APPENDIX C

DESIGN-LEVEL GEOTECHNICAL INVESTIGATION

CORNERSTONE EARTH GROUP

Type of Services	**Design-Level Geotechnical Investigation**
Project Name	**Winchester Assisted Living**
Location	**15860 Winchester Boulevard** **Los Gatos, California**
Client	**Swenson**
Client Address	**740 Front Street, Suite 315** **Santa Cruz, California**
Project Number	**100-66-1**
Date	**February 10, 2021**

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Project Name	**Winchester Assisted Living**
Location	**15860 Winchester Boulevard Los Gatos, California**

SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Swenson for the Winchester Assisted Living project in Los Gatos, California. The location of the site is shown on the Vicinity Map, Figure 1. We previously prepared preliminary geotechnical investigation letter reports for this site titled "Preliminary Geotechnical Investigation, Winchester Blvd and Shelburne Way Office Development" and "Preliminary Geotechnical Investigation, Winchester Blvd and Shelburne Way," dated September 21, 2015 and August 8, 2014, respectively. For our use, we were provided with the following documents:

- A set of architectural plans titled "Winchester Assisted Living, Winchester and Shelburne" prepared by Swenson, dated October 13, 2020.

1.1 PROJECT DESCRIPTION

The project site is located at 15860, 15880, and 15894 Winchester Boulevard and 17484 Shelburne Way in Los Gatos, California. We have reviewed conceptual plans and relatively recent aerial images of the site and visited the site. The site is currently occupied by three single-family residences consisting of at-grade, one-story structures, detached garages, and a storage shed. The 17484 Shelburne Way parcel is occupied by a paved parking area. During our field reconnaissance, in addition to the single-family residence and detached garage, we observed an approximately 10-foot by 10-foot concrete pad in the southeast corner of the 15894 Winchester Boulevard. The overall site is also occupied by several medium to large trees and various bushes and yards areas. We understand that a senior assisted living and memory care facility is currently planned for the site.

The planned development will include a 3-story building with one level of below-grade parking and is anticipated to be of wood- and/or steel-frame construction for the above-grade stories and of concrete construction for the below-grade parking level. As the site topography is sloped downward to the east, the below-grade parking level will daylight at-grade along the eastern side of the building with the entry from and exit to Shelburne Way. The footprint of the planned development will be approximately 29,600 square feet on the 0.32-acre site. Appurtenant parking, utilities, landscaping and other improvements necessary for site development are also planned.

The site is bounded by Winchester Boulevard to the west, Shelburne Way to the north, and residential development to the east and the south.

Structural loads are not currently known for the proposed structure; however, structural loads are anticipated to be typical of similar type structures. Grading is anticipated to include minor cuts/fills for site grading and cuts of up to 15 feet along Winchester Boulevard for construction of the below-grade parking level.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated October 15, 2020 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, temporary shoring, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of two borings drilled on January 8, 2021 with truck-mounted, hollow-stem auger drilling equipment. Additionally, during our preliminary investigation, we have previously drilled two borings on July 21, 2014 with a truck-mounted, hollow-stem auger drilling equipment. The borings were drilled to depths ranging from 30 to 45 feet. The borings were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

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

1.4 LABORATORY TESTING PROGRAM

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

1.5 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 Los Gatos range from about 0 to 50 feet (Rogers & Williams, 1974), see Figure 3, Vicinity Geologic Map.

2.2 REGIONAL SEISMICITY

While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated earlier estimates from their 2015 Uniform California Earthquake Rupture Forecast (Version 3) publication. The estimated probability of one or more magnitude 6.7 earthquakes (the size of the destructive 1994 Northridge earthquake) expected to occur somewhere in the San Francisco Bay Area has been revised (increased) to 72 percent for the period 2014 to 2043 (Aagaard et al., 2016). The faults in the region with the highest estimated probability of generating damaging earthquakes between 2014 and 2043 are the Hayward (33%), Calaveras (26%), and San Andreas Faults (22%). In this 30-year period, the probability of an earthquake of magnitude 6.7 or larger occurring is 22 percent along the San Andreas Fault and 33 percent for the Hayward Fault.

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)
Monte Vista-Shannon	0.8	1.3
San Andreas (1906)	4.0	6.4
Sargent	7.1	11.4
Zayante-Vergeles	10.2	16.4
Hayward (Southeast Extension)	12.0	19.3

A regional fault map is presented as Figure 4, illustrating the relative distances of the site to significant fault zones.

SECTION 3: SITE CONDITIONS

3.1 SITE HISTORY AND SURFACE DESCRIPTION

We reviewed aerial photographs available on Google Earth, dated 1993, 2000, and 2002 through 2014. We also reviewed historical topographic maps from 1928, 1939, 1942, 1943, 1955, 1960, 1961, 1964, 1966, 1973, 1975, 1978, 1980, and 1985 (www.historicaerials.com). Based on the topographic maps, the site was vacant in 1942 and development began in 1943 and continued throughout the 1950s. Based on the historical aerial photographs, the land was used for agricultural purposes in 1948 with evenly spaced orchard trees. Improvements were made at the site up to the 1980 aerial photograph (www.historicaerials.com).

The site is bounded by Winchester Boulevard to the west and Shelburne Way to the north, commercial development to the east, and apartment buildings to the south. The site is currently occupied by three single family residences with associated improvements and landscaping. The existing improvements include concrete and asphalt driveways, sidewalks, and fences. The site slopes at about 5 to 8 percent down toward the east, with grade differences on the order of about 10 to 14 feet across the site ranging from about Elevation 382 to 371 feet World Geodetic System 1984 (WSG 84) based on information contained in the previous reports and on the Google Earth website.

3.2 SUBSURFACE CONDITIONS

Below the vegetation, our borings EB-1 and EB-2 generally encountered interbedded layers of medium dense to very dense silty clayey sands and clayey sands, both with variable amounts of gravel, and stiff to hard lean clays with variable amounts of sand and gravel to the maximum depth explored of about 30 feet. Our boring EB-3 and EB-4 generally encountered hard sandy silty clays to depths ranging from 4½ to 7 feet (corresponding to Elevations 370½ to 373 feet), underlain by very dense sands varying from silty clayey sands to poorly graded sands with variable amounts of gravels, to a depth of 14 to 26½ feet below existing grade (corresponding to Elevations 353½ to 361 feet), and stiff to hard lean clays with variable amounts of sands and interbedded clayey sand with gravel to the maximum depth explored of approximately 45 feet. Based on our review of the provided plans, we understand that the excavation for the basement level will likely extend to Elevation 370 feet, and the material at the bottom of the excavation will be silty, clayey sand with gravel as shown in Cross Section A-A', Figure 5. It is noted that the soils in the Los Gatos area can contain significant amounts of oversize materials consisting of large gravels (1-3 inches in diameter), cobbles (3-8 inches), and some boulders (8 inches and greater).

3.2.1 Plasticity/Expansion Potential

We performed three Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of surficial and deeper soils. The results of the surficial PI tests indicated PIs ranging from 4 to 21 for the surficial and deeper soil, respectively, indicating low to moderate expansion potential to wetting and drying cycles for the surficial and deeper soil, respectively.

3.2.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 15 feet range from 5 percent below to the optimum estimated laboratory optimum moisture.

3.3 GROUNDWATER

Groundwater was encountered in one of our explorations (Boring EB-3) at a depth of 22 feet below current grades, corresponding with Elevation 353 feet (WSG 1984 datum). 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. The Seismic Hazard Zone Report (CGS, Los Gatos 7.5 Minute Quadrangle, 2002) indicates the historic high groundwater in the area to be approximately 10 feet below the ground surface. For design and planning purposes, we recommend a design groundwater depth of 12 feet below existing grade due to the potential for perched/nuisance water that may be encountered during excavation.

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

3.4 CORROSION SCREENING

We tested one sample collected from Boring EB-3 at depth of 3½ feet for resistivity, pH, soluble sulfates, and chlorides. The laboratory test results are summarized in Table 2A below.

Table 2A: Summary of Corrosion Test Results

Boring	Depth (feet)	Soil pH[1]	Resistivity[2] (ohm-cm)	Chloride[3] (mg/kg)	Sulfate[4,5] (mg/kg)
EB-3	3½	6.9	7,010	16	69

Notes: [1]ASTM G51
[2]ASTM G57 - 100% saturation
[3]ASTM D4327/Cal 422 Modified
[4]ASTM D4327/Cal 417 Modified
[5]1 mg/kg = 0.0001 % by dry weight

Many factors can affect the corrosion potential of soil including moisture content, resistivity, permeability, and pH, as well as chloride and sulfate concentration. Typically, soil resistivity, which is a measurement of how easily electrical current flows through a medium (soil and/or water), is the most influential factor. In addition to soil resistivity, chloride and sulfate ion concentrations, and pH also contribute in affecting corrosion potential.

3.4.1 Preliminary Soil Corrosion Screening

Based on the laboratory test results summarized in Table 2A, the soil is considered mildly corrosive to buried metallic for rebar and steel pipes (Chaker and Palmer, 1989); for rebar and steel pipes encased in concrete, the concrete cover protects the steel from corrosion.

