Appendix GEO

Preliminary Geotechnical Investigation and Peer Review

CORNERSTONE
EARTH GROUP

TYPE OF SERVICES Preliminary Geotechnical Investigation

PROJECT NAME Hill Valley Oaks Apartments

LOCATION Arnold Drive
Martinez, California

CLIENT Hill Valley Oaks, LLC

PROJECT NUMBER 351-1-1

DATE November 17, 2009

Revised December 11, 2009

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Project Name	**Hill Valley Oaks Apartments**
Location	**Arnold Drive** **Martinez, California**
Client	**Hill Valley Oaks, LLC**
Client Address	**PO Box 1869** **Pleasanton, California**
Project Number	**351-1-1**
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SECTION 1: INTRODUCTION

This preliminary geotechnical report was prepared for the sole use of Hill Valley Oaks, LLC for the Hill Valley Oaks Apartments in Martinez, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- A set of preliminary architectural plans (Sheets A1 through A18) prepared by JGA Architects dated August 13, 2009.

1.1 PROJECT DESCRIPTION

The project site is located just south of the intersection of Arnold Drive and Starflower Avenue, in Martinez, California. The approximately 5.1-acre site (currently designated as APN Nos. 161-400-009 & 010) is undeveloped and covered with low grasses and numerous mature trees. We understand that an apartment complex is currently planned for the site that will include seven buildings.

The planned 121-unit development will be 3 to 4 stories with one to two levels of below-grade parking. A concrete podium will likely support wood-frame construction. A portion of the ground floor space for one of the buildings may be used for retail or restaurant space. Appurtenant parking, retaining walls, utilities, landscaping and other improvements necessary for site development are also planned.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated October 5, 2009, and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of four borings drilled on October 21, 2009, with truck-mounted hollow-stem auger drilling equipment. The borings were drilled to depths ranging from approximately 19 to 39 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 & Preliminary Geologic Map, 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 (ASTM D2216), dry densities (ASTM D2937), grain size analyses (ASTM D 422), washed sieve analyses (ASTM D1140), and Plasticity Index (ASTM D4318) tests. 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 in the western Diablo Range of the Coast Ranges structural and geomorphic province of California. This represents one mountain range in a series of northwesterly-aligned mountains forming the Coast Ranges geomorphic province of California that stretches from the Oregon border nearly to Point Conception. In the San Francisco Bay area, most of the Coast Ranges have developed on a basement of tectonically mixed Cretaceous- and Jurassic-age (70- to 200-million years old) rocks of the Franciscan Complex. Locally younger sedimentary and volcanic rocks cap these basement rocks. Still younger surficial deposits that reflect geologic conditions for the last million years or so cover most of the Coast Ranges.

The geology of this region is influenced by its setting within the active tectonic boundary between the Pacific and North American plates. The overall relative movement between these two plates is ideally represented by horizontal right slip of about 6 cm/yr on a vertical interface oriented to the northwest. Throughout coastal California, the surface expression of this interface is the San Andreas Fault, including its principal northwest-aligned branches. In the San Francisco Bay region the San Andreas Fault system includes several major branches, in addition to maintaining a relatively continuous main trace. The study site is near one such branch, the Concord-Green Valley Fault, crossing through the Walnut Creek area. The Hayward Fault, roughly 14 miles west of the site, is a well known, active feature exhibiting abundant geologic evidence of recurring movement and are the sources of both nearly continuous micro-seismicity and also of several large historic earthquakes.

In addition to the deformation and sporadic large earthquakes resulting from predominately right-lateral shear movements along major branches of the San Andreas Fault system, the Coast Ranges are also affected by tectonic compression acting normal to the tectonic boundary. This compression drives the uplift and much of the internal deformation within the fault system.

Graymer et al. (1994) identify bedrock of the site area as Muir sandstone of Weaver (1953), as shown on the Regional Geologic Map, Figure 3. This is described as non-marine sandstone, massive, yellow, weathering arkosic sandstone. This unit is thought to be Miocene or Pliocene in age. Their map shows bedding dipping roughly 60 to 75 degrees to the southwest in the hills north, south and east of the site. A splayed trace of a Concord-Green Valley fault covered by Quaternary alluvium is shown east of the site, crossing the Concord area. Numerous smaller unnamed, inactive faults cross the site vicinity, including one mapped near the east end of the site.

2.2 REGIONAL SEISMICITY

The San Francisco Bay area is one of the most seismically active areas in the Country. While seismologists cannot predict earthquake events, the U.S. Geological Survey’s Working Group on California Earthquake Probabilities 2007 estimates 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. As seen with damage in San Francisco and Oakland due to the 1989 Loma Prieta earthquake that was centered about 50 miles south of San Francisco, significant damage can occur at considerable distances. Higher levels of shaking and damage would be expected for earthquakes occurring at closer distances.

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 30 kilometers of the site.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
Concord-Green Valley	1.7	2.7
Greenville	8.0	12.9
Calaveras (north)	10.1	16.3
Hayward (Total Length)	14.2	22.8
West Napa	16.4	26.4

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 BACKGROUND

Based on our review of historic topographic maps and aerial photographs dating back to 1915 and 1939, respectively, the site has undergone several significant changes in the past 40 years. Prior to the 1970s, the site was primarily and undeveloped parcel located northwest of Pacheco and just north of the former Arnold Industrial Highway (present day Highway 4). Prior to the construction of Highway 4, a farm road reportedly named Barney Hill Lane was present near the current alignment of Arnold Drive as far back as 1915. Barney Hill Lane connected to former Highway 21 (present day Interstate 680 corridor), but was bisected when the 2-lane Highway 4 was constructed. A drainage channel and two rows of trees are visible on the site in the 1939 aerial photograph lining the farm road that crossed the east end of the site. The drainage channel in the 1939 photograph appears to be the same channel presently located at the site. The farm road appears to be partially paved in the 1958 aerial photograph.

The remainder of the site appears to have been covered with low grasses and a few small bushes or trees until the 1960’s. The 1965 photograph shows the trees lining the channel and farm road to be mature and form a denser canopy over the channel. Highway 4 was widened to a 4-lane freeway in the 1965 photograph from about the eastern half of the site towards the east.

Between 1965 and 1975, significant grading appears to have occurred along the Highway 4 corridor to further widen the freeway to four lanes adjacent to the entire site, and to re-align, resurface and extend the farm road into what is now present day Arnold Drive all the way to Pacheco Boulevard to the east. Significant cuts appear in the vicinity of the Highway 4 widening and surrounding hillsides as evidenced by the light-colored bedrock exposed during grading. The drainage channel is still visible in the 1975 photograph, but all mature trees have been removed. The north-facing slopes appear to be vegetated with low grasses. The remainder of the site appears to have been cut or filled with man-made fill, possibly from the adjacent Arnold Drive grading activities.

The 1982 photograph shows little change from the 1975 photograph, except that the existing storm drain pipe is visible just north of the drainage channel, which reportedly diverts all upstream water from the original channel. Several small trees and bushes are visible on the north-facing slope.

The 1993 photograph shows the existing sewer pump station has been constructed, as well as the office building and parking lot to the east. Residential development to the north was also observed. The drainage channel and storm drain pipeline are visible, and the trees on the slope are more mature. No other changes were observed in the 1995 and 2003 photographs, except for the increased size and extent of the slope vegetation.

3.2 SURFACE DESCRIPTION

The project site is located just south of the intersection of Arnold Drive and Starflower Avenue, in Martinez, California. The approximately 5.1-acre site (currently designated as APN Nos. 161-400-009 & 010) is undeveloped and covered with low grasses and numerous mature trees. The flat portions of the site have recently been tilled.

The site is bounded by Arnold Drive and existing residential development to the north, Highway 4 to the south, and existing commercial development to the east and west. In addition, an existing sanitary sewer pump station and parking lot is located adjacent to Arnold Drive, just east of the Starflower Avenue intersection. A sewer force main pipeline reportedly extends from the pump station along the Arnold Drive.

As discussed, an existing drainage channel crosses the eastern end of the site, as shown on Figure 2. Based on our discussions with you, a storm drain pipeline reportedly extends from the north end of the drainage channel, drains east parallel to Arnold Drive, and turns southeast along the eastern property line. The top of the reinforced concrete pipe (RCP) is partially exposed along the east end of the site.

Based on our review of available topographic data, site grades range from approximately Elevation 153 feet near the southwest corner of the site (top of ridge) to approximately Elevation 95 feet in the existing drainage channel (datum unknown). A ridgeline extends from the southwest corner of the site eastward along the southern property boundary; the ridgeline grades range from approximately Elevation 140 to 153 feet. The ridge slopes down to the north at an inclination ranging from approximately 3:1 to 4:1 (horizontal:vertical). The northern portion of the site slopes more gently downward towards Arnold Drive and the middle of the site (existing drainage channel) at roughly 3 to 6 percent.

As shown on Figure 2, the existing drainage channel starts near the north middle portion of the site and crosses towards the southeast corner of the site. The channel slopes are at an inclination of roughly 2:1 and the channel is about 3 to 6 feet deep.

3.3 SUBSURFACE CONDITIONS

As discussed, most of the site had recently been tilled to a depth of about 8 to 12 inches that exposed loose surficial soils. Borings EB-1 and EB-2 generally encountered approximately 8 to 10 feet of artificial (undocumented) fill consisting of soft to hard lean clay with varying amounts of sand and gravel and medium dense to dense clayey sand. The fill was underlain by native alluvial soils consisting of stiff to very stiff lean clay to a depth of approximately 12 to 22 feet. The alluvial clay was interbedded with thin layers of medium dense clayey sand. The alluvial clay was underlain by 4 to 6 feet of medium dense silty sand (possibly residual soil derived from the underlying sandstone bedrock), with fines contents ranging from approximately 26 to 40 percent.

In Borings EB-1 and EB-2, Muir Formation sandstone was encountered at a depth of 26 and 18 feet, respectively, beneath the alluvial soils. The sandstone was generally friable to weak, low hardness, moderately weathered, massive, with very little to moderate cementation. The sandstone extended to the maximum depth explored at a depth of 39 feet.

In Boring EB-3, our explorations encountered 2 feet of artificial fill consisting of very stiff lean clay with sand underlain by Muir sandstone interbedded with sandy siltstone to a depth of 19 feet. In Boring EB-4, medium dense silty sand was encountered to a depth of approximately 5 feet that was underlain by Muir sandstone to the exploration depth of 24 feet.

3.3.1 Plasticity/Expansion Potential

We performed one Plasticity Index (PI) test on representative near-surface soil sample. Test results were used to evaluate expansion potential of surficial soils. The results of the surficial PI tests indicated a PI of 5, indicating low expansion potential to wetting and drying cycles.

3.3.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 15 feet range from 0 to 10 percent over the estimated laboratory optimum moisture.

