APPENDIX B:
GEOTECHNICAL INVESTIGATION

Geotechnical Investigation

The Hamptons Apartments
Cupertino, California

Report No. 238182 has been prepared for:

IRVINE COMPANY

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FIGURE 1 — VICINITY MAP
FIGURE 2 — SITE PLAN
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GEOTECHNICAL INVESTIGATION
THE HAMPTONS APARTMENTS
SAN JOSE, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for The Hamptons Apartments redevelopment to be constructed in Cupertino, California. The site location is shown on the Vicinity Map, Figure 1. The purpose of our investigation was to evaluate the geologic and subsurface conditions and to provide geotechnical recommendations for design of the proposed project.

As you know our predecessor Lowney Associates prepared a geotechnical report for the site titled, "Geotechnical Investigation for Pruneridge Avenue Apartments, Cupertino, California," dated January 6, 1997. Our previous borings at the site were also used to prepare this geotechnical report in addition to our current explorations.

For our use, we received architectural and civil plans titled, "The Hamptons Redevelopment, Development Application," prepared by Arquitectonica, dated May 28, 2015.

1.1 Project Description

The approximately 12½-acre site is located at 19500 Pruneridge Avenue in Cupertino, California and is currently occupied by an existing apartment complex. The layout of the proposed development is shown on the Site Plan, Figure 2. The site is bordered by the Wolfe Road exit ramp to the southwest, Wolfe Road to the west, Pruneridge Avenue to the north and northeast and commercial developments to the southeast. Based on the plans provided, the project consists of the demolition and redevelopment of the site with the construction of a 7-story residential apartment complex over one-level of below-grade vehicle parking and a portion of the site with two-levels of below-grade vehicle parking. Additional vehicle parking is also planned for level 1 (at-grade) and for the southern half of level 2 (one-story above-grade). The above-grade portion of the apartment complex will consist of Buildings A, B, C, D and E, and the Amenity Building. The structures will consist of wood-framed construction over a concrete podium. Additional improvements will include underground utilities and landscaping.

Based on the planned improvements, excavations on the order of approximately 15 and 25 are anticipated for the one-level and two-level below-grade parking garage, respectively. Structural loads have not been provided to us; therefore we assumed that structural loads will be representative for this type of construction.

1.2 Scope of Services

Our scope of services was presented in our agreement with you dated June 18, 2015. To accomplish this work, we provided the following services:

- Exploration of subsurface conditions by drilling four borings in the area of the proposed development and retrieving soil samples for observation and laboratory testing.

- Evaluation of the physical and engineering properties of the subsurface soils by visually classifying the samples and performing various laboratory tests on selected samples.

- Engineering analysis to evaluate structure foundations, site earthwork, slabs-on-grade, basement walls and pavements.

- Preparation of this report to summarize our findings and to present our conclusions and recommendations.

2.0 SITE CONDITIONS

2.1 Site Reconnaissance

Our Senior Staff Engineer performed a reconnaissance of the site on June 23, 2015. At the time of the reconnaissance, the site was occupied by the existing residential apartment buildings with partial below-grade parking garages, carport and storage structures, asphalt concrete-paved parking stalls, trees and landscaping. The site appeared relatively flat with minor grade variations for drainage purposes.

2.2 Exploration Program

Subsurface exploration was performed on June 23, 2015 using conventional, truck-mounted hollow-stem auger drilling equipment to investigate, sample, and log subsurface soils. Four hollow-stem auger exploratory borings were drilled to depths ranging from approximately 30 to 40 feet.

Our boring were backfilled in accordance with Santa Clara Valley Water District guidelines. The approximate locations of the borings are shown on the Site Plan, Figure 2. The logs of the borings and details regarding our field investigation are included in Appendix A; laboratory tests are discussed in Appendix B.

Two field infiltration tests were performed at the locations shown on Figure 2 with the hollow stem auger at a depth of approximately three feet below the ground surface. The hollow stem auger was filled with at least 1 foot of water and measurements were taken of the infiltration rate.

We previously performed a subsurface exploration at the site on December 3, 1996. Logs of the borings are included in Appendix C.

2.3 Subsurface Conditions

Current Exploration

All of our borings encountered a pavement section consisting of 2½ to 3 inches of asphalt concrete underlain by 5 to 5½ inches of aggregate base. Below the pavement sections, our borings generally encountered interbedded layers consisting of medium dense to dense clayey sand, dense to very dense poorly graded sand, and medium dense to dense poorly graded gravel to a depth ranging from approximately 16½ to 21½ feet, except for boring EB-3 which encountered stiff to hard lean clay to a depth of 16½ feet. Below depths of 16½ to 21½ feet, our borings generally encountered stiff to hard lean clay to a depth of 40 feet, the maximum depth explored, with some thin interbedded layers consisting of medium dense silty sand, medium dense clayey sand, and dense poorly graded sand.