In accordance with the 2019 CBC Section 1904.1, alternative cementitious materials for different exposure categories and classes shall be determined in accordance with ACI 318-19 Table 19.3.1.1, Table R19.3.1, and Table 19.3.2.1. Based on the laboratory sulfate test results, a cement type restriction is not required, although, in our opinion, it is generally a good idea to include some sulfate resistance and to maintain a relatively low water-cement ratio. We have summarized applicable exposure categories and classes from ACI 318-19, Table 19.3.1.1 below in Table 2B.

Table 2B: ACI 318-14 Table 19.3.1.1 Exposure Categories and Classes

Freezing and Thawing (F)	Sulfate (S, soil)	In Contact with Water (W)	Corrosion Protection of Reinforcement (C)
F0[1]	S0[2]	W0[3]	C0[4]

1 (F0) "Concrete not exposed to freezing-and-thawing cycles" (ACI 318-19)
2 (S0) "Water soluble sulfate in soil, percent by mass is less than 0.10" (ACI 318-19)
3 (W0) "Concrete dry in service" (ACI 318-19)
4 (C0) "Concrete dry or protected from moisture" (ACI 318-19)

We recommend the structural engineer and a corrosion engineer be retained to confirm the information provided and for additional recommendations, as required.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT SURFACE RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. According to the USGS online 2008 National Seismic Hazard Maps, the nearest active fault segment (Monte Vista-Shannon) is 1.3 kilometers away from the site. The site is not located within a State-designated Alquist Priolo Earthquake Fault Zone, or a Santa Clara County Fault Rupture Hazard Zone. The site is within the "B Fault" zone for the Monte Vista-Shannon (California Department of Conservations, Division of Mines and Geology, 1998) According to the Town of Los Gatos General Plan Update (Fault, Lineament & Coseismic Deformations Map, Plate #3, 1999) shows a "Lineation indicative of faulting" (interpreted from an aerial photograph analysis). The Santa Clara County Geologic Hazard Zones map shows the site is just outside of a fault surface rupture zone of a segment of the Monte Vista-Shannon fault, and according to the Fault Rupture Hazard Zones Map, the site is considered to have a low potential for fault rupture as it is in "areas outside recognized fault zones with no concentration of photo lineaments or evidence of widespread co-seismic deformation" (Town of Los Gatos General Plan Update, 1999). Based on the above and as shown in Figure 3, no known surface expression of fault traces is thought to cross the site; therefore, fault surface rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration (PGA) was estimated for analysis using a value equal to F_{PGA}*PGA, as allowed in the 2019 edition of the California Building Code. For our analysis we used a PGA of 1.264g.

4.3 LIQUEFACTION POTENTIAL

The site is not located within a State-designated Liquefaction Hazard Zone (CGS, Los Gatos 7.5 Minute Quadrangle, 2002) or a Santa Clara County Liquefaction Hazard Zone (Santa Clara County, 2003), and is located within a zone mapped as having very low to no liquefaction potential according to the Liquefaction Hazard Zones Map (Town of Los Gatos General Plan Update, Plate #5, 1999). However, we screened the site for liquefaction during our site exploration by retrieving samples from the site, performing visual classification on sampled materials, and performing various tests to further classify the soil properties.

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 3 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.

As discussed in the “Subsurface” section above, we primarily encountered stiff cohesive and very dense granular soils, and medium dense clayey sand encountered in EB-1 from about 17 to 21½ feet below ground surface is considered to have a low liquefaction potential as the Plasticity Index (PI) test performed on this material resulted in a PI of 21. Based on the above, our screening of the site for liquefaction indicates a low potential for liquefaction.

4.3.1 Ground Deformation and Surficial Cracking Potential

Since the site is not located within a currently mapped State-designated Liquefaction Hazard Zone and the previous borings indicate that the site is predominately underlain by clays, dense granular soils, or granular soils with enough fines that are not susceptible to liquefaction to a depth of at least 45 feet (the maximum depth explored), we do not anticipate any potential for ground deformation or surficial cracking as a result of liquefaction.

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.

While the Los Gatos Creek Trail is located approximately 900 feet away from the site, the site is considered to have a low potential for liquefaction potential and therefore, the potential for lateral spreading to affect the site is considered low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

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

4.6 LANDSLIDING

The site is located within a landslide hazard zone, according to the Santa Clara County Geologic Hazard Zones map. The site is not located within a CGS landslide hazard zone (CGS, Los Gatos 7.5 Minute Quadrangle, 2002), and is located in an area considered to have negligible potential for slope instability according to the Slope Stability Hazard Map (Town of Los Gatos General Plan Update, Plate #8, 1999). According to Google Earth, the site gradient is approximately 7 percent downward to the east (an elevation change of 14 feet over a horizontal distance of 200 feet). The surrounding areas have similar topography. Therefore, in our opinion, the potential for a landslide to affect the proposed improvements is considered low.

4.7 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone X, determined as “areas of 0.2 percent annual chance of flood; areas of 1 percent annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1 percent annual chance flood.” 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.

- Presence of gravels, cobbles, and boulders – below-grade improvements and garage level
- Presence of shallow groundwater – below-grade improvements and garage level
- Presence of cohesionless, sandy soils – below-grade improvements and garage level
- Differential movement at on-grade to on-structure transitions
- Construction considerations for existing improvements and residential areas

- Re-development considerations – all improvements

5.1.1 Presence of Gravels, Cobbles, and Boulders – Below-Grade Improvements and Garage Level

In the Los Gatos area, the soils can have up to about 60 percent Gravels (1-3 inches in diameter), Cobbles (3-8 inches), and Boulders (8 to 24 inches in size). Excavations into these materials should be anticipated and special handling may be required. As discussed in the "Subsurface" section and shown in Cross Section A-A', below the upper 4½ to 7 feet, dense to very dense silty, clayey sand and poorly graded sand with gravel was encountered to depths of 14 to 26 feet, corresponding with Elevations 361 to 353½ feet (WSG datum). Fines contents ranged from 5 to 20 percent. The contractor should consider the following issues during scheduling and evaluation of means and methods:

- Temporary Shoring:
  - Potential sloughing of excavation sidewalls – excavation and trimming of sidewalls may need to be done in limited sections that can be lagged during the same shift where layers of cleaner soils are encountered

- Foundation Excavations and Utility Trenches:
  - Trench sidewalls and excavations may not stand vertical when excavated, temporary shoring may be required
  - Contractor may not be able to cut excavations neat without the use of formwork
  - Footing bottoms will likely need to be proof compacted with vibratory equipment prior to placing reinforcing steel to address excavation disturbance

- Basement Subgrade Preparation:
  - Construction vehicle and foot traffic will likely disturb the basement subgrade during foundation and other construction activities that will occur prior to constructing the slab-on-grade. The subgrade will most likely need to be compacted just prior to placing the crushed rock, capillary break layer. The contractor may choose to delay subgrade compaction (typically performed prior to foundation construction) until just prior to slab-on-grade construction to reduce the need for rework.

5.1.2 Presence of Shallow Groundwater – Below-Grade Improvements and Garage Level

Groundwater was encountered at a depth of 22 feet in EB-3, Elevation 353 feet (WSG datum), and historic high groundwater has been mapped by CGS as about 10 feet below existing grade. Based on this information we recommend a design groundwater depth of 12 feet below existing grade be used. Our experience with sites in the vicinity indicates that shallow groundwater could significantly impact grading and underground construction, if encountered. These impacts typically consist of potentially wet and unstable basement subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering and shoring of utility trenches may be required in some isolated areas of the site, particularly where excavations extend deeper than about 12 feet below existing grade.

5.1.3 Cohesionless Sandy Soils – Below-Grade Improvements and Garage Level

As discussed, the site is underlain by silty sands to poorly graded sands to depths within the upper 15 feet (the anticipated maximum depth of the garage excavation). Due to the cohesionless (sandy) nature of the soils, the temporary excavations for at-grade improvements or construction of below-grade basement levels and foundations may be susceptible to localized sloughing and caving, especially in the upper 10 to 15 feet. Temporary vertical or near-vertical slopes will not likely be stable; therefore, excavations deeper than a few feet will require shoring. Temporary shoring using conventional soldier pile and wood lagging walls will require special design and construction consideration to prevent localized slough and caving during installation. General shoring guidelines are presented in the following sections of this report.

5.1.4 Differential Movement at On-grade to On-Structure Transitions

Some of the building area, specifically in the southwest corner, and other improvements will transition from on-grade support to overlying the basements. Where the depth of soil cover overlying the basement roof in the building area is thin or where basement walls extend to within inches of finished grade, these transition areas typically experience increased differential movement due to a variety of causes, including difficulty in achieving compaction of retaining wall backfill closest to the wall. We recommend construction and expansion joints be dowelled at this transition. If surface improvements are included that are highly sensitive to differential movement, additional measures may be necessary. We recommend consideration be given to where engineered fill is placed behind retaining walls extending to near finished grade, and that subslabs be included beneath flatwork or pavers that can cantilever at least 3 feet beyond the wall. We also recommend that retaining wall backfill be compacted to 95 percent where surface improvements are planned (see “Retaining Wall” section).

