley, which is a broad alluvial plane between the Santa Cruz Mountains to the southwest and west, and the Diablo Range to the northeast. The San Andreas Fault system, including the Monte Vista-Shannon Fault, exists within the Santa Cruz Mountains and the Hayward and Calaveras Fault systems exist within the Diablo Range. Alluvial soil thicknesses in this area of Santa Clara Valley and north San Jose range from about 300 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 along the southwestern edge of the site sometime between 1960 and 1968. A remnant piece of the Guadalupe River channel still remains to the southeast of the site.

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 was constructed sometime between 1993 and 1999. The site remained relatively unchanged after 1999.

3.2 SURFACE DESCRIPTION

The site is currently occupied by a one-story clubhouse and restaurant building, an at-grade parking lot, located in the northwestern corner of the site, a driving range, and associated surface improvements. The site is relatively level, but graded to drain to storm drainage 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. Several sand traps are located on the three practice holes that are located along the edges of the site.

The adjacent Guadalupe River Trail (levee) is situated approximately 4 to 8 feet above existing driving range site grades. Existing topographic information indicates existing site grades generally range from about Elevation 10 to 12 feet (datum unknown). A few localized mounded areas within the existing golf course fairways extends to roughly Elevation 16 feet. The top of the adjacent existing levee ranges from approximately Elevation 18 to 20 feet. The west end of the site drops to approximately Elevation 6 to 7 feet along the west property boundary.

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.

Undocumented Fills: Based on our review of the subsurface conditions noted in the prior geotechnical reports and in our own investigation, the fills range from about 7½ to 14 feet in thickness. The fills were highly variable in content and generally consisted of soil and asphalt, concrete, or wood debris. The fill depth and content was generally consistent with the conditions encountered in the previous investigations.

Alluvial Soils: Below the undocumented fills, the native alluvial soils generally consisted of interbedded layers of soft to very stiff, low to moderately plastic clay and occasional layers of loose to medium dense clayey or silty sand (generally less than about 2 feet in thickness) to a depth of about 32 feet below existing site grades. Loose to dense sands with variable amounts of silt and clay fines were encountered in our deep borings (or based on CPT correlations) from about 32 feet to 70 feet. The sands were underlain by medium stiff to very stiff clays that were encountered to the maximum depth explored of about 100 feet.

3.3.1 Plasticity/Expansion Potential

We performed five Plasticity Index (PI) tests on representative samples during our current investigation; additionally, 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 5 and 14; the results of the surficial PI tests indicated PIs ranging from 10 to 26. The PI results ranging up to 6 indicated a potential for the corresponding layers to be susceptible to liquefaction during seismic shaking, while the PIs of 9 or greater 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 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 approximately 0 to 13 percent over the estimated laboratory optimum moisture.

3.4 GROUND WATER

Ground water was estimated at depths of 12 and 13 feet below current grades based on pore pressure dissipation testing performed during the Cone Penetration Tests. Ground water was encountered at depths ranging from about 17 to 25 feet in our exploratory borings. Previous explorations encountered ground water at depths ranging from approximately 5 to 16 feet in the Terrasearch investigation (1987) and from 11 to 14 feet in United Soils Engineering investigation of 1996. Ground water was not reported in any of the borings to the maximum depth explored of 35 feet in the United Soils Engineering update report (2001). 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. 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 of our CPT analyses (CPT-1 through CPT-3) are presented on Figures 4A through 4C of this report. Calculations for these CPTs are attached as Appendix E.

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 ¾ to 1¾ 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 between independent foundation elements. In our opinion, differential settlements are anticipated to be on the order of ½ to 1¼ inches between independent foundation elements, assumed over a horizontal distance of about 35 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 along the southwester boundary of the site. The channel bottom is approximately 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-3 based on the LDI calculations are on the order of a few inches to about one foot.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. Provided the surficial fills are addressed as recommended in this report, in our opinion, the potential for significant differential seismic settlement affecting the proposed buildings is low. Some seismic differential settlement should be anticipated in the deeper fill areas across the site within pavement, driving range, and landscape areas.

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 4½ miles inland from the San Francisco Bay shoreline, and is approximately 8 to 15 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. Descriptions of each concern with brief outlines of our recommendations follow the listed concerns.

- Potential for significant static and seismic settlements
- Lateral spreading due to liquefaction and Guadalupe River bank stability
- Undocumented fill with buried debris
- Shallow ground water
- Presence of existing expansive surficial soils

5.1.1 Potential for Significant Static and Seismic Settlements

Static Settlement from New Fill Placement: As discussed, based on existing grades, we anticipate up to 6 feet of new fill will be required along the edges of the planned driving range/outfield to achieve the proposed grades. We anticipate settlements of less than 2 inches could occur in the underlying existing fills and alluvial soils, mainly due to consolidation of the alluvial soils from the fill placement.