Two field infiltration rate tests were performed near borings EB-2 and EB-4 with the hollow stem auger at a depth of three feet below the ground surface. A total of approximately 12½ and 11 inches of infiltration were measured within an hour in boring EB-2 and EB-4, respectively. Based on the test results, we estimate an infiltration rate of less than 1½ inches per hour.

December 1996 Exploration

Exploratory borings EB-1 through EB-10 drilled in 1996 generally encountered loose to very dense silty and clayey sands to a depth of about 18 feet, except for the southwestern corner of the site.

Below a depth of 18 feet, the sands were underlain by interbedded layers of dense to very dense silty and clayey sands and very stiff to hard clays to a depth of 35 feet, the maximum depth explored. The southeastern corner of the site encountered very stiff, silty and sandy clays and medium dense clayey sands to a depth of 10 feet, the maximum depth previously explored at this portion of the site.

2.4 Ground Water

Free ground water was not encountered in any of our current or previous borings at the time of drilling to the maximum depth explored depth of 40 feet. Based on the depth to historically high ground water map prepared by the California Geological Survey for the Cupertino Quadrangle (CGS, 2002), the depth to historically high ground water levels in the site vicinity is estimated to be greater than 50 feet below the ground surface. Based on the above information, we judged a ground water depth of 50 feet to be appropriate for design. Our borings were backfilled immediately after drilling. Fluctuations in the level of the ground water may occur due to variations in rainfall, underground drainage patterns, and other factors not evident at the time measurements were made.

3.0 GEOLOGIC HAZARDS

A brief qualitative evaluation of geologic hazards was made during this investigation. Our comments concerning these hazards are presented below.

3.1 Fault Rupture

The San Francisco Bay Area is one of the most seismically active regions in the United States. The significant earthquakes that occur in the Bay Area are generally associated with crustal movement along well-defined active fault zones of the San Andreas Fault system, which regionally trend in a northwesterly direction. A Regional Fault Map illustrating known active faults relative to the site is presented in Figure 3. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known formerly as a Special Studies Zone), or a Santa Clara County Fault Rupture Hazard Zone (SCC, 2002). A Regional Fault Map illustrating known active faults relative to the site is presented in Figure 3. As shown on Figure 3, no known surface expression of active faults is believed to cross the site. Fault rupture through the site, therefore, is not anticipated.

3.2 Maximum Estimated Ground Shaking

Based on Equation 11.8-1 of ASCE 7-10, a peak ground acceleration of 0.58g can be expected at the site.

3.3 Future Earthquake Probabilities

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. The U.S. Geological Survey's Working Group on California Earthquake Probabilities (WGCEP, 2014) estimates there is a 72 percent chance of at least one magnitude 6.7 earthquake occurring in the San Francisco Bay region between 2014 and 2044. This result is an important outcome of WGCEP's work because any major earthquake can cause damage throughout the region. The 1989 Loma Prieta earthquake demonstrated this potential by causing severe damage in Oakland and San Francisco, more than 50 miles from the fault epicenter.

Although earthquakes can cause damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

3.4 Liquefaction

The site is not located within an area mapped by the State of California and the Santa Clara County as having the potential for seismically induced liquefaction hazard. During cyclic ground shaking, such as earthquakes, cyclically-induced stresses may cause increased pore water pressures within the soil matrix, which results in liquefaction. Liquefied soil may lose shear strength that may lead to large shear deformations and/or flow failure (Youd et al., 2001). Liquefied soil can also settle as pore pressures dissipate following an earthquake. Limited field data is available on this subject; however, settlement on the order of 2 to 3 percent of the thickness of the liquefied zone has been measured in some cases.

Soils most susceptible to liquefaction are loose to moderately dense, saturated, non-cohesive soils with poor drainage, such as sands and silts with interbedded or capping layers of relatively low permeability soil.

Groundwater was not encountered in our explorations to a depth of 40 feet and CGS estimates depth to historically high ground water levels in the site vicinity to be greater than 50 feet below the ground surface. Therefore, we judge the risk of liquefaction at the project site to be low.

3.5 Dry Seismic Settlement

If near-surface soils vary in composition both vertically and laterally, strong earthquake shaking can cause non-uniform densification of loose to medium dense cohesionless soil strata. This results in movement of the near-surface soils. Our explorations encountered some medium dense clayey sand, silty sand, and poorly graded gravel layers at various depths. We understand that the entire site will have one-level of below-grade vehicle parking with a portion of the site having two-levels of below-grade parking. Therefore, we estimated dry seismic settlements based on the anticipated excavation depths for the construction of the below-grade parking. We estimate dry seismic settlement of the medium dense stratum to be on the order of ½-inch.