5.1.5 Construction Considerations for Existing Improvements and Residential Areas

The planned excavation associated with construction of the planned improvements could potentially cause displacements of the adjacent soils, utilities and ground surface. Displacement of adjacent soils could damage existing utilities and pavements. An effective shoring design and construction sequencing should be implemented to address these potential risks. The pipes should be relocated as determined by the Project Civil Engineer.

Additionally, the proposed improvements will be installed adjacent to and near single-family residences where there will be concerns about construction noise and vibrations. Additionally, construction activities should be performed in accordance with the Town’s construction ordinance requirements.

5.1.6 Re-Development Considerations – All Improvements

The site is currently occupied by three single family residences and associated improvements. Potential geotechnical issues that are often associated with redeveloping sites include demolition of existing improvements, abandonment of existing utilities, presence of undocumented fill, and presence of buried structures such as septic systems and dry wells. Detailed recommendations addressing these issues are discussed in the "Earthwork" section of this report.

5.2 PLANS AND SPECIFICATIONS REVIEW

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

5.3 CONSTRUCTION OBSERVATION AND TESTING

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

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION

All foundations, flatwork, pavements, utilities, and other improvements should be demolished and removed from the site. Recommendations in this section apply to the removal of these improvements, which may be present on the site, prior to the start of mass grading or the construction of new improvements for the project.

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

6.1.1 Demolition of Existing Slabs, Foundations and Pavements

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

6.1.2 Abandonment of Existing Utilities

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

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

The risk for owners associated with abandoning utilities in place include the potential for future differential settlement of existing trench fills, and/or partial collapse and potential ground loss into utility lines that are not completely filled with grout.

6.2 SITE CLEARING AND PREPARATION

6.2.1 Site Stripping

As discussed, the footprint of the new building will occupy most of the site; therefore, surface vegetation will likely be removed during excavation of the below-grade parking level. For areas outside the building footprint, the site should be stripped of all surface vegetation, and surface and subsurface improvements to be removed within the proposed development area. Demolition of existing improvements is discussed in the prior paragraphs. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Based on our site observations, surficial stripping should extend about 6 inches below existing grade in vegetated areas.

6.2.2 Tree and Shrub Removal

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

6.2.3 Presence of Gravels and Oversized Cobbles and Boulders

In the Los Gatos area, the soils can have up to about 60 percent Gravels, Cobbles, and Boulders (3 to 24 inches in size). Excavations into these materials should be anticipated and special handling may be required. As discussed in the “Subsurface” section and shown in Cross Section A-A’, dense to very dense silty, clayey sand and poorly graded sand with gravel was encountered to depths of 14 to 26 feet, corresponding with Elevation 361 to 353½ feet (WSG datum). Contractors should plan on forming footings, preparing slab-on-grade subgrade just prior to concrete placement, and other similar construction issues as relates to temporary shoring, utility excavation and granular material at the base of the basement excavation. These issues are further addressed within the “Foundations” section of this report.

6.3 REMOVAL OF EXISTING FILLS – AT-GRADE IMPROVEMENTS

While fills were not encountered in our borings and will likely be removed during excavation of the below-grade parking level, any fills encountered during site grading for planned at-grade improvements should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater.. Provided the fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations. Based on review of the samples collected from our borings, it appears that the fill may be reused. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should be screened out of the remaining material and be removed from the site. Additionally, gravels and oversized cobbles and boulders may be encountered during site grading and excavation and should be anticipated and planned for by the contractor. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below.

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

6.4 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. On a preliminary basis, the upper 15 feet at the site may be classified as OSHA Soil Type C materials. A Cornerstone representative should be retained to confirm the preliminary site classification. Recommended soil parameters for temporary shoring are provided in the “Temporary Shoring” section of this report.

Excavations performed during site demolition and fill removal should be sloped at 2:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Actual excavation inclinations should be reviewed in the field during construction, as needed. Excavations below building subgrade and excavations in pavement and flatwork areas should be sloped in accordance with OSHA soil classification requirements.

6.5 BELOW-GRADE EXCAVATIONS

Below-grade excavations may be constructed with temporary slopes in accordance with the “Temporary Cut and Fill Slopes” section above if space allows; however, based on the conceptual plans for the building, temporary shoring will be needed. Temporary shoring may support the anticipated cuts up to 15 feet. We have provided geotechnical parameters for shoring design in the section below. The choice of shoring method should be left to the contractor’s judgment based on experience, economic considerations and adjacent improvements such as utilities, pavements, and foundation loads. Temporary shoring should support adjacent improvements without distress and should be the contractor’s responsibility. A pre-condition survey including photographs and installation of monitoring points for existing site improvements should be included in the contractor’s scope. We should be provided the opportunity to review the geotechnical parameters of the shoring design prior to implementation; the project structural engineer should be consulted regarding support of adjacent structures.

6.5.1 Temporary Shoring

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

Table 3: Suggested Temporary Shoring Design Parameters

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

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

The restrained earth pressure may also be distributed as described in Figure 24 of the *FHWA Circular No. 4 – Ground Anchors and Anchored Systems* (with the hinge points at ¼H and ¾H) provided the total pressure is established from the uniform pressure above.

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

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

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

As previously mentioned, we recommend that a monitoring program be developed and implemented to evaluate the effects of the shoring on adjacent improvements. All sensitive improvements should be located and monitored for horizontal and vertical deflections and distress cracking based on a pre-construction survey. For multi-level excavations, the installation of inclinometers at critical areas may be desired for more detailed deflection monitoring. The monitoring frequency should be established and agree to by the project team prior to start of shoring construction.

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

6.5.2 Construction Dewatering

Dewatering and shoring of utility trenches may be required in some isolated areas of the site, particularly where excavations extend deeper than about 12 feet below existing grade, i.e.

elevator pits and deeper utility trenches. The dewatering system should be designed and implemented by the contractor. Depending on the groundwater quality and the extent of dewatering required, on-site retention, off-site disposal, or treatment prior to discharge, either into storm or sanitary sewer, may be required.

6.6 SUBGRADE PREPARATION

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

Due to the sandy soils likely to be encountered at the subgrade elevation, we recommend that subgrade compaction and proof rolling be performed within 24 hours of capillary break layer or slab-on-grade construction.

6.7 WET SOIL STABILIZATION GUIDELINES

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

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.

6.7.1 Scarification and Drying

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

6.7.2 Removal and Replacement

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

6.7.3 Below-Grade Excavation Stabilization

As the planned basement excavation may extend near the design groundwater level of 12 feet below existing grade, or encounter perched water, we recommend that the contractor plan to excavate an additional 12 to 18 inches below subgrade and backfill with clean, crushed rock. The crushed rock should be consolidated in place with light vibratory equipment. Rubber-tire equipment should not be allowed to operate on the exposed subgrade; the crushed rock should be stockpiled and pushed out over the stabilization fabric. As an alternative, the basement subgrade could possibly be over-excavated neat and covered with a minimum 3-inch-thick cement-sand slurry.

6.8 MATERIAL FOR FILL

6.8.1 Re-Use of On-site Soils

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

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

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

6.9 COMPACTION REQUIREMENTS

All fills, and subgrade areas where fill, slabs-on-grade, and pavements are planned, should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in the table below. In general, clayey soils should be compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory equipment; open-graded materials such as crushed rock should be placed in lifts no thicker than 18 inches 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 4: Compaction Requirements

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

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

6.10 TRENCH BACKFILL

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

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

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

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

6.11 SITE DRAINAGE

As the existing ground surface slopes to the east, surface runoff should not be allowed to flow over the top of or pond at the top or toe of engineered slopes or retaining walls. Ponding should also not be allowed on or 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. These facilities are not recommended where stormwater infiltration may affect slopes at lower elevations on or adjacent to the site. However, if slopes are not present at lower elevations that could potentially be affected, and 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.

Lined v-ditches should be included at the top of slopes and intermediate benches, and at the toe of slopes or behind retaining walls adjacent to planned or existing development. All v-ditches and drain inlets should be sized to accommodate the design storm events for the upslope tributary area. Concrete-lined v-ditches should be reinforced as required and have adequate control and construction joints and should be constructed neat in excavations; backfill around formed ditches should not be allowed.

Upslope sources of water should be evaluated. If upslope irrigation of is present or planned, additional surface and subsurface drainage, or construction of drained buttress fills may be needed to protect site improvements. We should be consulted if this issue will affect the project.

6.12 LOW-IMPACT DEVELOPMENT (LID) IMPROVEMENTS

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

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

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

- Locally, seasonal high groundwater is mapped at a depth of 10 feet, and therefore is expected to be at least 10 feet below the base of any infiltration measures.

6.12.1 Storm Water Treatment Design Considerations

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

6.12.1.1 General Bioswale Design Guidelines

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

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

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

6.12.1.2 Bioswale Infiltration Material

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

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

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

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

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

6.12.1.3 Bioswale Construction Adjacent to Pavements

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

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

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

6.13 LANDSCAPE CONSIDERATIONS

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

- Using drip irrigation

- Avoiding open planting within 3 feet of the building perimeter or near the top of 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, the proposed structure may be supported on conventional shallow foundations provided the recommendations in the "Earthwork" section and the sections below are followed.