3.4 GROUND WATER

Ground water was encountered in Borings EB-1 and EB-2 at depths ranging from 10 to 13½ feet below current grades, corresponding to Elevations 88 to 89½ feet (datum unknown). Ground water was not encountered in Borings EB-3 or EB-4 during drilling. 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.

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

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

As discussed above several significant faults are located within 30 kilometers of the site. The site is not located within a State-designated Alquist Priolo Earthquake Fault Zone. As shown in Figure 4, no known surface expression of active fault traces is thought to cross the site; therefore, fault rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. The California Geologic Survey maintains a website based on the USGS/CGS Probabilistic Seismic Hazards Assessment (PSHA) Model, 2002 (revised April 2003). The pseudo-peak acceleration for the site with a 10 percent chance of exceedance in 50 years is approximately 0.57g.

4.3 LIQUEFACTION POTENTIAL

Contra Costa County is not currently included in the State-designated Liquefaction Hazard Zone mapping performed by the California Geologic Survey (Walnut Creek 7½-Minute Quadrangle). The Association of Bay Area Governments (ABAG) has mapped the site as being in an area of very low to low liquefaction potential. Our field and laboratory programs addressed this issue by sampling potentially liquefiable layers above the underlying bedrock formation, performing visual classification on sampled materials, and performing various tests to further classify the soil properties.

4.3.1 Background

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data is available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 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.

4.3.2 Analysis and Results

As discussed in the “Subsurface” section above, native alluvial sand layers were encountered in Borings EB-1 and EB-2 ranging from approximately 2 to 6 feet thick. These layers were encountered beneath the artificial fill area and below the design ground water depth of approximately 10 feet (corresponding to approximately Elevation 88 to 91 feet). Following the procedures in the 1998 NCEER Workshop Proceedings (Youd et. al., 2001) and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008), these layers were screened for liquefaction triggering and potential post-liquefaction settlement. These methods compare ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil’s estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.0 are considered to be potentially liquefiable.

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

The soil’s CRR is estimated from the in-situ density and strength obtained from field SPT blowcounts (“N” value) from the exploratory borings. The “N” values are corrected for effective overburden stresses, taking into consideration both the ground water level at the time of exploration and the design ground water level, and stress reduction versus depth factors. The "N" values are also corrected for fines content, hammer efficiency, boring diameter, rod length, and sampler type (with or without liners).

Soils with significant quantities of plastic fines (PI greater than 12) and soils with "N" values of 30 are typically considered too plastic or too dense/stiff to liquefy. These soil layers have been screened out during our analyses and are not presented below. The results of our preliminary SPT analyses are presented in the table below.

Table 2: Results of Liquefaction Analyses – SPT Method

Boring Number	Depth to Top of Layer (feet)	Layer Thickness (feet)	SPT ($N_{160,CS}$)	Factor of Safety	Potential for Liquefaction	Estimated Total Settlement (inches)
EB-1	17½	2	38	1.0	Likely	¼
EB-1	22	4	40	1.3	Low	0
EB-2	12	6	30	0.6	Likely	1½

4.3.3 Summary

Our analyses indicate that two of the sand layers encountered in Borings EB-1 and EB-2 could potentially experience liquefaction triggering that could result in soil softening and post-liquefaction total settlement ranging from approximately ¼ to 1½ inches based on the Ishihara and Yoshimine (1992) method. As discussed in the SCEC report, differential movement for level ground sites over deep soil sites will be about half of the total settlement. Since the alluvial soil thickness along the alignment of the former drainage channel appears to vary abruptly, in our opinion, differential settlement could be greater than half the estimated total settlement.

Based on our preliminary analysis, we estimate that differential settlement due to liquefaction beneath Building 5 could be on the order of 1 inch across a horizontal distance of 50 feet. Portions of Building 6 could experience differential settlement on the order of ¼ inch or more, depending on the lateral extent and variable thickness of alluvial soils in that area. We recommend that additional subsurface exploration, laboratory testing and engineering analysis be performed during the design-level geotechnical investigation to further evaluate the potential for liquefaction-induced settlement beneath buildings that will straddle the fill/alluvial soil area of the site.

4.3.4 Ground Rupture Potential

The methods used to estimate liquefaction settlement assume that there is a sufficient cap of non-liquefiable material to prevent ground rupture or sand boils. For ground rupture to occur, the pore water pressure within the liquefiable soil layer will need to be great enough to break through the overlying non-liquefiable layer, which could cause significant ground deformation and settlement. Cuts on the order of 2 to 13 feet are proposed for Buildings 4 through 6 and the podium garage, which will straddle the fill and alluvial soil area. The proposed cuts will remove some of the existing fill and native alluvial soil in the vicinity of Boring EB-2, leaving about only 6 feet of non-liquefiable fill and native soil above potential liquefiable material. For Building 6, cuts on the order of 2 to 5 feet are proposed in the fill and alluvial soil area. In the vicinity of Boring EB-1, approximately 14 feet of non-liquefiable material will remain over potential liquefiable materials.

The work of Youd and Garris (1995) indicates that the 6-foot thick layer of non-liquefiable cap in the area of Boring EB-2 may not be sufficient to prevent ground rupture; therefore, the above settlement estimates in the vicinity of EB-2 may be too low if cracks or fissures occur in the native soils immediately below the proposed improvements. Further discussion of potential impacts due to liquefaction is presented in the “Conclusions” section.

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.

A 3 to 6 foot deep drainage channel crosses the eastern half of the site. Based on the conceptual site plans, we understand that this channel will be filled during site development. Therefore, in our opinion, the potential for lateral spreading to affect the site is low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. As the soils encountered at the site above the ground water level 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 south end of the site is flanked by a 20 to 25 foot high slope that is inclined at approximately 3:1 to 4:1 (horizontal:vertical). Numerous mature trees and low bushes are present on the slope face; the remainder of the slope is covered with sparse low grasses. Based on our site observations, shallow bedrock was exposed near the top of the slope with the topsoil layer gradually increasing in thickness towards the bottom of the slope. The topsoil mantling the slope generally consists of loose silty sand. Bedrock within the ridgeline generally dips steeply towards the southwest, which based on our review of geologic maps for the region, is typical for this area.

Based on our site observations and review of available subsurface data, the potential for deep-seated slope instability is considered low due to the generally favorable bedrock orientation and moderate slope inclinations. The sandy topsoil mantling the slope, and the underlying weathered shallow bedrock, may be susceptible to shallow sloughing or erosion during periods of heavy rainfall. Further discussion of the potential impacts due to development adjacent to the existing slopes is presented in the “Conclusions” section of this report.

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, described as an area outside of the 0.2% of annual flood plain. We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

The Association of Bay Area Governments has compiled a database of Dam Failure Inundation Hazard Maps (ABAG, 1995). The generalized hazard maps were prepared by dam owners as required by the State Office of Emergency Services; they are intended for planning purposes only. Based on our review of these maps, the site is not located within a dam failure inundation area.

SECTION 5: CONCLUSIONS

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

- Potential for significant post-construction settlement in the vicinity of Buildings 4, 5, 6 and the podium garage due to:
  - ✓ Potentially liquefiable alluvial soils
  - ✓ Cut/fill or material transitions
  - ✓ Presence of artificial (undocumented) fill

- Shallow ground water
- Potential for shallow soil creep or erosion on north-facing slopes

5.1 POST-CONSTRUCTION SETTLEMENTS

5.1.1 Liquefaction Settlement

As discussed, our preliminary liquefaction analysis indicates that there is a potential for liquefaction of localized sand layers during a significant seismic event. These sand layers were encountered in Borings EB-1 and EB-2 within the former alluvial soil area that has subsequently been filled by artificial (undocumented) fill.

Our preliminary analysis indicates that liquefaction-induced settlement on the order of ¼ to 1½ inches could occur, resulting in differential settlement up to 1 inch. Due to the proposed cuts for portions of Building 6, the potential for liquefied sands to vent to the ground surface through cracks in the surficial soils is considered moderate. Therefore, the magnitude of total settlement may be greater than that predicted in our analysis.

5.1.2 Cut/Fill or Material Transitions

Material transitions occur when two or more materials with differing geotechnical characteristics interface in a small area, such as within a single lot or building pad. The materials that comprise these transitions can include bedrock, surficial soils, and engineered fill. Because the geotechnical characteristics of the materials are different, the long-term and seismic performance of these materials is also different. For instance, fill materials, even if well compacted, are typically more compressible than bedrock materials and as a result will usually experience a greater amount of settlement. The differences in the amount of settlement or expansion between fill materials and bedrock materials can cause distress to building foundations and other site improvements. Such distress will often either add to the long-term maintenance costs or reduce the design life associated with the structure.

5.1.3 Undocumented Fill

As previously discussed, undocumented fill on the order of 2- to 10-feet-thick was encountered in Borings EB-1, EB-2 and EB-3 drilled at the site. The fill is highly variable and may not uniformly support the proposed structures. To support structures on a shallow foundation system, the existing fill within the footprint of Buildings 4, 5, 6 and podium garage would need to be removed and reworked as engineered fill prior to placing any new fill. Preliminary recommendations for mitigating undocumented fills are provided in the “Earthwork” section.

5.1.4 Settlement Mitigation Options

The above geotechnical concerns will all contribute to post-construction settlement of buildings straddling the fill and alluvial soil area of the site, and will likely exceed tolerable limits of differential movement. Therefore, the use of conventional shallow footings in these areas may not be feasible. There are several options that can be considered to mitigate differential settlement due to liquefaction, material transitions and undocumented fills. These options include:

1. Support shallow foundations over one of the following ground improvement options:

    A. Over-excavate potentially liquefiable soil, undocumented fills or material transitions and replacing with engineered fill material,

    B. Perform ground improvement (such as rammed aggregate piers, soil-cement mixing, or vibro-compaction/impact piers) to densify potential liquefiable layers and fill materials

2. Support buildings on a deep foundation system that derives support from the underlying bedrock with structural slabs designed to span unsupported between deep foundations and grade beams.

Options 1A and 1B can be designed and constructed to also mitigate the potential for ground rupture, allowing conventional slabs-on-grade to be utilized. Option 2 will likely include designing the deep foundations to accommodate the potential liquefaction-induced downdrag and ground rupture; therefore, conventional slabs-on-grade may not be utilized and structural slabs required.

On a preliminary basis, we recommend that Buildings 4, 5, 6 and the podium garage be supported on a deep foundation system, such as drilled, cast-in-placed friction piers deriving support from the underlying bedrock materials, or be supported on shallow footings bearing on engineered or ground improved soils. In shallow bedrock areas where liquefiable soils are not present, buildings can be supported on shallow footings. Preliminary foundation recommendations are presented in the “Foundations” section.