3.6 Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or “free” face such as an open body of water, channel, or excavation. In soils this movement is generally due to failure along a weak plane, and may often be associated with liquefaction. As cracks develop within the weakened material, blocks of soil displace laterally towards the open face. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free. Because of the low potential for liquefaction, we judge the risk of lateral spreading at the site to be low.

4.0 CORROSION EVALUATION

To evaluate the corrosion potential of the subsurface soils at the site, we submitted four samples collected during our subsurface investigation to an analytical laboratory for pH, resistivity, soluble sulfate and chloride content testing. The results of these tests are summarized in Table 1 below.

Table 1. Results of Corrosivity Testing

Sample	Depth (feet)	Chloride (mg/kg)	Sulfate (mg/kg)	pH	Resistivity (ohm-cm)	Estimated Corrosivity Based on Resistivity	Estimated Corrosivity Based on Sulfates
EB-1, 2A	3.5	6	6	7.7	8,824	Mildly	Negligible
EB-3, 4A	9.0	5	61	7.6	1,528	Severely	Negligible
EB-3, 5A	14.0	80	86	7.9	1,528	Severely	Negligible
EB-4, 3A	6.0	26	70	7.7	3,455	Moderately	Negligible

Notes: 1. mg/kg = milligrams per kilogram.

Many factors can affect the corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on classification developed by William J. Ellis (1978), the approximate relationship between soil corrosiveness was developed as shown in Table 2 below.

Table 2. Relationship Between Soil Resistivity and Soil Corrosivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very Severely Corrosive
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Chloride and sulfate ion concentrations and pH appear to play secondary roles in affecting corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried metallic improvements or reinforced concrete structures. Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to Portland cement concrete (PCC) by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. Soils containing high sulfate content could also cause corrosion of the reinforcing steel in concrete. Table 4.2.1 of the American Concrete Institute (ACI, 2008) provides requirements for concrete exposed to sulfate-containing solutions as summarized in Table 3.

Table 3. Relationship Between Sulfate Concentration and Sulfate Exposure

Water-Soluble Sulfate (SO_4) in soil, ppm	Sulfate Exposure
0 to 1,000	Negligible
1,000 to 2,000	Moderate[1]
2,000 to 20,000	Severe
over 20,000	Very Severe

[1]= seawater

Acidity is an important factor of soil corrosivity. The lower the pH (the more acidic the environment), the higher will the soil corrosivity be with respect to buried metallic structures. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures due to protective surface films which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint.

As shown in Table 1, the soil resistivity results ranged from 1,528 to 8,824 ohm-centimeters. Based on these results and the resistivity correlations presented in Table 2, the corrosion potential to buried metallic improvements may be characterized as mildly to severely corrosive. We recommend that a corrosion protection engineer be consulted about appropriate corrosion protection methods for buried metallic materials.

Based on our previous experience and Table 4.2.1 of the ACI, it is our opinion that sulfate exposure to PCC may be considered negligible to moderate for the native subsurface materials sampled.

5.0 CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical engineering viewpoint, the proposed development may be constructed as planned, in our opinion, provided the design and construction are performed in accordance with the recommendations presented in this report.

5.1 Primary Geotechnical Concerns

The primary geotechnical and geologic concerns at the site are as follows:

- Strong seismic shaking
- Demolition of the existing buildings prior to site development
- Corrosion potential of the near-surface soils
- Basement excavation support

We have prepared a brief description of the issues and present typical approaches to manage potential concerns associated with the long-term performance of the development.

5.1.1 Strong Seismic Shaking

We recommend that, at a minimum, the proposed apartment development be designed in accordance with the seismic design criteria as discussed in the Maximum Estimated Ground Shaking section above, and the site seismic coefficients presented in Table 5.

5.1.2 Demolition Debris

Construction debris is anticipated as a result of the site demolition required prior to site grading. The debris should be either: 1) collected and off-hauled to an appropriate facility prior to beginning the earthwork for the project, or 2) the concrete crushed and re-used as fill at the site. If generated, recycled materials containing asphalt concrete (AC) should not be used below interior floor slabs, therefore if recycled materials are proposed to be re-used beneath interior floor slabs, AC pavements should be segregated from the debris. It has been our experience that some debris will remain in the soil on-site after the demolition contractor has completed their work. Therefore, it should be anticipated that some debris will be encountered in shallow depth excavations for underground utilities and foundations. Some coordination between the demolition contractor, grading contractor and geotechnical engineer is needed to identify the scope of the excavation backfill and other similar work items. Recommendations for re-use of recycled materials are presented in the Earthwork section of this report.