7.2 SEISMIC DESIGN CRITERIA

We understand that the project structural design will be based on the 2019 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 explorations and review of local geology, the site is underlain by deep alluvial soils with typical SPT "N" values greater than 50 blows per foot. Therefore, we have classified the site as Soil Classification C. The mapped spectral acceleration parameters S_S and S_1 were calculated using the web-based program *ATC Hazards by Locations,* located at http://hazards.atcouncil.org, 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 5: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	C
Site Latitude	37.238154°
Site Longitude	-121.976704°
0.2-second Period Mapped Spectral Acceleration[1], S_S	2.480g
1-second Period Mapped Spectral Acceleration[1], S_1	0.898g
Short-Period Site Coefficient – Fa	1.2
Long-Period Site Coefficient – Fv	1.4
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	2.976g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.257g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.984g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.838g

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

7.3 SHALLOW FOUNDATIONS

7.3.1 Conventional Shallow Footings

Conventional shallow footings should bear on natural, undisturbed soil or engineered fill, be at least 18 inches wide, and extend at least 24 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil.

Footings constructed to the above dimensions and in accordance with the "Earthwork" recommendations of this report are capable of supporting maximum allowable bearing pressures of 3,000 psf for dead loads, 4,500 psf for combined dead plus live loads, and 6,000 psf for all loads including wind and seismic. These pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected for the portion of the footing extending below grade. Top and bottom mats of reinforcing steel should be included in continuous footings to help span irregularities and differential settlement.

7.3.2 Footing Settlement

Structural loads were not provided to us at the time this report was prepared; therefore, we assumed the typical loading in the following table.

Table 6: Assumed Structural Loading

Foundation Area	Range of Assumed Loads
Column Footing	200 to 400 kips
Perimeter Strip Footing	3 to 5 kips per lineal foot

Based on the above loading and the allowable bearing pressures presented above, we estimate that the total static footing settlement will be on the order of ½ to ¾-inch, with ¼ to ½-inch of post-construction differential settlement between adjacent foundation elements. As our footing loads were assumed, we recommend we be retained to review the final footing layout and loading, and to verify the settlement estimates above.

7.3.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of footing and the supporting subgrade, and also by passive pressures generated against footing sidewalls. An ultimate frictional resistance of 0.45 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 550 pcf may be used in design up to a maximum of 2000 psf. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

7.3.4 Spread Footing Construction Considerations

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

Footing excavations should be filled as soon as possible or be kept moist until concrete placement by regular sprinkling to prevent desiccation. It is noted that since the soils are primarily granular, some over-break may occur during the excavation and additional concrete may have to be placed. A Cornerstone representative should observe all footing excavations prior to placing reinforcing steel and concrete. If there is a significant schedule delay between our initial observation and concrete placement, we may need to re-observe the excavations.

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 INTERIOR SLABS-ON-GRADE

As the Plasticity Index (PI) of the surficial soils are 15 or less, the proposed slabs-on-grade may be supported directly on 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 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 near 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 PODIUM GARAGE SLABS-ON-GRADE

Garage slabs-on-grade should be at least 5 inches thick and if constructed with minimal reinforcement intended for shrinkage control only, should have a minimum compressive strength of 3,000 psi. If the slab will have heavier reinforcing because the slab will also serve as a structural diaphragm, the compressive strength may be reduced to 2,500 psi at the structural engineer’s discretion.

From our review of the project plans, we understand that a portion of the garage will be below grade. As discussed above, groundwater was encountered in one of our exploratory borings at a depth of 22 feet below the existing ground surface and historic high groundwater is mapped as 12 feet below existing grade. We recommend that a subdrain system be constructed behind the garage retaining walls and under the garage slab to drain any potential groundwater. Additional discussion and detailed design recommendations are presented in the “Retaining Wall” section below.

8.3 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 crushed rock should be placed below the vapor retarder and consolidated in place with vibratory equipment. The mineral aggregate shall be of such size that the percentage composition by dry weight as determined by laboratory sieves will conform to the following gradation:

Sieve Size	Percentage Passing Sieve
1"	100
¾"	90 – 100
No. 4	0 - 10

- 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.

8.4 EXTERIOR PEDESTRIAL FLATWORK

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

SECTION 9: VEHICULAR PAVEMENTS

9.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 15. The design R-value was chosen based on the results of the laboratory testing performed on a surficial sample collected from the proposed pavement area and engineering judgment considering the variable surface conditions.

Table 7: Asphalt Concrete Pavement Recommendations, Design R-value = 15

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	6.5	9.0
4.5	2.5	8.0	10.5
5.0	3.0	8.0	11.0
5.5	3.0	10.0	13.0
6.0	3.5	10.5	14.0
6.5	4.0	11.5	15.5

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

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

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 8: PCC Pavement Recommendations

Allowable ADTT	Minimum PCC Thickness (inches)
13	5½
130	6

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi, supporting the PCC on at least 4 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.

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

SECTION 10: RETAINING WALLS

10.1 STATIC LATERAL EARTH PRESSURES

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

Table 9: Recommended Lateral Earth Pressures

Sloping Backfill Inclination (horizontal:vertical)	Lateral Earth Pressure*	
	Unrestrained – Cantilever Wall	Restrained – Braced Wall
Level	45 pcf	45 pcf + 8H
3:1	55 pcf	55 pcf + 8H
2½:1	60 pcf	60 pcf + 8H
2:1	65 pcf	65 pcf + 8H
Additional Surcharge Loads	⅓ of vertical loads at top of wall	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

Basement walls should be designed as restrained walls. 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 2019 (CBC) states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. We reviewed seismic earth pressures for the proposed basement using procedures generally based on the Mononobe-Okabe method. Because the walls are likely approximately 11 feet in height or less, and peak ground accelerations are greater than 0.40g, we checked the result of the total seismic increment when added to the recommended active earth pressure against the recommended fixed (restrained) wall earth pressures. Because the wall is restrained, or will act as a restrained wall, and will be designed for 45 pcf (equivalent fluid pressure) plus a uniform earth pressure of 8H psf, based on current recommendations for seismic earth pressures, it appears that active earth pressures plus a seismic increment do not exceed the fixed wall earth pressures. Therefore, in our opinion, an additional seismic increment above the design earth pressures is not required as long as the walls are designed for the restrained wall earth pressures recommended above in accordance with the CBC.

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 exterior 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 as an alternative to 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 unless capped by hardscape. The drainage 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.

As discussed previously, consideration should be given to the transitions from on-grade to on-structure. Providing subslabs or other methods for reducing differential movement of flatwork or pavements across this transition should be included in the project design.

10.5 FOUNDATIONS

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

SECTION 11: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Swenson specifically to support the design of the Winchester Assisted Living project in Los Gatos, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

Recommendations in this report are based upon the soil and groundwater conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

Swenson may have provided Cornerstone with plans, reports and other documents prepared by others. Swenson 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

Aagaard, B.T., Blair, J.L., Boatwright, J., Garcia, S.H., Harris, R.A., Michael, A.J., Schwartz, D.P., and DiLeo, J.S., 2016, Earthquake outlook for the San Francisco Bay region 2014–2043 (ver. 1.1, August 2016): U.S. Geological Survey Fact Sheet 2016–3020, 6 p., http://dx.doi.org/10.3133/fs20163020.

"ATC Hazards by location". *Hazards.atcouncil.org*, 2019, https://hazards.atcouncil.org/#/

California Building Code, 2019, 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, 2002, State of California Seismic Hazard Zones, Los Gatos 7.5-Minute Quadrangle, California: Seismic Hazard Zone Report 069.

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

Ishihara, K., 1985, Stability of Natural Deposits During Earthquakes: Proceedings Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco.

Portland Cement Association, Thickness Design for Concrete Highway and Street Pavements: report, latest edition.