5.2 SHALLOW GROUND WATER

Shallow ground water was measured at depths ranging from approximately 10 to 13½ feet below the existing ground surface in the artificial (undocumented) fill area. Our experience with similar sites indicates that shallow ground water could significantly impact grading and underground construction for excavations extending near or below the ground water level. These impacts typically consist of potentially wet and unstable pavement or building pad subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering and shoring of utility trenches and basement excavations may be required in some isolated areas of the site. Preliminary recommendations addressing this concern are presented in the “Earthwork” section of this report.

5.3 SHALLOW SLOPE CREEP OR EROSION

Due to the loose, sandy nature of the topsoil mantling the north-facing slopes, shallow soil creep or erosion is expected to occur at the site during periods of heavy rainfall. Such movement is generally slow and gradual and occurs in the upper few inches or few feet of soils under the influence of gravity or during periods of intense rainfall. Though not a serious geologic hazard, this condition could be a nuisance to the proposed development where slow displacement or erosion of surficial soil could impact site improvements. Adequate erosion protection will need to be considered by the design team in these areas.

5.4 DESIGN-LEVEL GEOTECHNICAL INVESTIGATION

The preliminary recommendations contained in this report were based on limited site development information and limited exploration. As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that we be retained to 1) perform a design-level geotechnical investigation once detailed site development plans are available; 2) to review the geotechnical aspects of the project structural, civil, and landscape plans and specifications, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction; and 3) be present to provide geotechnical observation and testing during earthwork and foundation construction.

SECTION 6: PRELIMINARY EARTHWORK RECOMMENDATIONS

6.1 SITE DEMOLITION, CLEARING AND PREPARATION

6.1.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements within the proposed development area. A discussion of removal of existing fills is provided later in this report. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Based on our site observations, surficial stripping should extend about 2 to 4 inches below existing grade in vegetated areas to receive fill.

6.1.2 Tree and Shrub Removal

Trees and shrubs designated for removal should have the rootballs and any roots greater than ½-inch diameter removed completely. Grade depressions resulting from rootball removal should be cleaned of loose material and backfilled in accordance with the recommendations in the "Compaction" section of this report.

6.1.3 Abandonment of Existing Utilities

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

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

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

6.2 REMOVAL OF EXISTING FILLS

As discussed, portions of the site are blanketed by 2 to 10 feet of undocumented fill. If buildings in these areas are to be supported by deep foundations to mitigate post-construction settlement, on a preliminary basis, we recommend that the upper 2 feet of remaining fill in building pad areas be over-excavated and replaced with compacted fill to a lateral distance of at least 5 feet beyond the building footprint. If shallow foundations are considered for buildings straddling the fill areas, then all undocumented fill will need to be removed and replaced with engineered fill.

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 screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below.

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

6.3 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards.

Excavations performed during site demolition and fill removal should be sloped at 3:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Excavations extending more than 5 feet below building subgrade and excavations in pavement and flatwork areas should be slope at a 1:1 inclination unless the OSHA soil classification indicates that slope should not exceed 1.5:1.

6.4 MATERIAL FOR FILL

6.4.1 Re-Use of On-site Soils

On-site soils with an organic content less than 3 percent by weight may be reused as general fill. General fill should not have lumps, clods or cobble pieces larger than 6 inches in diameter; 85 percent of the fill should be smaller than 2½ inches in diameter. Minor amounts of oversize material (smaller than 12 inches in diameter) may be allowed provided the oversized pieces are not allowed to nest together and the compaction method will allow for loosely placed lifts not exceeding 12 inches, such an 815 or REX compactor.

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

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

6.5 COMPACTION REQUIREMENTS

Imported and engineered fill material should be placed in loose lifts 8 inches thick or less and compacted to at least 90 percent relative compaction in accordance with ASTM D1557 (latest version) requirements. Fill placed below the upper 5 feet of finished grade should be compacted to at least 93 percent relative compaction. In general, fill should be compacted at moisture contents at least 1 to 3 percent above the laboratory optimum.

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.

6.6 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.

6.7 PERMANENT CUT AND FILL SLOPES

All permanent cut and fill slopes in soil should have a maximum inclination of 2:1 (horizontal:vertical) for slopes up to 10 feet high; slopes greater than 10 feet should be inclined at no greater than 3:1. All permanent cuts in competent bedrock may have a maximum inclination of 2:1. Fill slopes should be overbuilt and trimmed back, exposing engineered fill when complete. Erosion control will be needed on all disturbed and engineered fill slopes.

6.8 SITE DRAINAGE

6.8.1 General Surface Drainage

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 pavements or concrete flatwork. Surface drainage should be directed towards suitable drainage facilities such as lined v-ditches or drain inlets. Lined v-ditches should be included at the top of slopes and intermediate benches, and at the toe of open space adjacent to planned 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.

6.8.2 Building Pad Surface Drainage

Ponding should not be allowed adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 2 to 3 percent towards suitable discharge facilities; landscape areas should slope at least 3 to 5 percent. Roof runoff should be directed away from building areas. Landscape drainage such as drain inlets and storm water filtration and/or infiltration trenches should be provided to collect and transmit storm water runoff to project storm drains, and/or detention or retention facilities.

6.8.3 Subsurface Drainage

Subdrains should be installed at the toe of any proposed cut slopes and behind site retaining and basement walls, depending on the actual conditions observed during construction. The actual location of subdrains should be determined in the field at the time of construction.

6.9 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. Alternatively, temporary shoring may support the planned cuts up to 15 to 20 feet. 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.

6.9.1 Temporary Shoring

Based on the site conditions encountered during our investigation, the proposed cuts into bedrock and surficial soils can likely be supported by soldier beams and tie-backs, 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 and incidental loading, existing structure foundation loads, and street loading. Heavy construction loads (cranes, etc.) and material stockpiles will need to 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. Detailed design criteria for shoring design should be developed during the design-level geotechnical investigation.

We performed our borings with hollow-stem auger drilling equipment and as such were not able to evaluate the potential for caving soils, which can create difficult conditions during soldier beam, tie-back, or soil nail installation; caving soils can also be problematic during excavation and lagging placement. The contractor is responsible for evaluating excavation difficulties prior to construction. Where relatively clean sands (especially encountered below ground water) 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. Shoring contractors should take into consideration the potential for slower production rates, and increased tie-back/soil nail shaft diameter (i.e. higher grout take) due to caving sands.

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

The above information is 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.

6.9.2 Construction Dewatering

Ground water levels in the existing fill areas are expected to be at about 5 to 7 feet below the planned excavation bottom. However, perched ground water could be encountered during excavation for basement parking. Therefore, temporary dewatering may be necessary during construction. Design, selection of the equipment and dewatering method, and construction of temporary dewatering should be the responsibility of the contractor. Modifications to the dewatering system are often required in layered alluvial soils and should be anticipated by the contractor. The dewatering plan, including planned dewatering well filter pack materials, should be forwarded to our office for review prior to implementation.

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

SECTION 7: PRELIMINARY FOUNDATION RECOMMENDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

As discussed, due to the potential for differential settlement for the proposed structures that will straddle cut/fill or material transitions or be underlain by potentially liquefiable materials, the proposed buildings should be supported on a deep foundation system consisting of drilled, cast-in-place friction piers. As an alternative to deep foundations, the differential settlement and ground rupture potential can be mitigated by either over-excavating material transitions or performing ground improvement in fill and alluvial soil areas. Differential foundation movement is anticipated to impact all or portions of Buildings 4, 5, 6 and the podium garage. If earthwork mitigation or ground improvement is performed, these buildings can likely be supported on shallow foundations. Buildings 1 through 3 can likely be supported on shallow footings without requiring settlement mitigation. Preliminary foundation recommendations are presented in the following sections.

7.2 SHALLOW FOUNDATIONS

7.2.1 Spread Footings

For buildings located in bedrock or non-fill areas, the buildings may be supported on shallow spread or continuous strip footings. Footings should bear on natural, undisturbed soil or engineered fill, be at least 18 inches wide, and extend at least 18 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.

On a preliminary basis, footings constructed to the above dimensions will likely be capable of supporting maximum allowable bearing pressures on the order of 3,000 to 4,000 psf for dead plus live loads. This pressure is a net value; the weight of the footing may be neglected for the portion of the footing extending below grade (typically, the full footing depth). Top and bottom mats of reinforcing steel should be considered in continuous footings to help span irregularities and differential settlement.

7.2.2 Footing Settlement

Structural loads were not provided to us at the time this report was prepared. Based on the range of allowable bearing pressures presented above and assuming buildings supported on shallow footings are bearing entirely in native bedrock, stiff soil, engineered fill, or overlying ground improvement, we estimate that the total static footing settlement will be on the order of 1 inch or less, with approximately ½-inch or less of post-construction differential settlement between adjacent foundation elements. Seismic settlements are assumed to be negligible where footings overly shallow bedrock or based on the ground mitigation requirements.

7.2.3 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.

7.3 DRILLED PIERS

To mitigate potential post-construction settlement for Buildings 4, 5, 6 and the podium garage, on a preliminary basis, these buildings should be supported on drilled, cast-in-place, straight-shaft friction piers. The piers should have a minimum diameter of 18 inches and extend to a depth of at least 5 feet into bedrock. Adjacent piers centers should be spaced at least three diameters apart, otherwise, a reduction for group effects may be required. Grade beams should span between piers and/or pier caps in accordance with structural requirements. Slabs should be designed to span unsupported between piers and grade beams to mitigate the effects of differential movement from liquefaction-induced ground rupture.

On a preliminary basis, the vertical capacity of the piers may be evaluated based on an allowable skin friction of approximately 600 psf for combined dead plus live loads based on a factor of safety of 2.0; dead loads should not exceed two-thirds of the allowable capacities. The allowable skin friction may be increased by one-third for wind and seismic loads. Depending on the finished foundation elevation, it may be necessary to neglect all or a portion of the skin friction within the fill or native alluvial soils. Total settlement of individual piers or pier groups of four or less should not exceed ½ inch to mobilize static capacities.

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

7.4 GROUND IMPROVEMENT

7.4.1 General

Another option to mitigate potential post-construction settlement for Buildings 4, 5, 6 and the podium garage, would be to perform ground improvement in areas where increased settlement is estimated to occur. Ground improvement, such as impact or vibro-piers, stone columns, or other similar system, should improve the subsurface soils to reduce total differential (static and seismic) settlements to an allowable level for spread footings. Rather than eliminating all of the potential settlement, ground improvement would be used to reduce settlements to acceptable levels for a shallow foundation system. The intent of ground improvement is to increase the density of potentially liquefiable soils or loose fill material by laterally displacing and/or densifying the existing in-place soils. The degree to which the density is increased will depend on the improvement method and spacing. In addition to increasing the density, ground improvement may also provide an additional increase in bearing capacity and soil stiffness at individual improvement locations. Ground improvement can also provide better support for slab-on-grade areas so that the potential for adverse slab differential movement is reduced after strong seismic shaking.