Fill Containing Construction Debris: As discussed in the "Subsurface Conditions" section, we encountered 7½ to 14 feet of fill that generally consisted of medium dense clayey sand and medium stiff to very stiff clays mixed with construction debris, primarily consisting of asphalt and concrete rubble. The upper few feet of fill was predominantly soil and the deeper fills consisted of a mixture of soil and debris. Considering the stiffness of these fills, as well as the composition, frequency and thickness of the construction debris encountered, in our opinion, the static settlement estimates from the planned fill placement discussed above are representative, and that significant long term settlements typically associated with municipal landfill sites are not anticipated.

Significant Foundation Settlements: In addition to the potential for significant static foundation settlements, which we estimate to be greater than 1-inch from the range of building loads, as well as areal fill settlements less than 2 inches for the added fill, our analyses indicated the potential for significant differential settlements during strong seismic shaking is moderate to high in the proposed building area. Based on our analyses, total building settlement resulting from combined static and seismic loading conditions is estimated to range from about 2 to 3½ inches, with differential settlement within the proposed building footprint estimated to be about 1½ to 2½ inches 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. 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 lateral spreading on the order of a few inches to about one foot is theoretically possible due to the presence of the river bank and subsurface conditions and following strong ground shaking. To mitigate this potential, we recommend that a shear key of improved soil be constructed between the golf and entertainment building (and associated improvements) and Guadalupe River, located along the southwest edge of the site. The ground between the improved soil and the river channel should be anticipated to slump and spread toward the creek channel during a large seismic event. Surface improvements, including pathways and pavements located beyond the shear key areas, may experience movement during the considered ground motion. Additional recommendations are provided later in this report.

5.1.3 Undocumented Fill with Buried Debris

As discussed, undocumented fills blanket the site that ranged from approximately 7½ to 14 feet at previous and current exploration locations. The fill encountered in our borings and the previous explorations performed by Terrasearch (1987) and U.S.E. (2001) contained significant debris (such as concrete and asphalt fragments and minor wood debris) mixed within the matrix of soil fill. Based on our review of the condition of the existing fill, the potential for future total and differential settlement of the overlying fill layer is considered moderate to high. Recommendations for building and parking areas are discussed below; detailed recommendations are presented in the "Earthwork" section of this report.

Building Area: Since it is not likely practical to remove and re-compact the undocumented fill within the building area, and because settlement mitigation will be required for static and seismic settlement beneath the building, it is our opinion that only a partial re-compaction of existing fills would be needed in the building area. Therefore, prior to placing new fills within the building area, the upper 24 inches should be re-compacted as engineered fill and any oversized fragments of asphalt or concrete debris removed.

Parking Areas: Within the proposed parking area, we recommend that the upper 12 inches of fill also 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 5 to 17 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 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. This criterion has been a requirement for the evaluation of potential import sources to date. Detailed grading and foundation recommendations addressing this concern are presented in the following sections.

5.2 PLANS AND SPECIFICATIONS REVIEW

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

5.3 CONSTRUCTION OBSERVATION AND TESTING

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

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION, CLEARING AND PREPARATION

6.1.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements within the proposed development area. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Alternately, at the owner's option, as several feet of new fill is planned to be placed across the site and there will be a requirement for relatively deeper scarification and re-compaction of the upper foot across the site, any vegetation may be mowed and raked, leaving about an inch or less. Then the upper 12 inches of soil may be tilled, oversized pieces of concrete debris removed, thoroughly mixing the remaining organics into the soil, prior to compaction and subsequent fill placement.

6.1.2 Tree and Shrub Removal

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

6.1.3 Demolition of Existing Slabs, Foundations and Pavements

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

6.1.4 Abandonment of Existing Utilities

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

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

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

6.2 REWORK OF EXISTING UNDOCUMENTED FILLS

As discussed, up to 14 feet of undocumented fill was encountered during the investigations at the site. Due to the environmental impacts of the fills as presented in prior environmental documents prepared for the site, we anticipate that the existing fills will be left in place and mitigation measures employed, if necessary. Prior to placement of new site fills across the proposed parking lot and fairway portions of the site, the existing vegetation should be stripped and the upper 12 inches of existing fill should be scarified, oversized pieces of concrete debris removed, moisture conditioned and re-compacted as engineered fill as discussed in the "Compaction" section below. In the proposed building area, the upper 18 inches of fill should be over-excavated, the bottom scarified at least 6 inches, oversized pieces of concrete debris removed, then the exposed fill should be moisture conditioned and re-compacted as engineered fill.