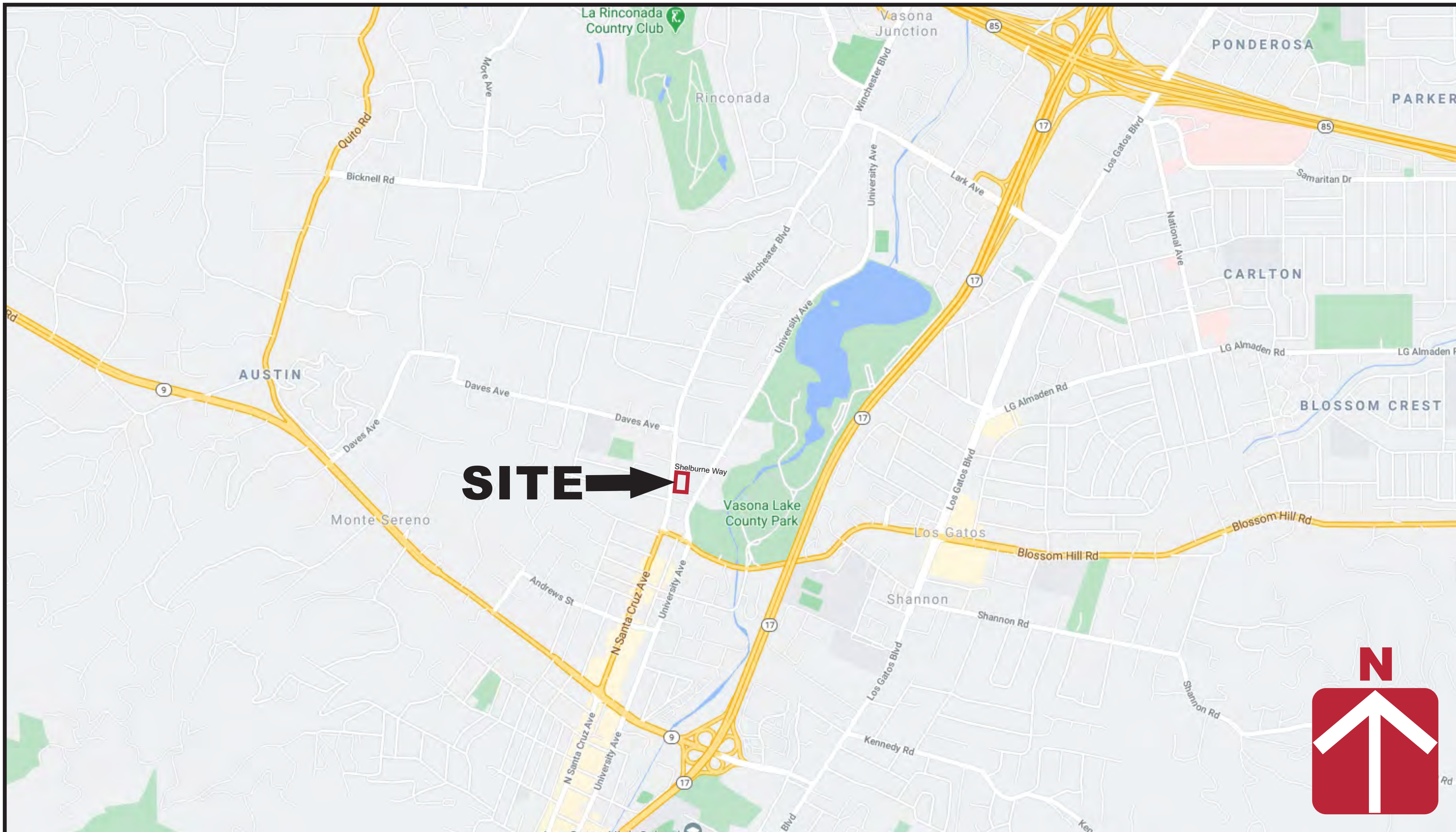
Rogers, T.H., and J.W. Williams, 1974 Potential Seismic Hazards in Santa Clara County, California, Special Report No. 107: California Division of Mines and Geology.

Seed, H.B. and I.M. Idriss, 1971, A Simplified Procedure for Evaluation soil Liquefaction Potential: JSMFC, ASCE, Vol. 97, No. SM 9, pp. 1249 – 1274.

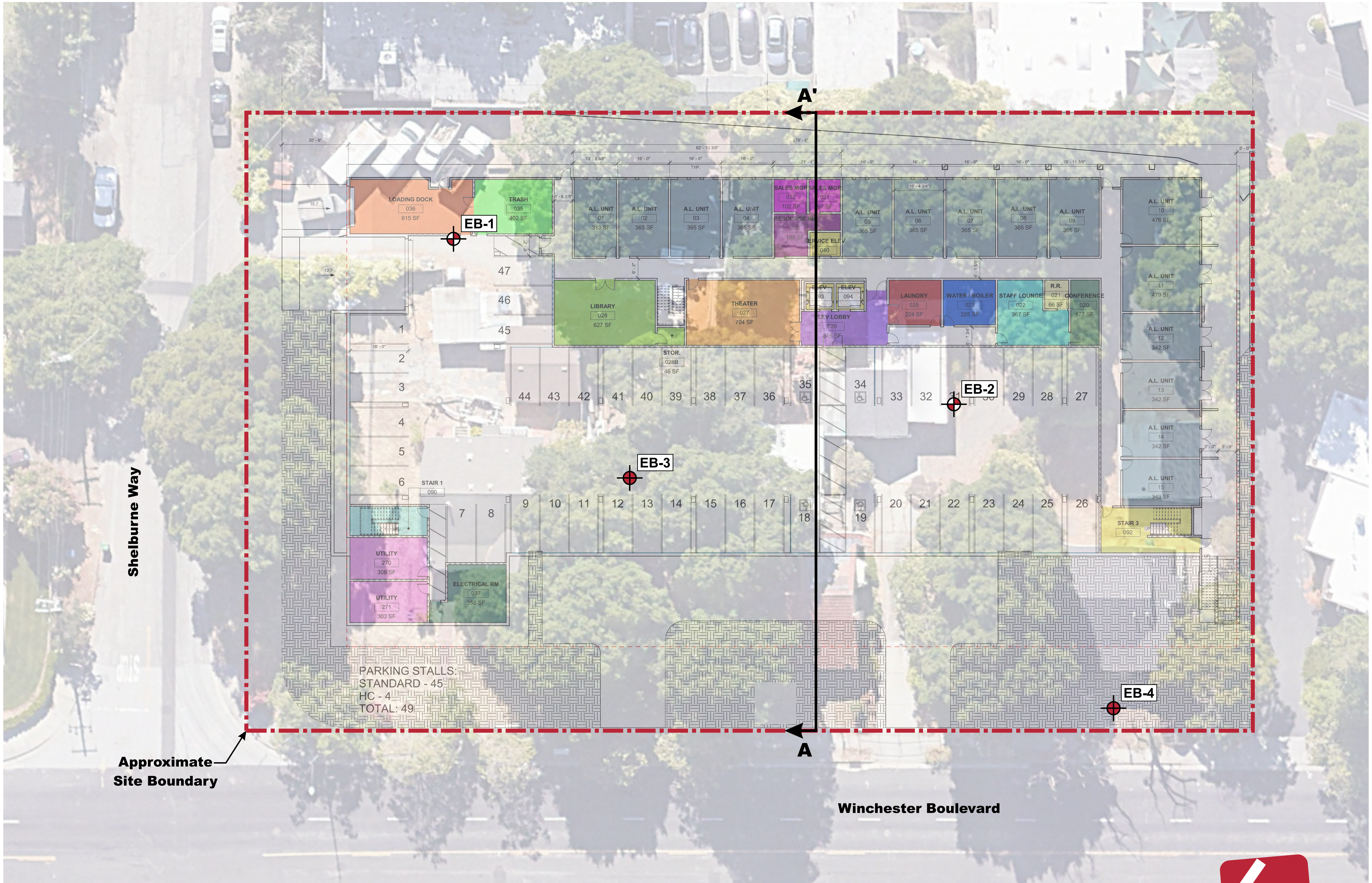
State of California Department of Transportation, 2020, Highway Design Manual, Seventh Edition.

Townley, S.D. and M.W. Allen, 1939, Descriptive Catalog of Earthquakes of the Pacific Coast of the United States, 1769 to 1928: Bulletin of the Seismological Society of America, Vol. 29, No. 1, pp. 1247-1255.

Working Group on California Earthquake Probabilities, 2015, The Third Uniform California Earthquake Rupture Forecast, Version 3 (UCERF), U.S. Geological Survey Open File Report 2013-1165 (CGS Special Report 228). KMZ files available at:www.scec.org/ucerf/images/ucerf3_timedep_30yr_probs.kmz



CORNERSTONE EARTH GROUP	**Vicinity Map**	Project Number 100-66-1
	Winchester Assisted Living Winchester Boulevard and Shelburne Way Development Los Altos, CA	Figure Number Figure 1
		Date January 2021 / Drawn By RRN



Legend

A A' Approximate location of cross section

Approximate location of exploratory boring (EB) (Cornerstone, this investigation)

Approximate location of exploratory boring (EB) (Cornerstone, 2014)

0 30 60
APPROXIMATE SCALE (FEET)