7.4.2 Rammed Aggregate Piers Foundation System

Rammed Aggregate Piers (RAPs), such as the Geopier® Impact RAPs foundation system are typically 20- to 24-inch-diameter elements spaced approximately 4 to 8 feet on center. They are constructed by driving a hollow steel pipe mandrel into the ground and compacting layers of crushed rock with a 150-ton vibratory hammer. Typical improvement depths range from approximately 15 to 50 feet. Since Impact Piers displace existing soils as they are densified, no additional spoils are generated during construction. Conventional RAPs are constructed by drilling a 30-inch diameter shaft (similar to drilled piers) and compacting layers of either crushed rock or Class 2 aggregate base with a high-impact ramming tool attached to an excavator. Typical improvement depths range from approximately 15 to 30 feet. Drill spoils would need to be re-used on-site or off-hauled.

Ground improvement designs should typically include, but not be limited to the following:

- drawings showing the ground improvement layout, spacing and diameter
- the foundation layout plan
- proposed ground improvement length
- top and bottom elevations
- Post-construction CPT tip resistance criteria to be achieved in the sand layers after installation and refusal criteria

Additional exploration will be required during the design-level investigation to further characterize the subsurface conditions in ground improvement areas.

7.4.3 Ground Improvement Performance Testing

In our opinion, performing ground improvement for the portions from approximately 10 to 20 feet of the subsurface profile in the fill and alluvial soil areas could potentially reduce the estimated settlements to tolerable levels. However, design of ground improvement including depths and limits will be the responsibility of the design-build ground improvement contractor. The performance criteria should be based on reducing the estimated total foundation settlements to tolerable levels approved by the structural engineer. Cornerstone should work with and provide geotechnical input and design parameters to the ground improvement design-built contractor. We should review the final design and plans to confirm foundation estimates and recommendations following the design-level geotechnical investigation.

SECTION 8: CONCRETE SLABS

The following recommendations are for buildings supported on shallow foundations overlying shallow bedrock or ground improvement, as previously discussed. If Buildings 4, 5, 6 and the podium garage are supported on deep foundations, portions of the lower level garage slab may need to be designed as a structural slab that is capable of spanning between deep foundation elements.

8.1 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.

In general, garage slabs should be supported on at least 4 inches of which should consist of either Class 2 aggregate base or ¾-inch clean, crushed rock place and compacted in accordance with the "Compaction" section of this report. If there will be areas within the garage that are moisture sensitive, such as equipment and elevator rooms, the recommendations in the "Interior Slabs Moisture Protection Considerations" section below may be incorporated in the project design if desired. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

8.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

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

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

- The 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 should not be allowed versus light broom or limited trowel finishing.

SECTION 9: VEHICULAR PAVEMENTS

9.1 ASPHALT CONCRETE

The following preliminary 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 20. The design R-value was chosen based on the relatively sandy and low plasticity soils encountered within the upper 10 feet and engineering judgment considering the variable surface conditions.

Table 4: Preliminary Asphalt Concrete Pavement Recommendations

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness** (inches)
4.0	2.5	6.0	8.5
4.5	2.5	7.0	9.5
5.0	3.0	7.0	10.0
5.5	3.0	9.0	12.0
6.0	3.5	10.0	13.5
6.5	4.0	11.0	15.0

*Caltrans Class 2 aggregate base; minimum R-value of 78
**Preliminary subgrade design R-value = 20

Additional laboratory testing should be performed during the design-level geotechnical investigation to further evaluate the design R-value for potential subgrade materials on site.

SECTION 10: RETAINING WALLS

10.1 LATERAL EARTH PRESSURES

The structural design of any site retaining walls 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. A drainage system should be constructed behind the walls to prevent the build-up of hydrostatic pressures. The following preliminary lateral earth pressures may be considered for preliminary cost estimating and conceptual design:

Table 5: Preliminary Lateral Earth Pressures

Sloping Backfill Inclination (horizontal:vertical)	Lateral Earth Pressure*	
	Unrestrained – Cantilever Wall	Restrained – Braced Wall
Level	40 pcf	40 pcf + 8H (in psf)
3:1	50 pcf	50 pcf + 8H (in psf)
2½:1	55 pcf	55 pcf + 8H (in psf)
2:1	60 pcf	60 pcf + 8H (in psf)
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

** 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 BELOW-GRADE WALL DRAINAGE

Miradrain, AmerDrain or other equivalent drainage matting should be used for wall drainage where below-grade walls are temporarily shored and the shoring will be flush with the back of the permanent walls. The drainage panel should be connected at the base of the wall by a horizontal drainage strip and closed or through-wall system such as the TotalDrain system from AmerDrain. 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.

SECTION 11: LIMITATIONS

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

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

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

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

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

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

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

SECTION 12: REFERENCES

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AERIAL PHOTOGRAPHS:

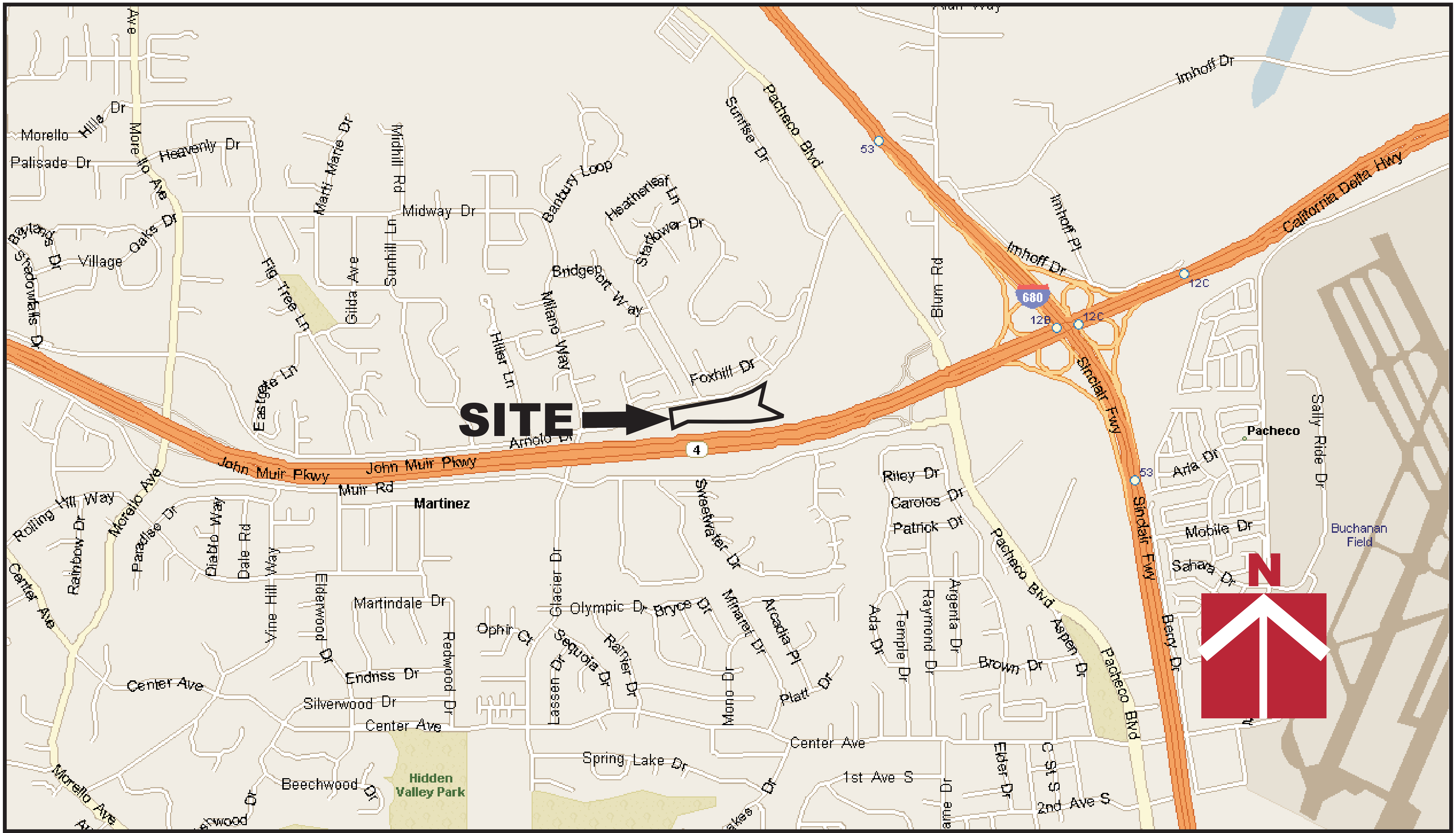
Geomorphic features on the following aerial photographs obtained from Environmental Data Resources (EDR) were interpreted as part of this investigation:

Year	Source	Type	Scale
1939	Fairchild	Black & white	1″ = 555′
1946	Jack Ammann	Black & white	1″ = 655′
1958	Cartwright	Black & white	1″ = 555′
1965	Cartwright	Black & white	1″ = 333′
1975	NASA	Black & white	1″ = 550′
1982	USGS	Black & white	1″ = 690′
1993	USGS	Black & white	1″ = 666′
1998	USGS	Black & white	1″ = 666′
2005	EDR	Color	1″ = 604′

HISTORIC TOPOGRAPHIC MAPS:

Geomorphic features on the following USGS topographic maps obtained from Environmental Data Resources (EDR) were interpreted as part of this investigation:

Year	Quad	Series	Scale
1915	Concord	15-Minute	1:62500
1948	Concord	15-Minute	1:50000
1949	Walnut Creek	7.5-Minute	1:24000
1959	Concord	7.5-Minute	1:62500
1968	Walnut Creek	7.5-Minute	1:24000
1973	Walnut Creek	7.5-Minute	1:24000
1980	Walnut Creek	7.5-Minute	1:24000
1993	Walnut Creek	7.5-Minute	1:24000