Provided the existing fills meet the "Material for Fill" requirements below, the fills may be reused when backfilling the excavations. Based on review of the samples collected from our borings, it appears that the surficial fills may be reused. The project environmental consultant should provide direction regarding re-use of deeper fills containing significant amounts of debris and whether they are suitable for re-use as engineered fill. If these soils or other materials are encountered that do not meet the requirements, such as debris, wood, concrete, those materials should screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the "Compaction" section below.

Fills extending into planned pavement, flatwork, and driving range areas will likely be left in place. The upper 12 inches of fill below pavement subgrade should be re-worked and compacted as discussed in the "Compaction" section below. Due to the presence of deep fills throughout the site, 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 southwestern 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 over a few to several years.

If desired to reduce the potential and magnitude of differential settlement, the fill can be removed to a deeper depth and replaced as engineered fill. The project environmental consultant should provide guidance regarding re-use of on-site fills for this option, as needed.

6.3 TEMPORARY CUT AND FILL SLOPES

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

Excavations performed during site demolition should be sloped at 3:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Excavations extending more than 5 feet below building subgrade and excavations in pavement and flatwork areas should be slope at a 1½:1 inclination unless the OSHA soil classification indicates differently.

6.4 CONSTRUCTION DEWATERING

Ground water levels are expected to be as shallow as 5 feet below existing grades in low areas of the site and likely 7 to 10 feet deep in higher elevations of the site. Therefore, temporary dewatering may be necessary during construction. If required, design, selection of the equipment and dewatering method, and construction of temporary dewatering should be the responsibility of the contractor. Modifications to the dewatering system are often required in layered alluvial soils and should be anticipated by the contractor. The dewatering plan, including planned dewatering well filter pack materials, should be forwarded to our office for review prior to implementation.

The dewatering design should maintain ground water at least 2 feet below localized excavations such as deepened footings, elevator shafts, and utilities. If the dewatering system was to shut down for an extended period of time, destabilization and/or heave of the excavation bottom requiring over-excavation and stabilization, flooding and softening, and/or shoring failures could occur; therefore, we recommend that a backup power source be considered.

Depending on the ground water quality and previous environmental impacts to the site and surrounding area, settlement and storage tanks, particulate filtration, and environmental testing may be required prior to discharge, either into storm or sanitary, or trucked to an off-site facility.

6.5 SUBGRADE PREPARATION

After site clearing and demolition is complete, and prior to backfilling any excavations resulting from fill removal or demolition, the excavation subgrade and subgrade within areas to receive additional site fills, slabs-on-grade and/or pavements should be scarified to a depth of 12 inches, moisture conditioned, and compacted in accordance with the "Compaction" section below using equipment capable of compacting a 12-inch lift. Subgrade areas that will have already received new fill for the proposed grades may be scarified to 6 inches.

6.6 SUBGRADE STABILIZATION MEASURES

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

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

6.6.1 Scarification and Drying

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

6.6.2 Removal and Replacement

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

6.6.3 Chemical Treatment

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

6.7 MATERIAL FOR FILL

6.7.1 Re-Use of On-site Soils

On-site soils with an organic content less than 3 percent by weight and not containing significant debris may be reused as general fill during demolition and initial site preparation. 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.

6.7.2 Re-Use of On-Site Site Improvements

We anticipate that varying quantities of asphalt concrete (AC) grindings and aggregate base (AB) will be generated during site demolition. If the AC grindings are mixed with the underlying AB to meet Class 2 AB specifications, they may be reused within the new pavement and flatwork structural sections, including within below-grade parking garage slab-on-grade areas (provided crushed rock is not required due to the proximity to ground water). AC/AB grindings may not be reused within the habitable building areas. Laboratory testing will be required to confirm the grindings meet project specifications.

6.7.3 Potential Import Sources

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

6.8 COMPACTION REQUIREMENTS

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

Table 2: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Expansive Soils	87 – 92	>3
	Low Expansion Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Expansive Soils	93	>3
	Low Expansion Soils	93	>1
Trench Backfill	On-Site Expansive Soils	87 – 92	>3
Trench Backfill	Low Expansion Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Low Expansion Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum
Flatwork Subgrade	On-Site Expansive Soils	87 - 92	>3
Flatwork Subgrade	Low Expansion Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Expansive Soils	87 - 92	>3
Pavement Subgrade	Low Expansion Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	NA

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

6.8.1 Construction Moisture Conditioning

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

6.9 TRENCH BACKFILL

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

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

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

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

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

6.9.1 Flexible Utility Connections

We anticipate up to about 1¾ inches of seismic settlement after a design-level earthquake. Flexible utility connections are recommended for critical utilities such as the water and gas lines and electrical trenches that will be connected to the proposed buildings supported on ground improvement or deep foundations. Depending on the site mitigation measures chosen, we can provide additional recommendations, as needed.