N

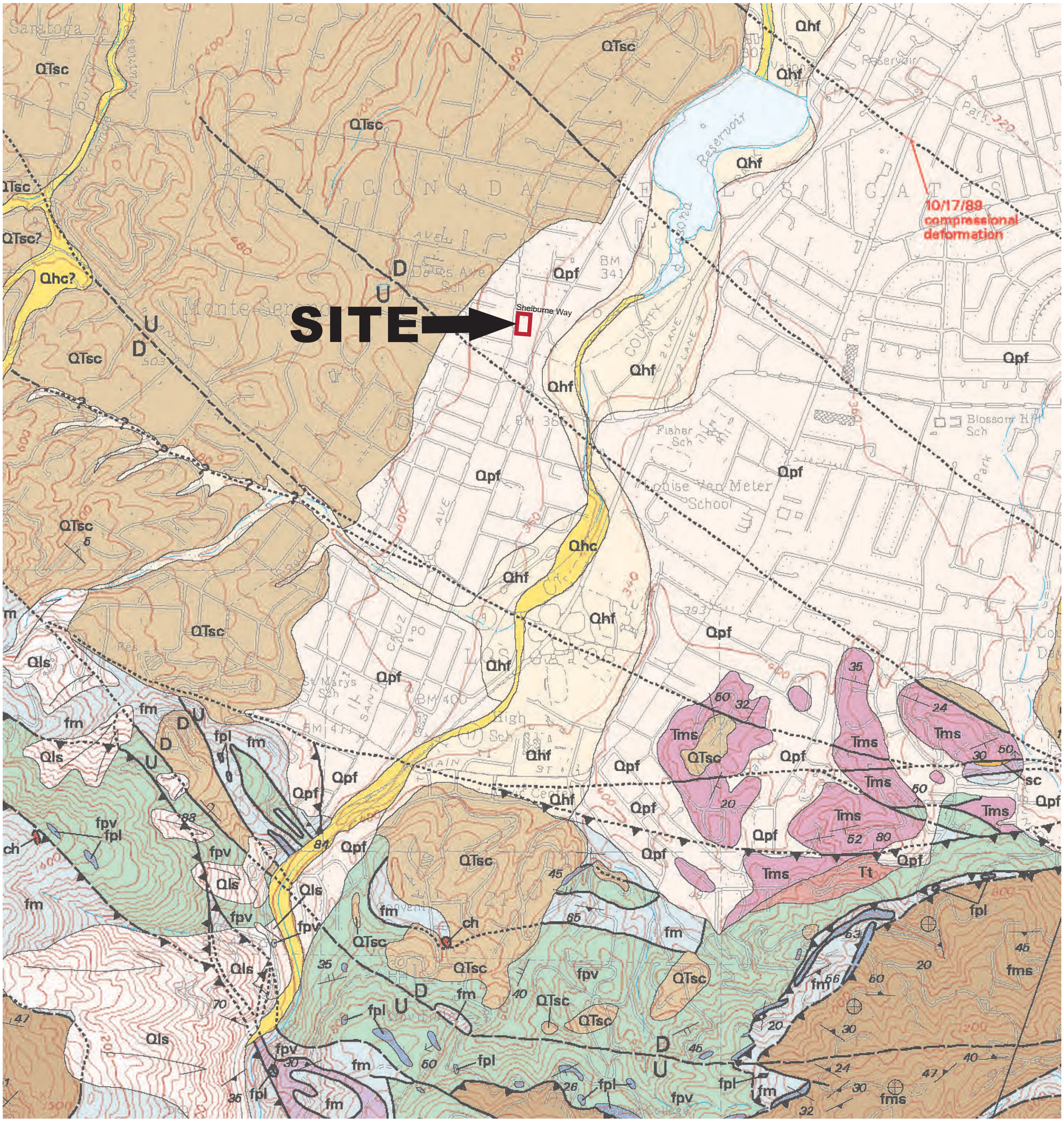
Base by Google Earth, dated 06/19/2019
Overlay by Swenson, Ground Floor - A-2.0, dated 10/13/2020

Site Plan

Winchester Assisted Living
Winchester Boulevard and
Shelburne Way Development
Los Altos, CA

CORNERSTONE EARTH GROUP

Project Number 100-66-1
Figure Number Figure 2
Date February 2021
Drawn By RRN



Geologic Units

Qhc Steam channel deposits (Holocene)

Qhf Alluvial fan deposits (Holocene)

Qpf Alluvial fan deposits (Pleistocene)

QTsc Santa Clara Formation (Pleistocene and Pliocene)

Explanation

Contact - dashed where approximate, dotted where concealed

N

0 2000 4000

APPROXIMATE SCALE (FEET)

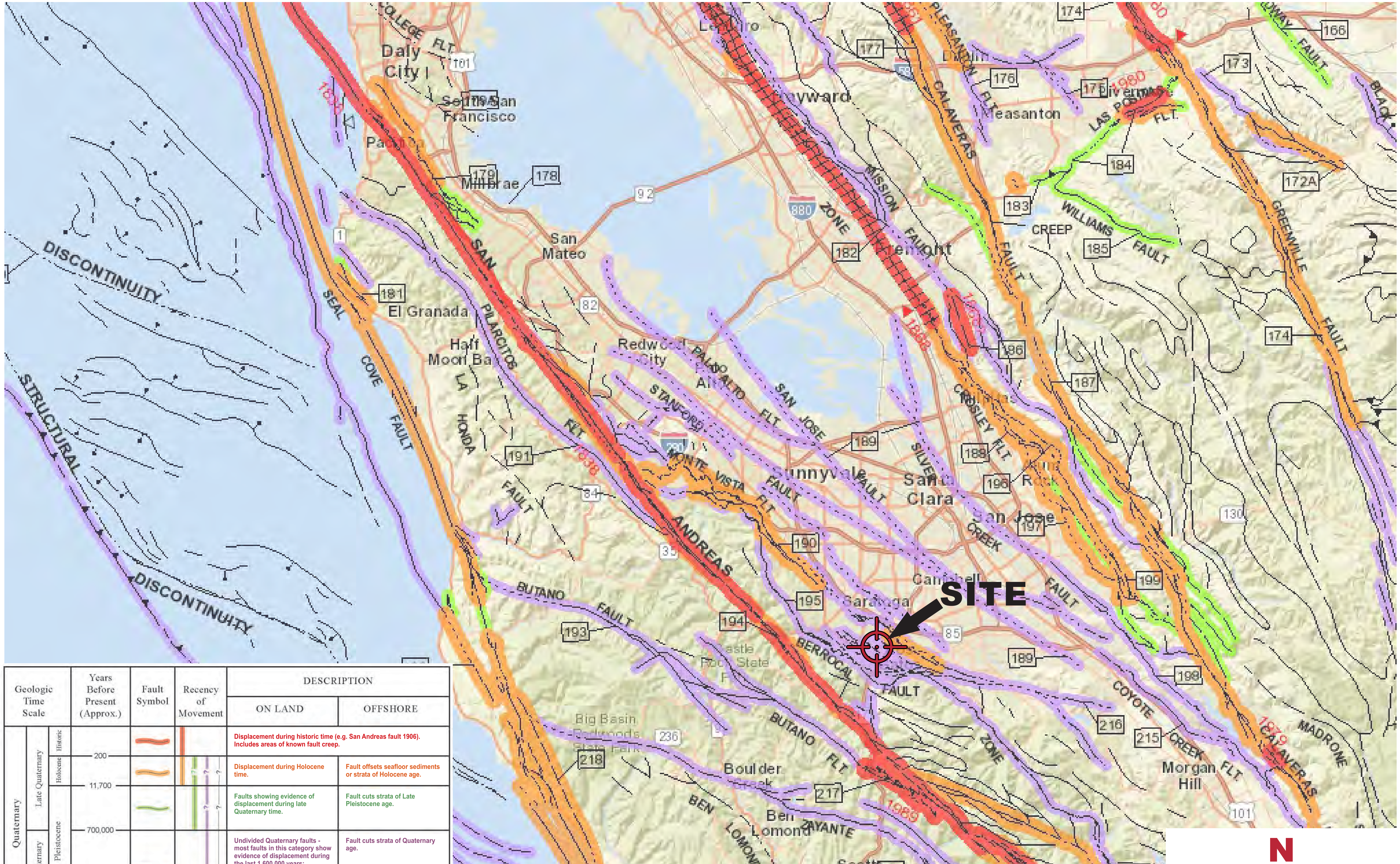
Base: USGS, Sheet 1: Los Gatos Quadrangle, by McLaughlin, Clark, Brabb, and Helly, Geologic Maps and Structure Sections of the Southwestern Santa Clara Valley and Southern Santa Cruz Mountains, Santa Clara and Santa Cruz Counties, California, by McLaughlin, Clark, Brabb, Helly, and Colón, 2001

CORNERSTONE EARTH GROUP

Vicinity Geologic Map

Winchester Assisted Living Winchester Boulevard and Shelburne Way Development Los Altos, CA

Project Number	100-66-1
Figure Number	Figure 3
Date	February 2021
Drawn By	RRN



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

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

N

0 5 10

APPROXIMATE SCALE (MILES)