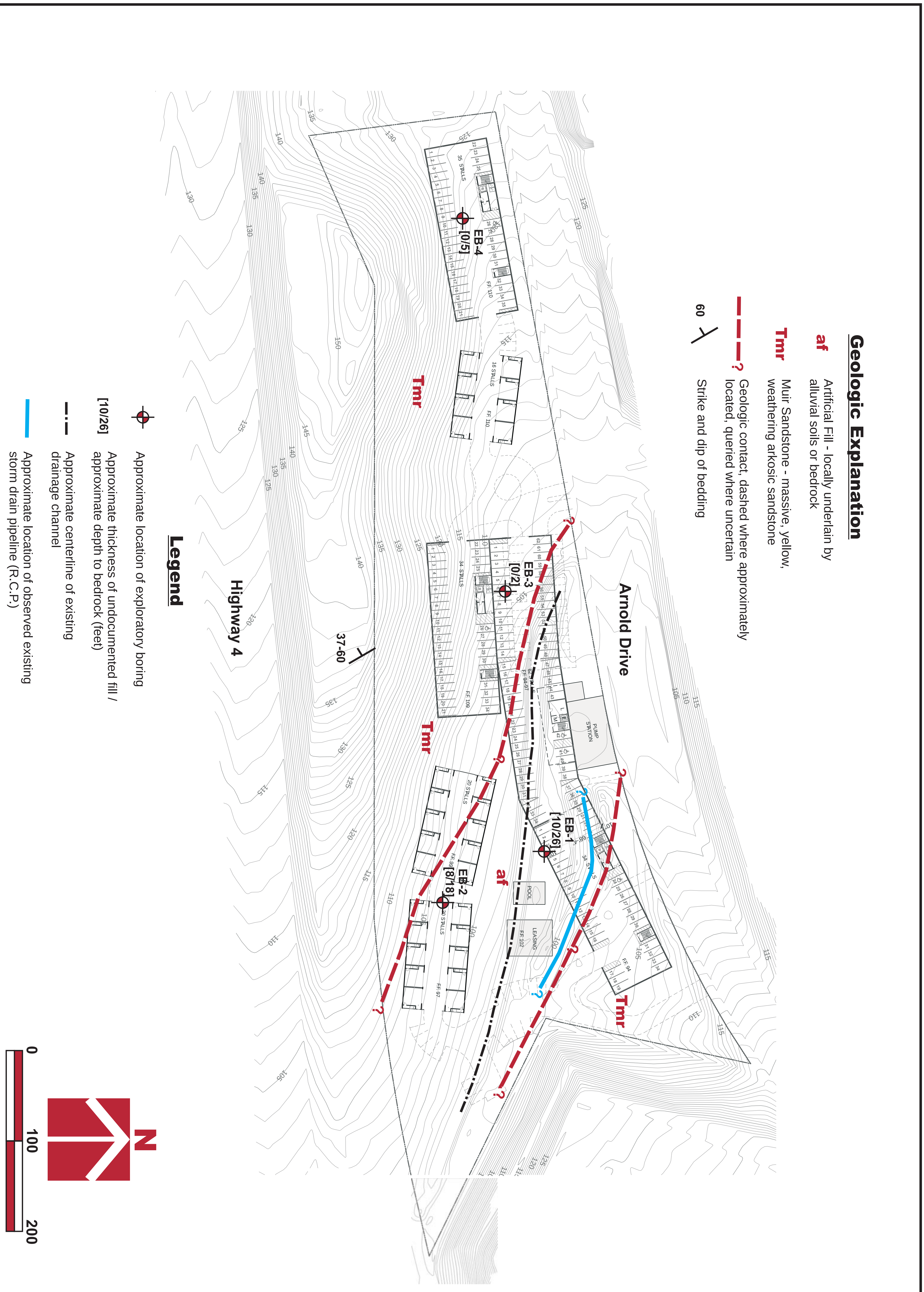


CORNERSTONE EARTH GROUP

Vicinity Map

Hill Valley Oaks Apartments
Martinez, CA

Project Number	351-1-1
Figure Number	Figure 1
Date	November 2009
Drawn By	FLL



Base map by JGA Architechts, Dated 8-13-09.

CORNERSTONE EARTH GROUP

Site Plan and Preliminary Geologic Map

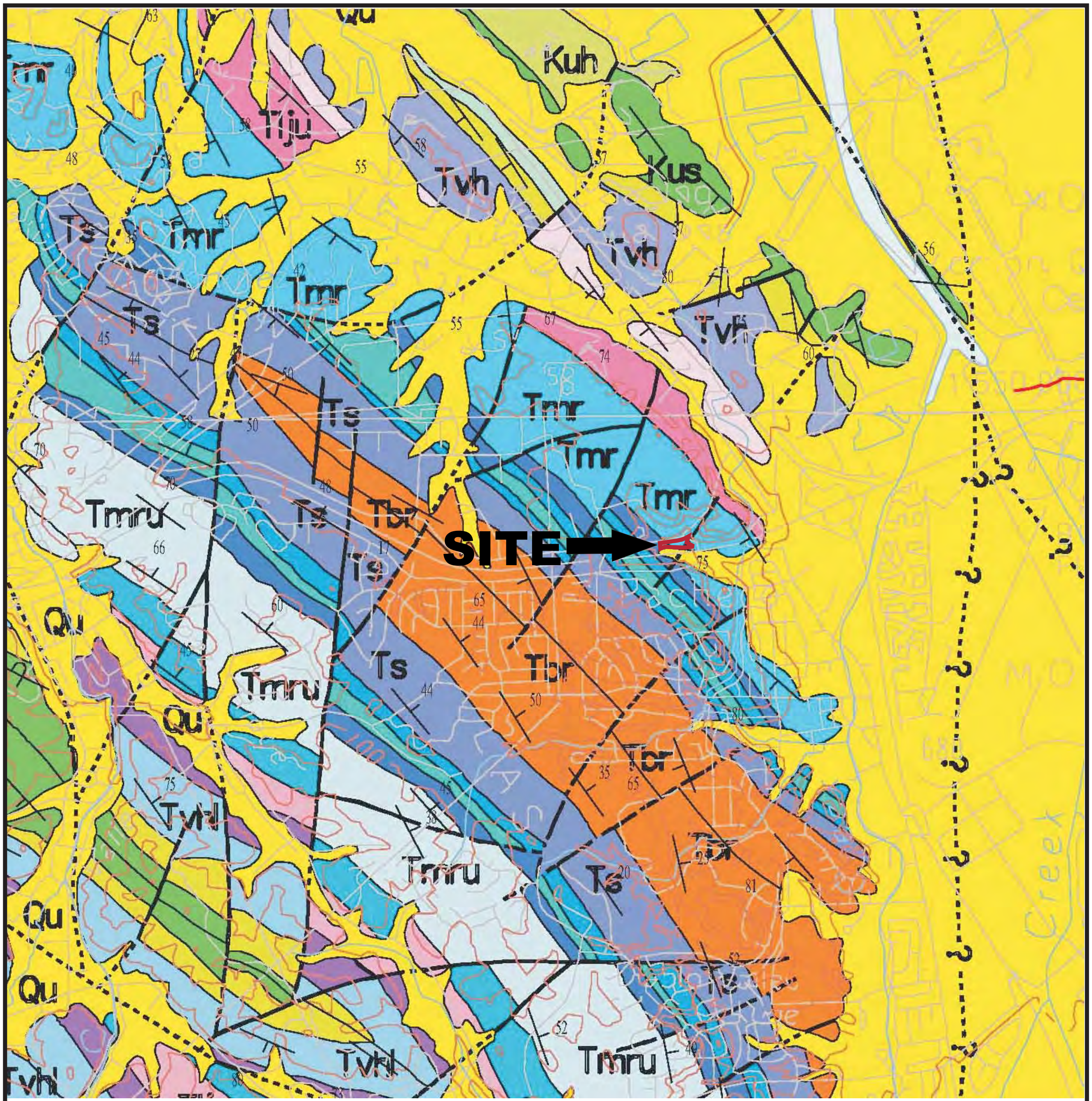
Hill Valley Oaks Apartments
Martinez, CA

Project Number 351-1-1

Figure Number Figure 2

Date November 2009

Drawn By FLL



Local Geologic Units

Tmr	Muir Sandstone - massive, yellow, weathering arkosic sandstone
Tbr	Briones Formation, sandstone, siltstone, conglomerate and shell breccia

Explanation

Geologic contact, dashed where approximately located

Fault, dashed where approximately located, queried where uncertain

52 Strike and dip of bedding

N

0 6,250 12,500

APPROXIMATE SCALE (FEET)

Base by Graymer, R.W., Jones, D.L., and Brabb, E.E., 1994, Preliminary Geologic Map Emphasizing Bedrock Formations in Contra Costa County, California: a digital database: USGS Open File Report 94-622.

CORNERSTONE
EARTH GROUP

Regional Geologic Map

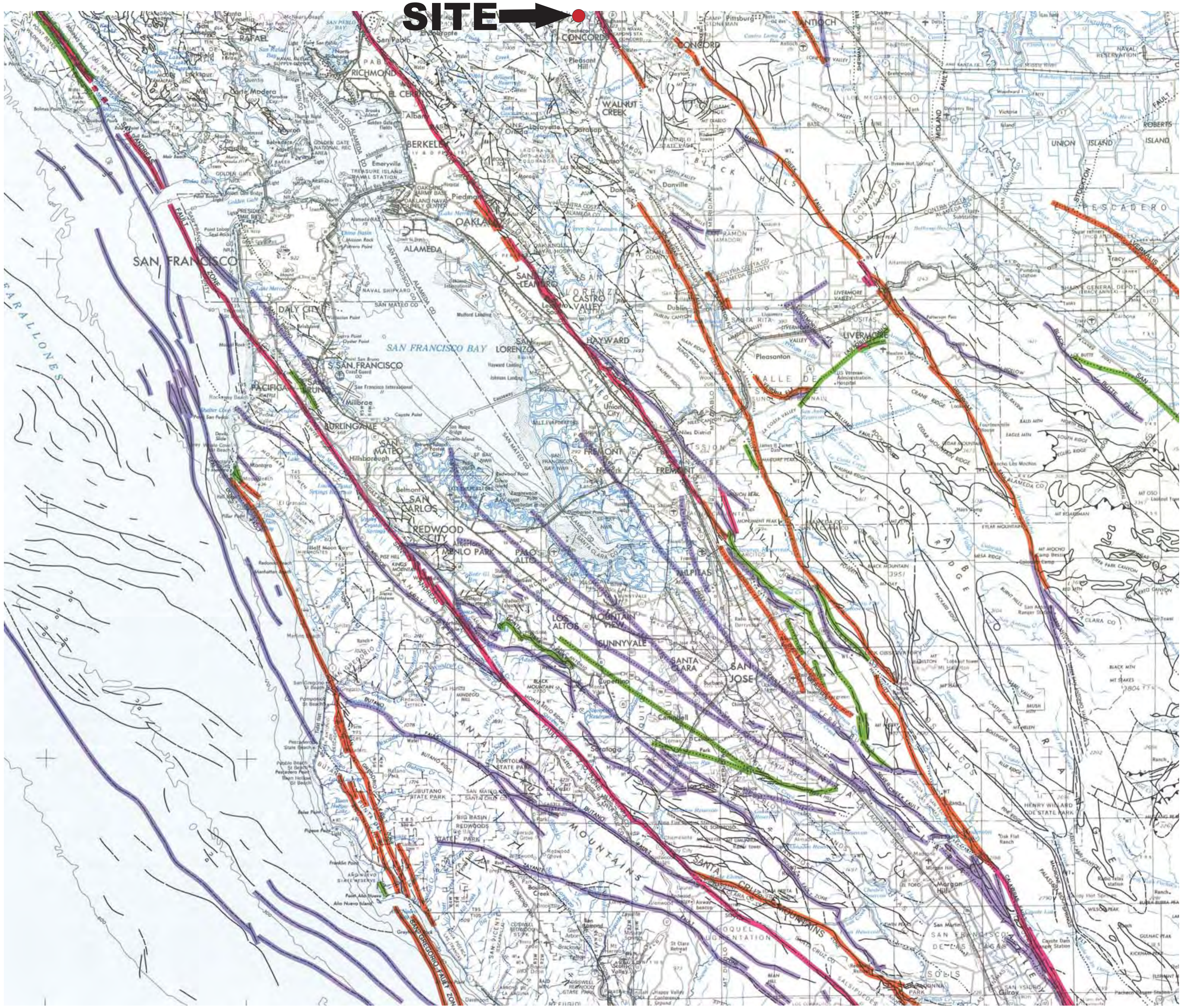
Hill Valley Oaks Apartments
Martinez, CA