6.10 SITE DRAINAGE

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

6.11 LOW-IMPACT DEVELOPMENT (LID) IMPROVEMENTS

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

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

- The near-surface soils at the site are clayey, and categorized as Hydrologic Soil Group D, and are expected to have infiltration rates of less than 0.2 inches per hour. In our opinion, these clayey soils will significantly limit the infiltration of stormwater.
- Locally, seasonal high ground water is mapped at a depth of about 5 feet, and therefore is expected to be within 10 feet of the base of the infiltration measure.
- Environmental characterization of the on-site fills are not known at this time and should be considered as a potential for mobilization could result from infiltration.
- In our opinion, infiltration locations within 10 feet of the buildings would create a geotechnical hazard.

6.11.1 Storm Water Treatment Design Considerations

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

6.11.1.1 General Bioswale Design Guidelines

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

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

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

6.11.1.2 Bioswale Infiltration Material

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

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

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

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

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

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

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

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

6.11.1.3 Bioswale Construction Adjacent to Pavements

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

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

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

6.12 LANDSCAPE CONSIDERATIONS

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

- Using drip irrigation

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

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

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

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

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

In our opinion, 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 and subsequent sections below are followed, the proposed structures may be supported on shallow foundations. We recommend a design-build ground improvement contractor design the mitigation using an appropriate ground improvement technique to meet the project requirements. We also evaluated supporting the proposed Topgolf building on deep foundations, such as augercast piles; however, downdrag forces on the piles due to liquefiable layers as deep as 20 feet 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.

7.2 SEISMIC DESIGN CRITERIA

We understand that the project structural design will be based on the 2013 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The "Seismic Coefficients" used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Based on our 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 3: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.42335°
Site Longitude	-121.96975°
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 of D in Table 3 of this report, and the presented seismic coefficients, appear valid due to the above exception, as the structure likely has 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 will be triggered, and additional geotechnical analysis will need to be performed.

7.3 SHALLOW FOUNDATIONS

Provided ground improvement is performed in accordance with recommendations in this report, the structures may be supported on spread footings, which should bear on the ground improvement elements discussed in the later sections, be at least 24 inches wide, and extend at least 18 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 range from approximately 2 to 3½ and 1½ to 2½ inches, respectively. Ground improvement should be designed to reduce total settlement due to static and seismic conditions to tolerable levels as described below.

7.3.1 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 0.5 can likely be applied to the footing dead load underlain by ground improvement elements, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

7.3.2 Spread Footing Construction Considerations

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

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

7.4 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 will not be reworked due to environmental considerations, 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, because the site soils may be 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. It should be noted by potential ground improvement contractors that debris buried in the existing undocumented fill, such as concrete or asphalt fragments, may reduce production rates or require pre-drilling to depths on the order of 7 to 12 feet prior to installing DDCs.

7.4.2 General

The intent of the ground improvement design beneath the proposed building would be to increase the density of the potentially liquefiable sands generally at the depths of about 15 to 20 feet and 35 to 50 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 soft to medium stiff clays present between depths of about 15 to 30 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.

We recommend that the ground improvement contractor's design 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-driling 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 LATERAL SPREADING MITIGATION

In our opinion, the methods used for ground improvement could also be used to create a shear key to mitigate lateral spreading adjacent to the Guadalupe River channel. Using the same ground improvement method used in the building area, and therefore mobilizing one subcontractor to perform the installation, would likely be the most cost effective alternative. Typical shear key geometry would include 3 to 4 rows of tightly spaced ground improvement elements at no greater than 6 feet on-center, extending to a depth at least 5 feet below the deepest liquefiable layers within a zone equal to two times the depth of the adjacent Guadalupe River channel. We estimate this depth to be approximately 25 feet. The rows should extend laterally to at least a plane extending at 45 degrees from the corners of the proposed structure at the closest side. The rows could also be extended to protect any other critical improvements if desired.

Once final development and mitigation methods are known, we should be retained to perform supplemental stability analysis and design a shear key structure for the site to mitigate lateral spreading.

7.5.1 At-Grade Improvements

At-grade improvements located between the shear key and the river channel or areas such as the driving range (not protected by a shear key) may experience some lateral displacement during a seismic event. Provided some added long-term maintenance for at-grade improvements is acceptable, pathways, the turf driving range/outfield, and pavements can be located in those areas. This type of movement can result in cracks paralleling the river channel or significant displacement. Added maintenance may include crack sealing, grinding and resurfacing, or complete reconstruction.