Project Number 100-66-1

Figure Number Figure 4

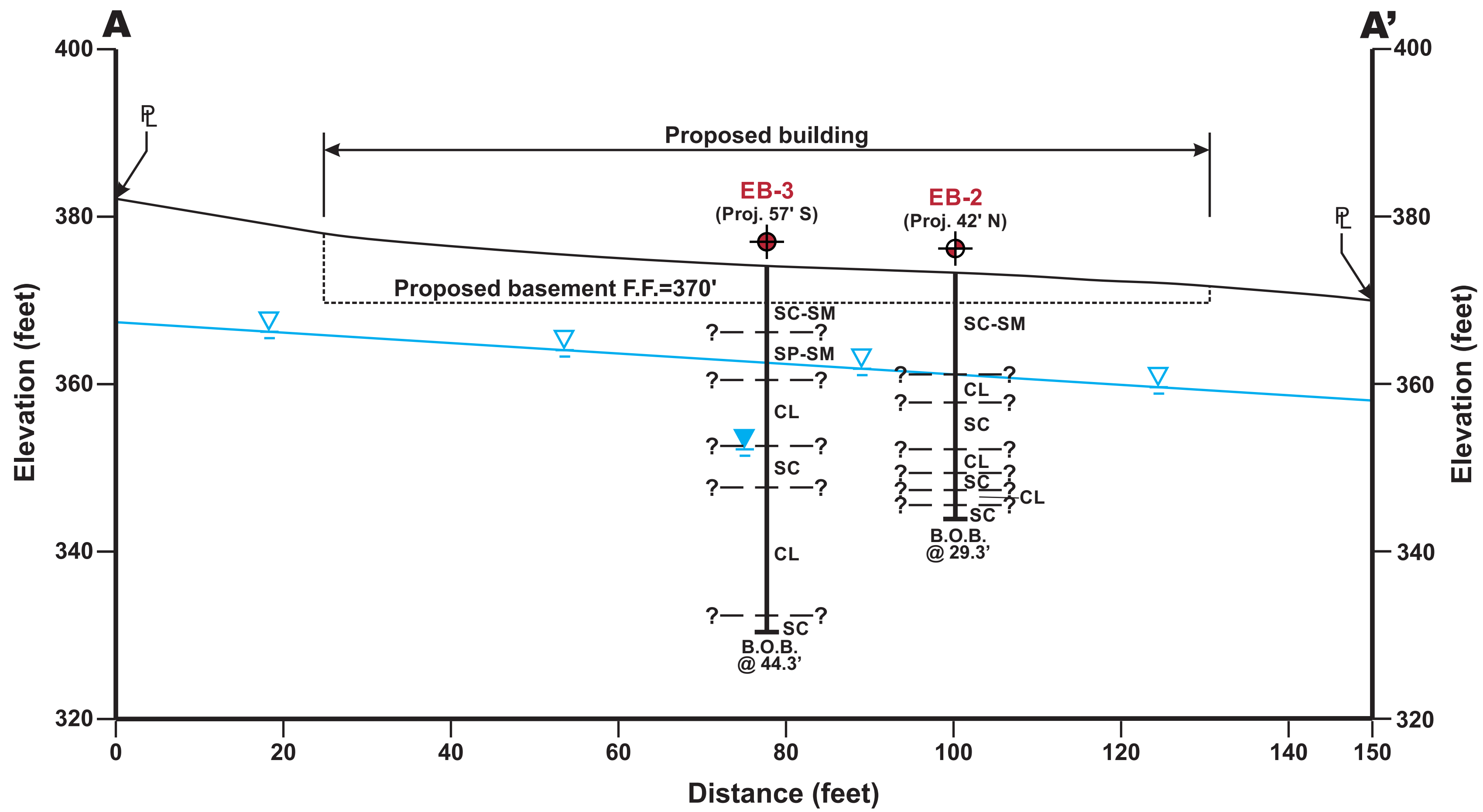
Drawn By RRN

Date January 2021

Regional Fault Map

Winchester Assisted Living Winchester Boulevard and Shelburne Way Development Los Altos, CA

CORNERSTONE EARTH GROUP



Section A-A'

(View Looking North)
1"=20' H:V

Symbols

CL	Lean Clay with Sand
SC	Clayey Sand with Gravel
SC-SM	Silty Clayey Sand with Gravel
SP-SM	Poorly-Graded Silty Sand with Gravel
▽	Approximate design groundwater depth
▼	Approximate encountered groundwater depth
⊕	Approximate location of exploratory boring (EB) (Cornerstone, this investigation)
⊕	Approximate location of exploratory boring (EB) (Cornerstone, 2014)

Notes:
1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
2) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
3) See Figure 2 for location of cross section.

Project Number 100-66-1
Figure Number Figure 5
Date February 2021
Drawn By RRN

Geologic Cross Section A-A'
Winchester Assisted Living
Winchester Boulevard and Shelburne Way Development
Los Altos, CA

CORNERSTONE EARTH GROUP

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, auger drilling equipment. Four 8-inch-diameter exploratory borings were drilled on January 8, 2021 and July 21, 2014 to depths of 30 to 45 feet. The approximate locations of exploratory borings are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

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

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

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

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

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	Cu>4 AND 1<Cc<3	GW	WELL-GRADED GRAVEL
			Cu>4 AND 1>Cc>3	GP	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	Cu>6 AND 1<Cc<3	SW	WELL-GRADED SAND
			Cu>6 AND 1>Cc>3	SP	POORLY-GRADED SAND
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY
			PI>4 AND PLOTS<"A" LINE	ML	SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY
			PI PLOTS <"A" LINE	MH	ELASTIC SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT

OTHER MATERIAL SYMBOLS

Poorly-Graded Sand with Clay	Sand
Clayey Sand	Silt
Sandy Silt	Well Graded Gravelly Sand
Artificial/Undocumented Fill	Gravelly Silt
Poorly-Graded Gravelly Sand	Asphalt
Topsoil	Boulders and Cobble
Well-Graded Gravel with Clay	
Well-Graded Gravel with Silt	

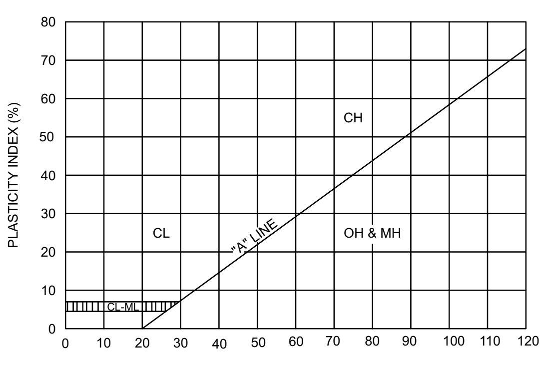
SAMPLER TYPES

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

ADDITIONAL TESTS

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

PLASTICITY CHART



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

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

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

CORNERSTONE EARTH GROUP	LEGEND TO SOIL DESCRIPTIONS	Figure Number A-1

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME Winchester Boulevard and Shelburne Way
PROJECT NUMBER 384-8-1
PROJECT LOCATION Los Gatos, CA

DATE STARTED 7/21/14 **DATE COMPLETED** 7/21/14
GROUND ELEVATION 375 FT +/- **BORING DEPTH** 29.8 ft.
DRILLING CONTRACTOR Exploration Geoservices,Inc.
LATITUDE 37.238207° **LONGITUDE** -121.976361°
DRILLING METHOD Mobile B-40, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY PKM
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
375.0	0	**Silty, Clayey Sand (SC)** medium dense, dry, brown, fine to coarse sand, some fine subangular to subrounded gravel	22	MC-1B	105	6			
372.5		**Silty, Clayey Sand with Gravel (SC-SM)** very dense, dry to moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	50/6"	MC					
	5		50/3"	MC-3A		8			
	10		71	SPT-4		6			
	15		91	SPT					
358.0		**Clayey Sand (SC)** medium dense, moist, brown, fine to coarse sand, some fine subangular to subrounded gravel Liquid Limit = 37, Plastic Limit = 16	22	SPT-6		15	21		
	20		13	SPT-7B		25			
353.5		**Clayey Sand with Gravel (SC)** very dense, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	52	SPT					
			27	SPT					>4.5 ○
350.5	25	**Sandy Lean Clay (CL)** hard, moist, brown, fine to coarse sand, some fine subangular to subrounded gravel, low to moderate plasticity							
			50/4"	MC-10B	112	18			>4.5 ○
345.2	30	Bottom of Boring at 29.8 feet.							

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME Winchester Boulevard and Shelburne Way

PROJECT NUMBER 384-8-1

PROJECT LOCATION Los Gatos, CA

DATE STARTED 7/21/14 **DATE COMPLETED** 7/21/14

GROUND ELEVATION 379 FT +/- **BORING DEPTH** 29.3 ft.

DRILLING CONTRACTOR Exploration Geoservices,Inc.

LATITUDE 37.237771° **LONGITUDE** -121.976618°

DRILLING METHOD Mobile B-40, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY PKM

AT TIME OF DRILLING Not Encountered

NOTES

AT END OF DRILLING Not Encountered

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

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
379.0	0	**Silty, Clayey Sand with Gravel (SC-SM)** very dense, dry to moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel Liquid Limit = 20, Plastic Limit = 16	50/3"	MC-1B	108	5	4		
			50/6"	MC					
	5		50/5"	SPT-3		5			
		medium dense	25	SPT-4		5			
	10	dense	44	SPT					
366.5		**Lean Clay with Sand (CL)** stiff, moist, brown, fine sand, moderate plasticity	12	SPT-6		24			○
363.5	15	**Clayey Sand with Gravel (SC)** dense to medium dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel	76	MC-7C	124	10			
			48	MC					
	20		44	SPT					○
358.0		**Lean Clay with Sand (CL)** stiff, moist, brown, fine to coarse sand, moderate plasticity							
355.0		**Clayey Sand with Gravel (SC)** dense, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	49	MC-10B	125	12			
353.0	25	**Sandy Lean Clay (CL)** hard, moist, brown, fine to coarse sand, low to moderate plasticity	33	SPT					>4.5 ○
351.0		**Clayey Sand with Gravel (SC)** very dense, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	50/4"	MC-12B	122	11			
349.7	30								

Bottom of Boring at 29.3 feet.

BORING NUMBER EB-3

CORNERSTONE EARTH GROUP

PROJECT NAME Winchester Assisted Living
PROJECT NUMBER 100-66-1
PROJECT LOCATION Los Gatos, CA

DATE STARTED 1/8/21 **DATE COMPLETED** 1/8/21
GROUND ELEVATION 375 ft. **BORING DEPTH** 44.3 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.238062° **LONGITUDE** -121.976653°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY JLC
▽ **AT TIME OF DRILLING** 28 ft.
NOTES
▼ **AT END OF DRILLING** 22 ft.