Project Number
351-1-1

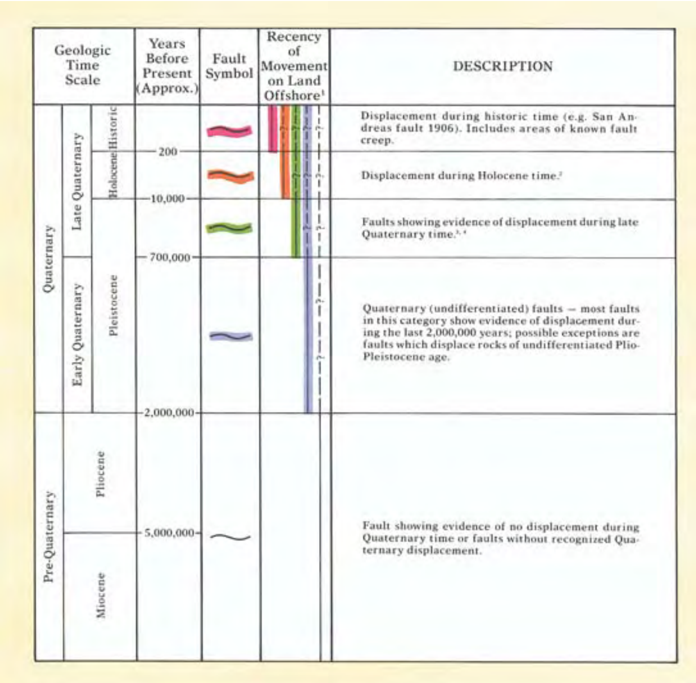
Figure Number
Figure 3

Date
November 2009

Drawn By
FLL



Base Map: "Map showing recency of faulting, San Francisco - San Jose Quadrangle California,"
by Bortugno, et.al. (California Division of Mines and Geology) dated 1991.



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement on Land Offshore[1]	DESCRIPTION
Quaternary	Late Quaternary	Historic	200	🟥		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.
		Holocene	10,000	🟧		Displacement during Holocene time.[2]
		Pleistocene	700,000	🟩		Faults showing evidence of displacement during late Quaternary time.[3, 4]
	Early Quaternary		2,000,000	🟪		Quaternary (undifferentiated) faults — most faults in this category show evidence of displacement during the last 2,000,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.
Pre-Quaternary	Pliocene		5,000,000	~		Fault showing evidence of no displacement during Quaternary time or faults without recognized Quaternary displacement.
	Miocene					

N

0 10 20

APPROXIMATE SCALE (MILES)

CORNERSTONE EARTH GROUP

Regional Fault Map

Hill Valley Oaks Apartments
Martinez, CA

Project Number
351-1-1

Figure Number
Figure 4

Date
November 2009

Drawn By
FLL

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment. Four 8-inch-diameter exploratory borings were drilled on October 27, 2009, to depths of approximately 19 to 39 feet. The approximate locations of exploratory borings are shown on the Site Plan & Preliminary Geologic Map, 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 and other site features as references. Boring elevations were based on interpolation of plan contours. The locations and elevations 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-2487-98)

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

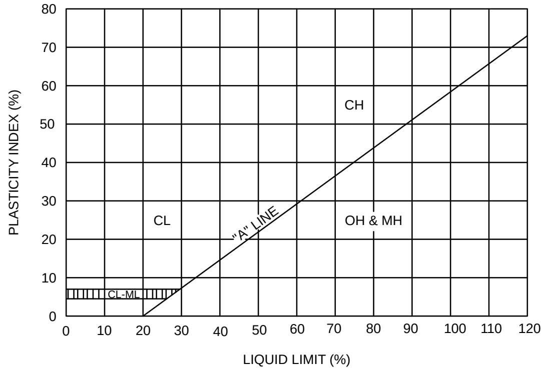
SAMPLER TYPES

- ☒ SPT
- ⧗ Modified California (2.5" I.D.)
- ▯ Rock Core
- ■ Shelby Tube
- ○ No Recovery
- ✋ 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

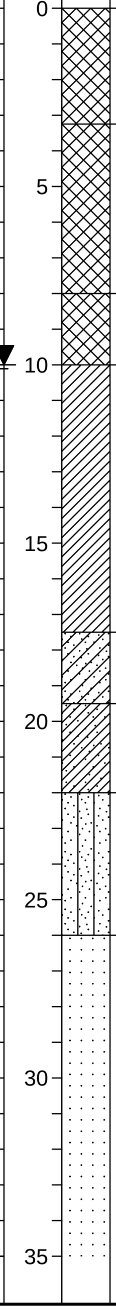
BORING NUMBER EB-1

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME Hill Valley Oaks Apartments
PROJECT NUMBER 351-1-1
PROJECT LOCATION Martinez, CA
DATE STARTED 10/27/09 **DATE COMPLETED** 10/27/09
GROUND ELEVATION 98.5 FT +/- **BORING DEPTH** 39 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
NORTHING **EASTING**
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY JLF
▽ **AT TIME OF DRILLING** 10 ft.
NOTES
▼ **AT END OF DRILLING** 10 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.



ELEVATION (ft)	DEPTH (ft)	DESCRIPTION
98.5	0	**Sandy Lean Clay (CL) [Fill]** hard, moist, olive brown with light brown mottles, fine to medium sand, low plasticity
95.3		**Clayey Sand with Gravel (SC) [Fill]** dense to medium dense, moist, light brown and gray mottled, fine to medium sand, fine to coarse subangular gravel (claystone), some clay nodules
	5	
90.5		**Sandy Lean Clay (CL) [Fill]** soft, moist, dark gray, fine sand, low to moderate plasticity (sample lost with MC, recovered with SPT sampler)
88.5 ▼	10	**Lean Clay with Sand (CL)** stiff, moist, dark gray to olive gray, fine sand, moderate plasticity
	15	
81.0		**Clayey Sand (SC)** medium dense, moist to wet, gray and brown mottled, fine sand, fine to medium sand
79.0		**Sandy Lean Clay (CL)** stiff, moist, olive gray, fine sand, low to moderate plasticity
	20	
76.5		**Silty Sand (SM)** medium dense, moist, reddish brown, fine to medium sand
	25	
72.5		**Muir Sandstone [Tmr]** low hardness, friable to weak, moderate weathering, light gray, light cementation
	30	
63.5	35	

Continued Next Page

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
62	MC-1B	94	24			○
55	MC-2B	104	19			
21	MC-3B	106	21			
3	SPT-4A		22		50	○
28	MC-5A	103	21			
38	MC-6A	100	20		33	
47	MC-7A	106	20		26	
50/3"	SPT-8		18			
50/3"	SPT					

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0 5.0

BORING NUMBER EB-1

PAGE 2 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME Hill Valley Oaks Apartments
PROJECT NUMBER 351-1-1
PROJECT LOCATION Martinez, 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
1.0 2.0 3.0 4.0 5.0

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
63.5	35		**Muir Sandstone [Tmr]** low hardness, friable to weak, moderate weathering, light gray, light cementation							
				50 2"	☒ SPT					
59.5			Bottom of Boring at 39.0 feet.							
	40									
	45									
	50									
	55									
	60									
	65									
	70									
	75									

BORING NUMBER EB-2

PAGE 1 OF 1

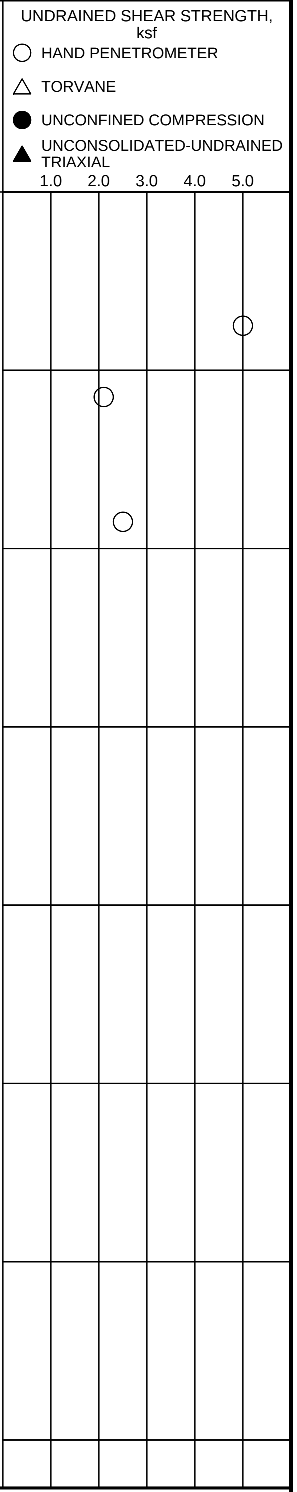
CORNERSTONE EARTH GROUP

PROJECT NAME Hill Valley Oaks Apartments
PROJECT NUMBER 351-1-1
PROJECT LOCATION Martinez, CA
DATE STARTED 10/27/09 **DATE COMPLETED** 10/27/09
GROUND ELEVATION 103 FT +/- **BORING DEPTH** 19 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
NORTHING **EASTING**
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY JLF
∇ **AT TIME OF DRILLING** 13.5 ft.
NOTES
▼ **AT END OF DRILLING** 13.5 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.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION
103.0	0	**Lean Clay with Sand (CL) [Fill]** hard, moist, light brown, fine sand, low plasticity
101.0		**Clayey Sand with Gravel (SC) [Fill]** dense, moist, light gray and brown mottled, fine to coarse sand, fine to coarse subangular gravel
100.0		
	5	**Lean Clay with Sand (CL) [Fill]** very stiff, moist, olive, gray and brown mottled, fine to coarse sand, moderate plasticity
95.0		**Lean Clay with Sand (CL)** very stiff, moist, gray with brown mottles, fine sand, moderate plasticity
	10	
91.0		**Silty Sand (SM)** medium dense, moist to wet, gray and brown mottled, fine to medium sand
	15	
85.0		**Muir Sandstone [Tmr]** low hardness, friable to weak, moderate weathering, light gray, light cementation
83.0	20	Bottom of Boring at 19.0 feet.
	25	
	30	
	35	

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE
50 4"	MC-1	107	15		
73	MC				
42	MC-3B	99	22		
38	MC-4B	111	18		
27	MC-5A		16		40
50 2"	SPT				



BORING NUMBER EB-3

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME Hill Valley Oaks Apartments
PROJECT NUMBER 351-1-1
PROJECT LOCATION Martinez, CA
GROUND ELEVATION 106 FT +/- **BORING DEPTH** 19 ft.
NORTHING **EASTING**
GROUND WATER LEVELS:
▽ **AT TIME OF DRILLING** Not Encountered
▼ **AT END OF DRILLING** Not Encountered

DATE STARTED 10/27/09 **DATE COMPLETED** 10/27/09
DRILLING CONTRACTOR Exploration Geoservices, Inc.
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
LOGGED BY JLF
NOTES

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.