7.6 DEEP FOUNDATIONS

7.6.1 Augercast Piles

As an alternative to using ground improvement with shallow foundations, drilled, cast-in-place, conventional or partial-displacement augercast piles (ACIP piles) may be used to support the proposed building. 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 do not appear to be feasible. It should be noted by potential foundation contractors that debris buried in the existing undocumented fill, such as concrete or asphalt fragments, may reduce production rates or require pre-drilling to depths on the order of 7 to 12 feet prior to installing augercast piles.

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.

7.6.1.1 Vertical Capacity and Estimated Settlement

The proposed structural loads may be 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 presented the results in Figure 5. The blue line on Figure 5 presents the vertical capacity for piles installed into un-improved ground, which accounts for potential downdrag effects in the upper 20 feet of the soil profile. The red line presents the estimated vertical capacity for piles 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 4 feet below existing grade, which will be about 5 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. The allowable capacities may be increased by one-third for wind and seismic loads. 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.

7.6.1.2 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.

We utilized the computer program L-Pile to model the load-deflection (p-y) curves representing the soil conditions surrounding the pile, and estimate the ultimate lateral load capacity of the pile. The following table presents the probable response of the piles under short-term loading conditions; the structural engineer should apply an appropriate factor of safety on the shears and moments presented. A cracked (assumed 30 percent reduction) pile stiffness (EI) of 8.1×10^9 lb-in^2 has been assumed in our analysis for 16-inch diameter ACIP piles. We also assumed a concrete compressive strength of greater than 4,000 psi for the concrete modulus calculations. If the pile stiffness varies by less than 20 percent of our assumed stiffness, the lateral load parameters below may be interpolated by multiplying the values by the ratio of the different pile stiffness values. We should be retained to re-evaluate the lateral load capacity for piles with stiffnesses significantly different from what was assumed.

Table 4: Ultimate Lateral Load Capacity – 16-Inch Diameter ACIP Piles

Pile Type	Fixity Condition	Lateral Deflection (inches)	Maximum Shear (kips)	Maximum Moment (kip-feet)	Depth to Maximum Moment (feet)	Depth to Zero Moment (feet)
16-inch ACIP	Free-Head	0.25	6	21	6	20
		0.50	11	39	6	21
16-inch ACIP	Fixed-Head	0.25	16	63	0	22
		0.50	25	107	0	25

7.6.1.3 Passive Resistance against Pile Caps and Grade Beams

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 to evaluate appropriate allowable passive pressures that maintain strain compatibility between the piles and pile caps, if additional passive resistance is required.

7.6.1.4 Pre-Production Test Program

One field pile load test should be performed per 150 to 250 piles, and at least one test pile per building area, and at location(s) recommended by the geotechnical engineer. Static load tests include installing a test pile, which can either be in a production pile location or not, with four surrounding piles that serve as anchor piles to resist the jacking pressure. During test pile installation, the contractor should allow for monitoring of strains within 5 feet of the pile tip to determine the amount of loading due to skin friction and end bearing. A member of our staff should be present during test pile installation and testing.

7.6.1.5 Construction Considerations

The installation of all test and production piles should be observed on a full-time basis by a Cornerstone representative to confirm that the piles are constructed in accordance with our recommendations and project requirements. Since the piles will derive their capacity from skin friction, the production piles should be installed to the design tip elevation. The geotechnical project engineer should review the installation records for conformance. We may recommend additional testing of piles, or additional installations, if any pile installations vary from normal installation practices.

We recommend that ACIP pile contractors have at least 3 years of installation experience.

7.6.2 Drilled Piers for Driving Range Net Poles

The proposed driving range fence poles may be supported by drilled, cast-in-place, straight-shaft friction piers. The piers should have a minimum diameter of 18 inches and extend to a depth of at least 15 feet below existing site grade. Adjacent piers centers should be spaced at least three diameters apart, otherwise, a reduction for group effects may be required.

7.6.2.1 Vertical Capacity and Estimated Settlement

The vertical capacity of the piers may be designed based on an allowable skin friction of 100 psf for combined dead plus live loads based on a factor of safety of 2.0; dead loads should not exceed two-thirds of the allowable capacities. The allowable skin friction may be increased by one-third for wind and seismic loads. Frictional resistance to uplift loads may be developed along the pier shafts based on an ultimate frictional resistance of 80 percent of the downward capacities; the structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate uplift capacity.

Total settlement of individual piers or pier groups of four or less should not exceed ½-inch to mobilize static capacities and post-construction differential settlement over a horizontal distance of 30 feet should not exceed ¼-inch due to static loads.