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

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
375.0	0	**Sandy Silty Clay (CL-ML)** hard, dry to moist, brown, fine sand, some fine subrounded gravel, low plasticity	50/6"	MC-1	98	6			>4.5 ○
			50/5"	MC-2B	111	5		50	>4.5 ○
370.5	5	**Silty, Clayey Sand with Gravel (SC-SM)** very dense, dry to moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	50/4"	MC-3		5			
367.0		**Poorly Graded Sand with Silt and Gravel (SP-SM)** very dense, dry to moist, light brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	50/4"	MC-4		2			
	10								
361.0		**Lean Clay with Sand (CL)** hard, moist, brown, fine sand, moderate plasticity	64	SPT-5		13			>4.5 ○
	15								
358.0		**Sandy Lean Clay (CL)** stiff, moist, brown, fine sand, low to moderate plasticity	32	SPT-6		21			○ (1.0)
	20								
353.5 ▼		**Clayey Sand with Gravel (SC)** very dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel	50/5"	MC-7	115	12			
	25								

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CORNERSTONE EARTH GROUP

PROJECT NAME Winchester Assisted Living

PROJECT NUMBER 100-66-1

PROJECT LOCATION Los Gatos, CA

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

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
348.0			**Lean Clay with Sand (CL)** very stiff to hard, moist, brown, fine to coarse sand, moderate plasticity							
				50/5"	SPT-8		16			○
	30									
				50/5"	SPT-9		8			○ >4.5
	35									
				50/4"	SPT-10		19			○
	40									
332.0			**Clayey Sand with Gravel (SC)** very dense, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel							
330.7				50/4"	SPT-11		11			
	45		Bottom of Boring at 44.3 feet.							
	50									
	55									

BORING NUMBER EB-4

CORNERSTONE EARTH GROUP

PROJECT NAME Winchester Assisted Living
PROJECT NUMBER 100-66-1
PROJECT LOCATION Los Gatos, CA

DATE STARTED 1/8/21 **DATE COMPLETED** 1/8/21
DRILLING CONTRACTOR Exploration Geoservices, Inc.
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
LOGGED BY JLC
NOTES

GROUND ELEVATION 380 ft. **BORING DEPTH** 44.8 ft.
LATITUDE 37.237673° **LONGITUDE** -121.976985°
GROUND WATER LEVELS:
AT TIME OF DRILLING Not Encountered
AT END OF DRILLING Not Encountered

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

Undrained shear strength, ksf: ○ Hand Penetrometer; △ Torvane; ● Unconfined Compression; ▲ Unconsolidated-Undrained Triaxial (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
380.0	0	**Sandy Silty Clay (CL-ML)** hard, dry to moist, brown, fine sand, some fine subrounded gravel, low plasticity Liquid Limit = 20, Plastic Limit = 14							
			82	MC-1B	113	6	6		>4.5 ○
			32	MC-2B	105	6			>4.5 ○
	5		34	MC-3B	110	6			>4.5 ○
373.0		**Silty, Clayey Sand with Gravel (SC-SM)** very dense, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel							
			50/6"	MC-4		7		22	
	10		50/6"	SPT-5		2			
			50/6"	SPT-6		6		25	
	15								
363.5		**Clayey Sand with Gravel (SC)** very dense, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel							
			50/1"	SPT-7		10			
	20								
			50/5"	SPT-8		14			
	25								

Continued Next Page

BORING NUMBER EB-4

CORNERSTONE EARTH GROUP

PROJECT NAME Winchester Assisted Living

PROJECT NUMBER 100-66-1

PROJECT LOCATION Los Gatos, CA

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
353.5		**Lean Clay with Sand (CL)** very stiff, moist, brown, fine to coarse sand, moderate plasticity							
			50/3"	SPT-9		22			○
	30								
349.5		**Clayey Sand with Gravel (SC)** very dense, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel							
			50/6"	SPT-10		15			
	35								
343.5		**Lean Clay with Sand (CL)** hard, moist, brown, fine to coarse sand, moderate plasticity							
			50/4"	SPT-11		19			○ >4.5
	40								
			50/4"	SPT-12		17			○ >4.5
335.2	45	Bottom of Boring at 44.8 feet.							
	50								
	55								

APPENDIX B: LABORATORY TEST PROGRAM

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

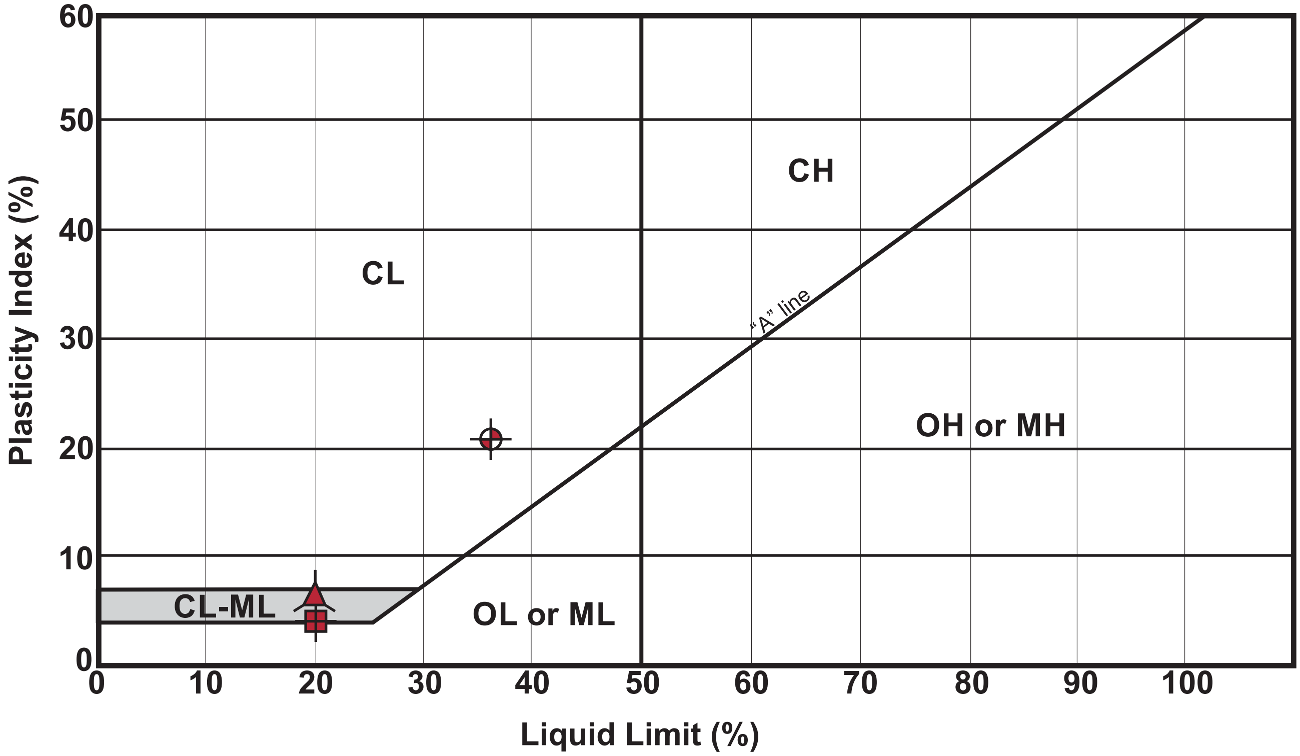
Moisture Content: The natural water content was determined (ASTM D2216) on 45 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

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

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

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

Plasticity Index (ASTM D4318) Testing Summary



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
	EB-1	18.5	15	37	16	21	—	Clayey Sand (SC) (CL fines)
	EB-2	2.0	5	20	16	4	—	Silty, Clayey Sand (SC-SM) (CL-ML fines)
	EB-4	2.0	6	20	14	6	—	Sandy Silty Clay (CL-ML)

Samples prepared in accordance with ASTM D421

CORNERSTONE EARTH GROUP

Plasticity Index Testing Summary

Winchester Boulevard and Shellburne Way Develoment Los Gatos, CA

Project Number	100-66-1
Figure Number	Figure B1
Date	January 2021
Drawn By	FLL

Corrosivity Tests Summary

CORNERSTONE EARTH GROUP

Job Number	100-66-1	Date Tested	1/19/2021
Job Name	Winchester & Shellburn	Tested By	FLL
Location	Los Gatos, CA		

Sample I.D.			Soil Visual Description	Moisture Content %	pH	Temp. at Testing C°	Resistivity (Ohm-cm) Corrected to 15.5 C°		Chloride Dry Wt. mg/kg	Sulfate Dry Wt. mg/kg
Boring	Sample No.	Depth, ft.					As Received	Saturated		
				ASTM D2216	ASTM G51		G57	ASTM G57	ASTM D4327	ASTM D4327
EB-3	2A	3.5	Brown Sandy Silty Clay (CL-ML)	6.0	6.9	22.0	-	7,010	16	69