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
106.0	0		**Lean Clay with Sand (CL) [Fill]** very stiff, moist, light brown and gray mottled, fine to medium sand, moderate plasticity	68	MC-1A	97	19			
104.0			**Muir Sandstone [Tmr]** low to moderate hardness, friable to weak, moderate weathering, light gray and light brown mottled, light cementation	50 5"	MC					
	5			50 2"	MC-3		7			
			increasing silt and clay	31	SPT-4		17			
	10									
				50 6"	SPT-5		14			
	15									
89.0			**Sandy Siltstone [Tmr]** low hardness, weak, fine to medium sand, bluish gray	50 5"	SPT					
87.0			Bottom of Boring at 19.0 feet.							
	20									
	25									
	30									
	35									

○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

1.0 2.0 3.0 4.0 5.0

CORNERSTONE EARTH GROUP

BORING NUMBER EB-4

PAGE 1 OF 1

PROJECT NAME Hill Valley Oaks Apartments

PROJECT NUMBER 351-1-1

PROJECT LOCATION Martinez, CA

DATE STARTED 10/27/09 **DATE COMPLETED** 10/27/09

GROUND ELEVATION 123 FT +/- **BORING DEPTH** 24 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

NORTHING ______ **EASTING** ______

DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY JLF

▽ **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

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 1.0 2.0 3.0 4.0 5.0
123.0	0		**Silty Sand (SM)** medium dense, moist, dark brown, fine sand Liquid Limit = 19, Plastic Limit = 14							
				47	MC-1A	118	12	5	48	
				51	MC-2	122	12			
118.0	5		**Muir Sandstone [Tmr]** low hardness, friable to weak, moderate weathering, light brown to light gray with reddish brown mottling							
				36	MC-3B	112	13			
				50/6"	MC-4	103	13			
	10									
				50/5"	SPT					
	15									
				50/4"	SPT					
			color change to gray							
	20									
99.0			Bottom of Boring at 24.0 feet.	50/3"	SPT					
	25									
	30									
	35									

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 19 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities

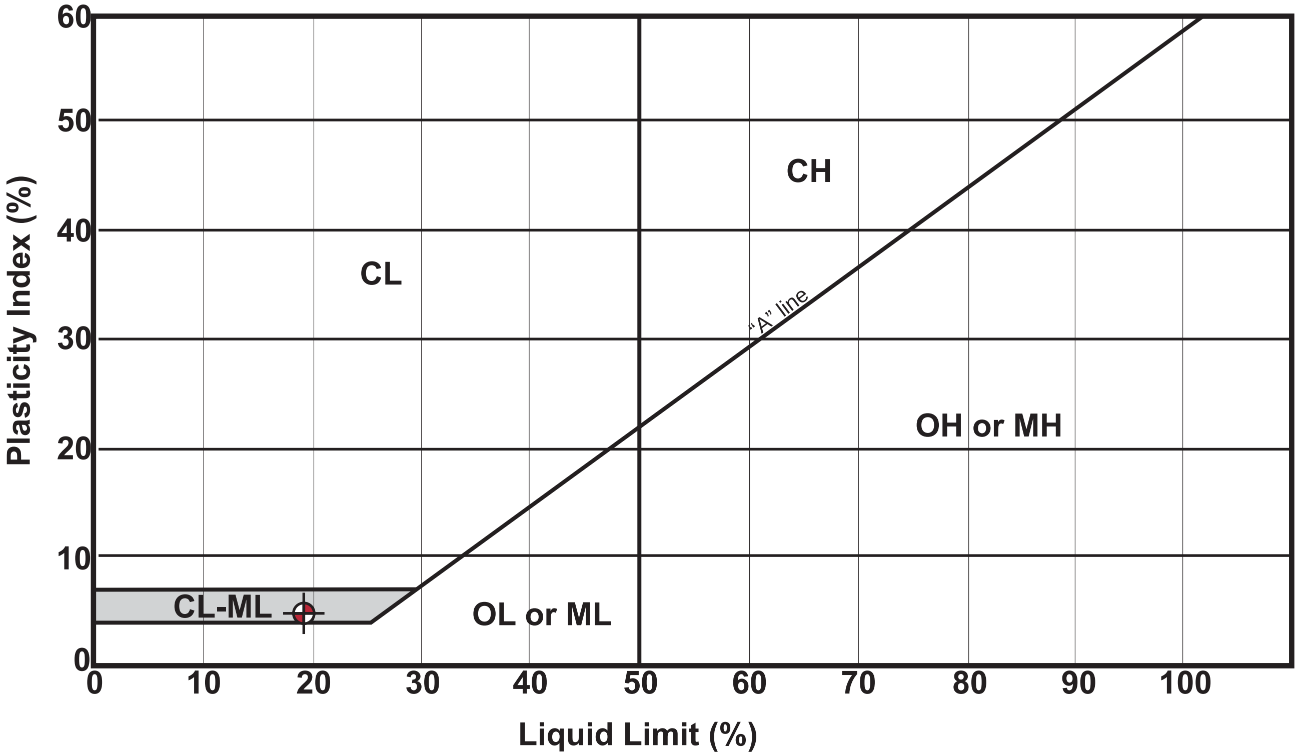
In place dry density determinations (ASTM D2937) were performed on 13 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Washed Sieve Analyses

The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on five 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: One Plasticity Index determinations (ASTM D4318) was 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 indicate that the surficial soil is low plasticity. Results of this test are shown on the boring log at the appropriate sample depths and the attached Figure B-1.

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-4	2.0	12	19	14	5	48	Silty Sand (SM)

CORNERSTONE EARTH GROUP

Plasticity Index Testing Summary

Hill Valley Oaks Apartments
Martinez, CA

Project Number
351-1-1

Figure Number
Figure B1

Date
November 2009

Drawn By
FLL

CE&G
CAL ENGINEERING & GEOLOGY

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Walnut Creek
California 94596

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Fax: 925.935.9773
www.caleng.com

2 May 2016

City of Martinez
525 Henrietta Street
Martinez, California 94553
Attention: Khalil Yowakim, P.E.

RE: Geotechnical Report Peer Review
Amare Apartment Homes
Martinez, California

Dear Mr. Yowakim:

At your request, we have completed our geologic and geotechnical review of the geotechnical report and preliminary project plans for the proposed Amare Apartment Homes development to be constructed between 2050 Arnold Drive and 2530 Arnold Drive in Martinez, California.

The following project documents were reviewed:

- Geotechnical Report by Cornerstone Earth Group titled, "Preliminary Geotechnical Investigation, Hill Valley Oaks Apartments, Arnold Drive, Martinez, California, Hill Valley Oaks, LCC," dated 17 November 2009.
- Plans by Humann Company, Inc. titled, "Amare Apartment Homes, APN 161-400-009 & 010, Arnold Drive, Martinez, California," sheets C01, C02, and C03 dated 11 April 2016 and sheets C3.1 and C3.2 dated 10 March 2016.

Our review has included examination of the above referenced materials for pertinent information regarding the technical feasibility of the project. We have also performed reconnaissance level observations of the project site and reviewed information in our files which include published soils and geologic information.

PROPOSED PROJECT

We understand that it is currently proposed to develop the property with nine apartment buildings and associated on-grade parking. Each apartment building will be two stories tall and contain between 12 and 15 units.

REVIEW OF GEOTECHNICAL REPORT

Our review of the Cornerstone geotechnical report revealed that the report is generally complete with respect to identifying the geotechnical conditions that will impact the project. However, the 2009 Cornerstone geotechnical report does not reflect the current project depicted on Human Company's plans.

We have the following comments based on our review of the geotechnical report:

Comment 1. Due to the age of the report, it will be necessary for Cornerstone to prepare an update or supplement to their 2009 report.

Comment 2. Page 1 of the report describes the project as consisting of 121 units with buildings that will be 3 to 4 stories tall. This description is inconsistent with the current plans, which show 128 units with building that are two stories tall.

The report should be updated and reissued to reflect the current project and building code requirements.

Comment 3. Page 6, Section 3.3.1, "Plasticity/Expansion Potential," indicates that one plasticity index (PI) test was completed on a representative near-surface soil sample. The PI test was completed on sample number MC-1A from boring EB-4 at a depth of approximately 2 feet. The soil had was classified as a silty sand (SM) having a PI of 5 percent.

Borings EB-1, EB-2, and EB-3 encountered sandy lean clay (CL) and lean clay with sand (CL) within 3 feet from the ground surface. In addition, surficial soils maps published by the United Stated Department of Agriculture (USDA) National Resource Conservation Service (NRCS) maps indicate that the northern half of the site is underlain by soils of the Positas loam series which consist of silt (ML) and clay (CL, CH) with plasticity indices between 20 and 35 percent.

The expansion potential of the near surface soils should be reevaluated by taking into account the near surface clay encountered in borings EB-1, EB-2, and EB-3 and the USDA NRCS soil mapping.

Comment 4. Page 8 recommends that additional subsurface exploration, laboratory testing and engineering analysis (additional geotechnical work) be performed during the design-level geotechnical investigation to further evaluate the potential for liquefaction-induced settlement beneath building that will straddle the fill/alluvial soil area of the site.

The additional geotechnical work should be completed prior to finalizing the project plans.

Comment 5. Page 23, Section 9, "Asphalt Concrete," recommends using a subgrade R-Value of 20 based on the relatively sandy and low plasticity soils encountered within the upper 10 feet. This, however, seems high for the clayey soil encountered at the site.

This R-Value should be reevaluated by completing R-Value tests during the course of the additional geotechnical work and by taking into account the near surface clay encountered in borings EB-1, EB-2, and EB-3 and the USDA NRCS soil mapping.

Comment 6. After completion of a supplement or update to the Cornerstone report, we should be provided with a copy to complete a review of responses to our comments. Similarly, Cornerstone should review the project plans prepared by Humann Company to confirm that the plans have been prepared in conformance with their recommendations. Cornerstone's review should be documented in writing and provided to the City.