7.6.1.2 Lateral Capacity

Lateral loads exerted on the structure may be resisted by a passive resistance based on an ultimate equivalent fluid pressure of 300 pcf acting against twice the projected area of piers below the pier cap or grade beam within pier groups of two or more and over two pier diameters for single piers, up to a maximum uniform pressure of 2,000 psf at depth. The upper 2 feet of soil should be neglected when determining lateral capacity due to the existing fills. The structural engineer should apply an appropriate factor of safety to the ultimate passive pressures.

7.4.1.4 Construction Considerations

The excavation of all drilled shafts should be observed by a Cornerstone representative to confirm the soil profile, verify that the piers extend the minimum depth into suitable materials and that the piers are constructed in accordance with our recommendations and project requirements. The drilled shafts should be straight, dry, and relatively free of loose material before reinforcing steel is installed and concrete is placed. If ground water cannot be removed from the excavations prior to concrete placement, drilling slurry or casing may be required to stabilize the shaft and the concrete should be placed using a tremie pipe, keeping the tremie pipe below the surface of the concrete to avoid entrapment of water or drilling slurry in the concrete.

It should be noted by potential foundation contractors that debris buried in the existing undocumented fill, such as concrete or asphalt fragments, may reduce production rates or require pre-drilling to depths on the order of 7 to 12 feet prior to installing draft shafts. Due to the variable nature of some of the undocumented fills blanketing the site, the use of casing of each drilled shaft may be required.

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 INTERIOR SLABS-ON-GRADE

As the Plasticity Index (PI) of the surficial soils ranges up to 26, the proposed slabs-on-grade should be supported on at least 12 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. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

8.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

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

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

8.3 EXTERIOR FLATWORK

Exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading, including the golf target pads, should be at least 4 inches thick and supported on at least 6 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

SECTION 9: VEHICULAR PAVEMENTS

9.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a 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 several 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 5: Asphalt Concrete Pavement Recommendations, Design R-value = 10

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 TI 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.

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

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

9.2 PORTLAND CEMENT CONCRETE

The exterior Portland Cement Concrete (PCC) pavement recommendations tabulated below are based on methods presented in the Portland Cement Association (PCA) design manual (PCA, 1984). Recommendations for garage slabs-on-grade were provided in the “Concrete Slabs and Pedestrian Pavements” section above. We have provided a few pavement alternatives as an anticipated Average Daily Truck Traffic (ADTT) was not provided. An allowable ADTT should be chosen that is greater than what is expected for the development.

Table 6: PCC Pavement Recommendations, Design R-value = 10

Allowable ADTT	Minimum PCC Thickness (inches)
0.8	5.0
13	5.5
130	6.0

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

9.2.1 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 8 inches. The compressive strength, underlayment, and construction details should be consistent with the above recommendations for PCC pavements.

9.3 PAVEMENT CUTOFF

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

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

SECTION 10: RETAINING WALLS

10.1 STATIC LATERAL EARTH PRESSURES

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

Table 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.

10.2 SEISMIC LATERAL EARTH PRESSURES

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

10.3 WALL DRAINAGE

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

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

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

10.4 BACKFILL

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

10.5 FOUNDATIONS

Exterior site retaining walls may be supported on a continuous spread footing with a minimum width and depth of 15 inches and are capable of supporting maximum allowable bearing pressures of 2,000 psf for dead loads, 3,000 psf for combined dead plus live loads, and 4,000 psf for all loads including wind and seismic. These pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively.

SECTION 11: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Arco Murray specifically to support the design of the TopGolf San Jose 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 ground water conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

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

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

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

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

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

SECTION 12: REFERENCES

Boulanger, R.W. and Idriss, I.M., 2004, Evaluating the Potential for Liquefaction or Cyclic Failure of Silts and Clays, Department of Civil & Environmental Engineering, College of Engineering, University of California at Davis.

California Building Code, 2013, Structural Engineering Design Provisions, Vol. 2.

California Department of Conservation Division of Mines and Geology, 1998, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada, International Conference of Building Officials, February, 1998.

California Division of Mines and Geology (2008), "Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, September.

California Geological Survey, 2003, State of California Seismic Hazard Zones, Milpitas 7.5-Minute Quadrangle, California: Seismic Hazard Zone Report 051.

Idriss, I.M., and Boulanger, R.W., 2008, Soil Liquefaction During Earthquakes, Earthquake Engineering Research Institute, Oakland, CA, 237 p.

Rogers, T.H., and J.W. Williams, 1974 Potential Seismic Hazards in Santa Clara County, California, Special Report No. 107: California Division of Mines and Geology.

State of California Department of Transportation, 2009, Highway Design Manual, Sixth Edition, July 24, 2009.

USGS, 2013, Earthquake Ground Motion Parameters - A Computer Program for determining mapped ground motion parameters available at http://geohazards.usgs.gov/designmaps/us/application.php.

Working Group on California Earthquake Probabilities, 2007, The Uniform Earthquake Rupture Forecast, Version 2 (UCRF 2), U.S.G.S. Open File Report 2007-1437.

Yoshimine, M., Nishizaki, H., Amano, KI, and Hosono, Y., 2006, Flow Deformation of Liquefied Sand Under Constant Shear Load and Its Application to Analysis of Flow Slide in Infinite Slope, Soil Dynamics and Earthquake Eng. 26, 253-264.

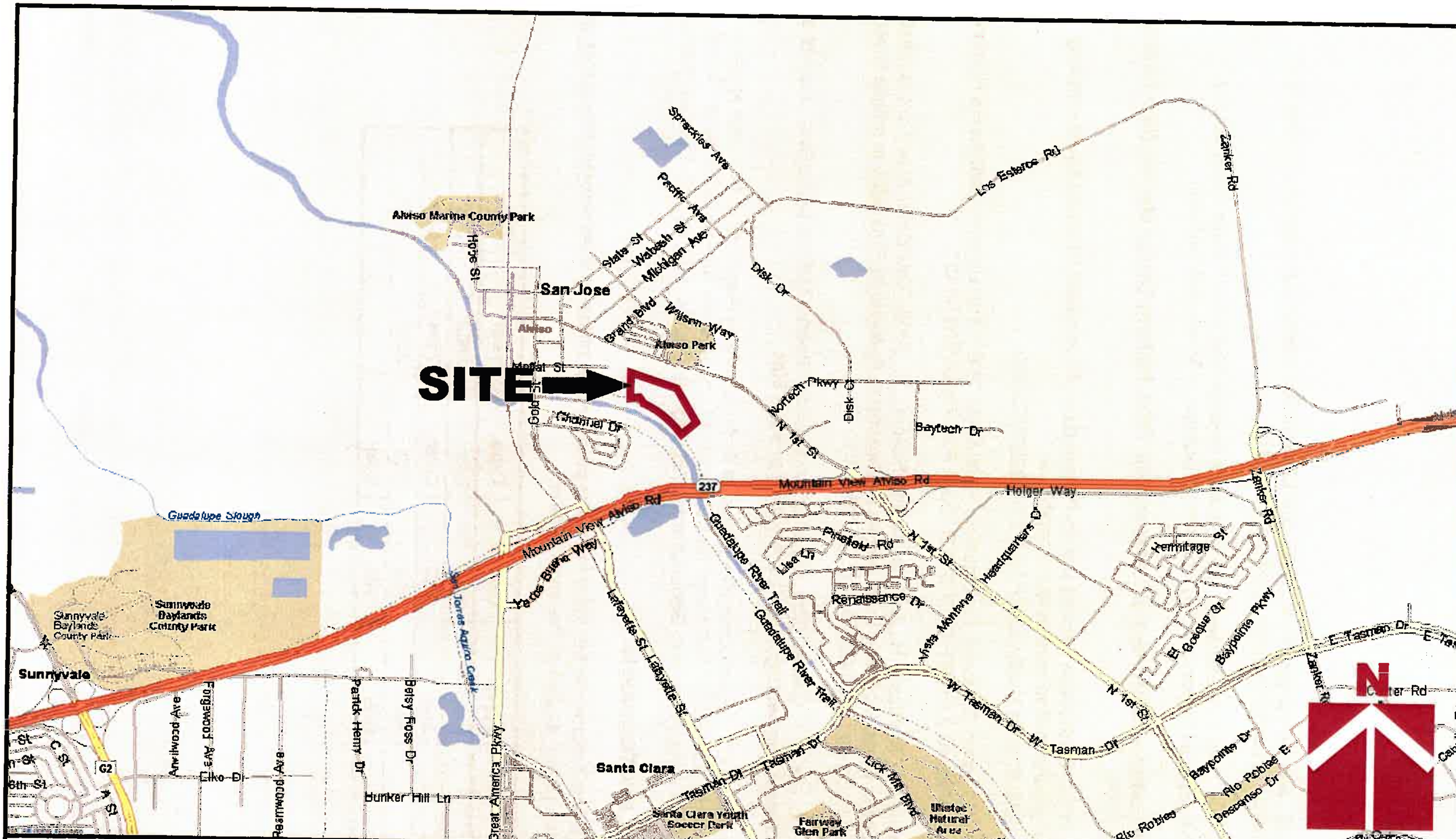
Youd, T.L. and C.T. Garris, 1995, Liquefaction-Induced Ground-Surface Disruption: Journal of Geotechnical Engineering, Vol. 121, No. 11, pp. 805 - 809.

Youd et al., 2001, "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vo. 127, No. 10, October, 2001.