CLOSURE

This review has been performed by request of the City of Martinez. Our role has been to provide technical advice to assist the City in its discretionary permit decisions, and we are afforded the same protection under state law. Our services have been limited to the review of the documents listed above, and a visual review of the property. We have no control over the future construction on this property and make no representations regarding its future conditions.

We have employed accepted engineering geology and civil and geotechnical engineering procedures, and our professional opinions and conclusions are made in accordance with generally accepted engineering geology and civil and geotechnical engineering principles and practices. This standard is in lieu of all other warranties, either expressed or implied.

Yours truly,

CAL ENGINEERING & GEOLOGY, INC.

Christopher A. Hockett
No. Geo GE 2928
No. Civil C 71938

Chris Hockett, G.E. 2928
Associate Engineer

5. Cal Engineering & Geology. *Geotechnical Report Peer Review Amare Apartment Homes, Martinez, California*. May 2, 2016.

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2 May 2016

City of Martinez
525 Henrietta Street
Martinez, California 94553
Attention: Khalil Yowakim, P.E.

RE: Geotechnical Report Peer Review
Amare Apartment Homes
Martinez, California

Dear Mr. Yowakim:

At your request, we have completed our geologic and geotechnical review of the geotechnical report and preliminary project plans for the proposed Amare Apartment Homes development to be constructed between 2050 Arnold Drive and 2530 Arnold Drive in Martinez, California.

The following project documents were reviewed:

- Geotechnical Report by Cornerstone Earth Group titled, "Preliminary Geotechnical Investigation, Hill Valley Oaks Apartments, Arnold Drive, Martinez, California, Hill Valley Oaks, LCC," dated 17 November 2009.
- Plans by Humann Company, Inc. titled, "Amare Apartment Homes, APN 161-400-009 & 010, Arnold Drive, Martinez, California," sheets C01, C02, and C03 dated 11 April 2016 and sheets C3.1 and C3.2 dated 10 March 2016.

Our review has included examination of the above referenced materials for pertinent information regarding the technical feasibility of the project. We have also performed reconnaissance level observations of the project site and reviewed information in our files which include published soils and geologic information.

PROPOSED PROJECT

We understand that it is currently proposed to develop the property with nine apartment buildings and associated on-grade parking. Each apartment building will be two stories tall and contain between 12 and 15 units.

REVIEW OF GEOTECHNICAL REPORT

Our review of the Cornerstone geotechnical report revealed that the report is generally complete with respect to identifying the geotechnical conditions that will impact the project. However, the 2009 Cornerstone geotechnical report does not reflect the current project depicted on Human Company's plans.

We have the following comments based on our review of the geotechnical report:

Comment 1. Due to the age of the report, it will be necessary for Cornerstone to prepare an update or supplement to their 2009 report.

Comment 2. Page 1 of the report describes the project as consisting of 121 units with buildings that will be 3 to 4 stories tall. This description is inconsistent with the current plans, which show 128 units with building that are two stories tall.

The report should be updated and reissued to reflect the current project and building code requirements.

Comment 3. Page 6, Section 3.3.1, "Plasticity/Expansion Potential," indicates that one plasticity index (PI) test was completed on a representative near-surface soil sample. The PI test was completed on sample number MC-1A from boring EB-4 at a depth of approximately 2 feet. The soil had was classified as a silty sand (SM) having a PI of 5 percent.

Borings EB-1, EB-2, and EB-3 encountered sandy lean clay (CL) and lean clay with sand (CL) within 3 feet from the ground surface. In addition, surficial soils maps published by the United Stated Department of Agriculture (USDA) National Resource Conservation Service (NRCS) maps indicate that the northern half of the site is underlain by soils of the Positas loam series which consist of silt (ML) and clay (CL, CH) with plasticity indices between 20 and 35 percent.

The expansion potential of the near surface soils should be reevaluated by taking into account the near surface clay encountered in borings EB-1, EB-2, and EB-3 and the USDA NRCS soil mapping.

Comment 4. Page 8 recommends that additional subsurface exploration, laboratory testing and engineering analysis (additional geotechnical work) be performed during the design-level geotechnical investigation to further evaluate the potential for liquefaction-induced settlement beneath building that will straddle the fill/alluvial soil area of the site.

The additional geotechnical work should be completed prior to finalizing the project plans.

Comment 5. Page 23, Section 9, “Asphalt Concrete,” recommends using a subgrade R-Value of 20 based on the relatively sandy and low plasticity soils encountered within the upper 10 feet. This, however, seems high for the clayey soil encountered at the site.

This R-Value should be reevaluated by completing R-Value tests during the course of the additional geotechnical work and by taking into account the near surface clay encountered in borings EB-1, EB-2, and EB-3 and the USDA NRCS soil mapping.

Comment 6. After completion of a supplement or update to the Cornerstone report, we should be provided with a copy to complete a review of responses to our comments. Similarly, Cornerstone should review the project plans prepared by Humann Company to confirm that the plans have been prepared in conformance with their recommendations. Cornerstone’s review should be documented in writing and provided to the City.

CLOSURE

This review has been performed by request of the City of Martinez. Our role has been to provide technical advice to assist the City in its discretionary permit decisions, and we are afforded the same protection under state law. Our services have been limited to the review of the documents listed above, and a visual review of the property. We have no control over the future construction on this property and make no representations regarding its future conditions.

We have employed accepted engineering geology and civil and geotechnical engineering procedures, and our professional opinions and conclusions are made in accordance with generally accepted engineering geology and civil and geotechnical engineering principles and practices. This standard is in lieu of all other warranties, either expressed or implied.

Yours truly,

CAL ENGINEERING & GEOLOGY, INC.

CHRISTOPHER A. HOCKETT
No. Geo GE 2928
No. Civil C 71938

Chris Hockett, G.E. 2928
Associate Engineer

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13 June 2022

Hector Rojas, AICP
Community and Economic Development Department.
525 Henrietta Street
Martinez, California, 94553

RE: Second Geotechnical Report Peer Review
Amáre Apartment Homes
Martinez, California
CE&G Document 160350-002

Hector Rojas:

At your request, we have completed our geologic and geotechnical review of the updated preliminary geotechnical report for the proposed Amáre Apartment Homes development to be constructed between 2050 Arnold Drive and 2530 Arnold Drive in Martinez, California. Our initial review of the preliminary geotechnical report was completed in 2016. The first peer review letter is attached for completeness.

The following project documents were reviewed:

- Geotechnical Report by Cornerstone Earth Group titled, "Updated Preliminary Geotechnical Investigation, Amáre Apartment Homes, Arnold Drive, Martinez, California, The Austin Group, LLC," dated 27 May 2022, project number 1365-1-1.

- Plans by Johnson Lyman Architects, Inc. titled, "Amáre Apartment Homes, Martinez, California," dated 1 October 2021.

Our review has included examination of the above referenced documents for pertinent information regarding the technical feasibility of the project. We have also performed reconnaissance level observations of the project site and reviewed information in our files which include published soils and geologic information.

PROPOSED PROJECT

We understand that it is currently proposed to develop the property with apartment buildings and on-grade parking. From Cornerstone's report:

> The project site is located just south of the intersection of Arnold Drive and Starflower Avenue, in Martinez, California. The approximately 5.1-acre site (currently designated as APN nos. 161-400-009 & 010) is undeveloped and covered with low grasses and numerous mature trees. An apartment complex is currently planned for the site that will include six buildings.
>
> The planned 183-unit development will include tuck-under parking at the ground floor. The buildings will be of wood-frame construction and be supported on either shallow foundations or drilled piers. Appurtenant parking, retaining walls up to 8-feet high, utilities, landscaping and other improvements necessary for site development are also planned. Moderate to minor cuts and fills will be required to create level building pads. Grading will reportedly require importing up to about 13,000 cubic yards of soil to grade the site.

REVIEW OF GEOTECHNICAL REPORT

Our review of the Cornerstone updated preliminary geotechnical report revealed that the report is generally complete with respect to identifying the geotechnical conditions that will impact the project. However, we have the following comments.

Comment 1: Page 6, Section 3.3.1, "Plasticity/Expansion Potential," indicates that one plasticity index (PI) test was completed on a representative near-surface soil sample. The PI test was completed on sample number MC-1A from boring EB-4 at a depth of approximately 2 feet. The soil was classified as a silty sand (SM) having a PI of 5 percent.

Borings EB-1, EB-2, and EB-3 encountered sandy lean clay (CL) and lean clay with sand (CL) within 3 feet from the ground surface, which generally have higher plasticity indices than silty sand. In addition, surficial soils maps published by the United Stated Department of Agriculture (USDA) National Resource Conservation Service (NRCS) maps indicate that the northern half of the site is underlain by soils of the Positas loam series which consist of silt (ML) and clay (CL, CH) with plasticity indices between 20 and 35 percent.

To reduce the potential for flatwork and pavements to heave and crack, the expansion potential of the near surface soils should be reevaluated by completing Atterberg limits tests during the course of the additional geotechnical work and taking into account both the near surface clay encountered in borings EB-1, EB-2, and EB-3 and the USDA NRCS soil mapping, as well as the proposed grading or the site.

Comment 2: Page 9 and page 12 recommend that additional subsurface exploration, laboratory testing and engineering analysis (additional geotechnical work) be performed during the design-level geotechnical investigation to further evaluate the potential for liquefaction-induced settlement beneath building that will straddle the fill/alluvial soil area of the site.

The additional geotechnical work should be completed prior to finalizing the project plans.

Comment 3: Page 23, Section 9, "Asphalt Concrete," recommends using a subgrade R-Value of 20 based on the relatively sandy and low plasticity soils encountered within the upper 10 feet. This, however, seems high for the clayey soil encountered at the site.

This R-Value should be reevaluated by completing R-Value tests during the course of the additional geotechnical work and by taking into account the near surface clay encountered in borings EB-1, EB-2, and EB-3 and the USDA NRCS soil mapping, as well as the proposed grading and areas where import fill will be used.

Comment 4: After completion of a supplement or update to the Cornerstone report, we should be provided with a copy to complete a review of responses to our comments. Similarly, Cornerstone should review the project plans prepared by Johnson Lyman Architects to confirm that the plans have been prepared in conformance with their recommendations. Cornerstone's review should be documented in writing and provided to the City.

CLOSURE

This review has been performed by request of the City of Martinez. Our role has been to provide technical advice to assist the City in its discretionary permit decisions, and we are afforded the same protection under state law. Our services have been limited to the review of the documents listed above, and a visual review of the property. We have no control over the future construction on this property and make no representations regarding its future conditions.

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Yours truly,

CAL ENGINEERING & GEOLOGY, INC.

Chris Hockett, G.E. 2928
Principal Engineer

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