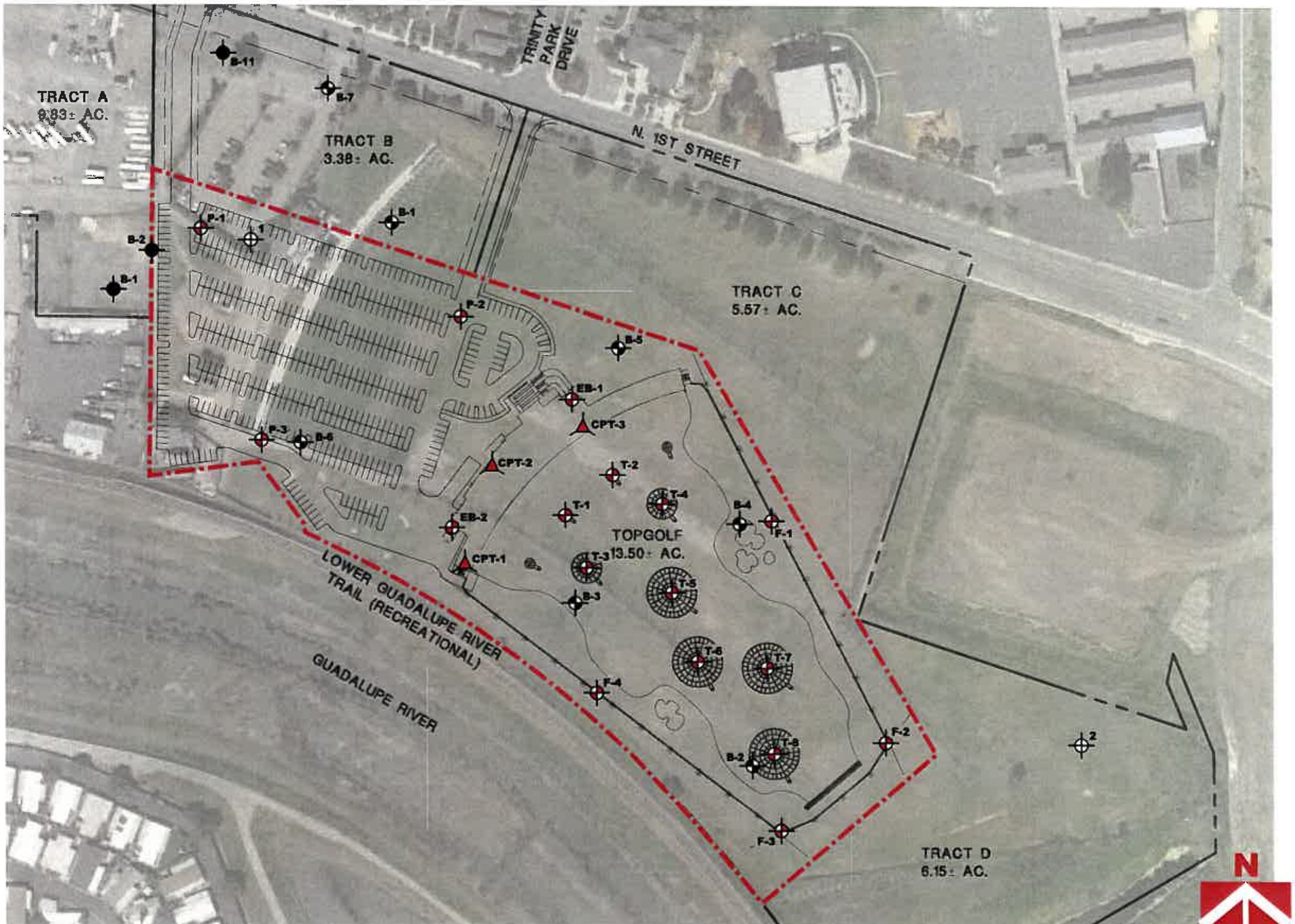
AERIAL PHOTOGRAPHS

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		



CORNERSTONE EARTH GROUP	Vicinity Map	Project Number 854-1-1	
	TopGolf San Jose, CA	Figure Number Figure 1	
		Date January 2016	Drawn By RRN



Legend

- Approximate location of previous boring (Terrasearch, 1987)
- Approximate location of previous exploratory boring (U.S.E., 1996)
- Approximate location of previous exploratory boring (U.S.E., 2001)
- Approximate location of boring
- Approximate location of cone penetration test

Base by Google Earth, dated 10/30/2015
Overlay by ARCO Murray, "Test Fit Plan #7", dated 8/4/2015

CORNERSTONE EARTH GROUP

Site Plan	
TopGolf San Jose, CA	
Project Number	854-1-1
Figure Number	Figure 2
Date	January 2016
Drawn By	RRN