5.1.3 Corrosion Potential of Near-Surface Soils

As discussed above, the corrosion potential to buried metallic improvements constructed within the native soils may be characterized as mildly to severely corrosive. A qualified corrosion engineer should be contacted to provide specific recommendations regarding corrosion protection for buried metal pipe or buried metal pipe-fittings.

5.1.4 Basement Excavation Support

The walls of the basement excavation may be supported by several methods including tiebacks, soldier beams and wood lagging or temporary slopes if space is adequate. The choice should be left to the contractor's judgment since economic considerations and/or the individual contractor's construction experience may determine which method is more economical and/or appropriate. Support of any adjacent existing structures without distress should also be the contractor's responsibility. We recommend that the contractor forward their plan for the support system to the structural engineer and geotechnical engineer for pre-construction review. In addition, it should be the contractor's responsibility to undertake a pre-construction survey with benchmarks and photographs of the adjacent properties as well as to conduct periodic monitoring.

5.2 Plans, Specifications, and Construction Review

We recommend that our firm perform a plan review of the geotechnical aspects of the project design for general conformance with our recommendations. In addition, subsurface materials encountered in the relatively small diameter, widely spaced borings may vary significantly from other subsurface materials on the site. Therefore, we also recommend that a representative of our firm observe and confirm the geotechnical specifications of the project construction. This will allow us to form an opinion about the general conformance of the project plans and construction with our recommendations. In addition, our observations during construction will enable us to note subsurface conditions that may vary from the conditions encountered during our investigation and, if needed, provide supplemental recommendations. For the above reasons, our geotechnical recommendations are contingent upon our firm providing geotechnical observation and testing services during construction.

6.0 EARTHWORK

6.1 Clearing and Site Preparation

The proposed project area should be cleared of all surface and subsurface improvements to be removed and deleterious materials including existing building foundations, slabs, irrigation lines, utilities, fills, pavements, debris, designated trees, shrubs, and associated roots. Abandonment of existing buried utilities is discussed below. Excavations extending below the planned finished site grades should be cleaned and backfilled with suitable material compacted as recommended in the "Compaction" section of this report. We recommend that backfilling of holes or pits resulting from demolition and removal of existing building foundations, buried structures or other improvements be carried out under our observation and that the backfill be observed and tested during placement.

After clearing, any vegetated areas within the proposed improvements should be stripped to sufficient depth to remove all surface vegetation and topsoil containing greater than 3 percent organic matter by weight. The actual stripping depth required depends on site usage prior to construction and should be established in the field by us at the time of construction. The stripped materials should be removed from the site or may be stockpiled for use in landscaped areas, if desired.

6.2 Removal of Undocumented Fill

If undocumented fill is encountered, it should be removed down to native soil. If the fill material meets the requirements in the “Material for Fill” section below, it may be reused as engineered fill. Side slopes of fill removal excavations in building and pavement areas should be sloped at inclinations no steeper than 3:1 (horizontal:vertical) to minimize abrupt variations in fill thickness. All fill should be compacted in accordance with the recommendations for fill presented in the “Compaction” section of this report.

6.3 Abandoned Utilities

Abandoned utilities within the proposed building area should be removed in their entirety. Utilities within the proposed building area would only be considered for in-place abandonment provided they do not conflict with new improvements, if the ends and all laterals are located and completely grouted, and the previous fills associated with the utility do not pose a risk to the structure.

Utilities outside the building area should be removed or abandoned in-place by grouting or plugging the ends with concrete. Fills associated with utilities abandoned in-place could pose some risk of settlement; utilities that are plugged could also pose some risk of future collapse or erosion should they leak or become damaged.

6.4 Subgrade Preparation

After the site has been properly cleared, stripped and necessary excavations have been made, exposed surface soils in those areas to receive fill or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the recommendations for fill presented in the “Compaction” section. The finished compacted subgrade should be firm and non-yielding under the weight of compaction equipment.

6.5 Material for Fill

All on-site soils below the stripped layer having an organic content of less than 3 percent by weight are suitable for use as fill at the site. In general, fill material should not contain rocks or lumps larger than 6 inches in greatest dimension, with 15 percent or less larger than 2½ inches in the greatest dimension.

Import fill material should be inorganic, have a PI of 20 or less and should have sufficient binder to reduce the potential for sidewall caving of foundation and utility trenches. Non-expansive fill (NEF) should have a PI of 15 or less. Samples of the proposed import fill should be submitted to us at least 10 working days prior to delivery to the site to allow for visual review and laboratory testing. This will allow us to evaluate the general conformance of the import fill with our recommendations.

Consideration should also be given to the environmental characteristics and corrosion potential of any imported fill. Suitable documentation should be provided for import material. In addition, it may be appropriate to perform laboratory testing of the environmental characteristics and corrosion potential of imported materials. Import soils should not be more corrosive than the on-site native materials, including pH, soluble sulfates, chlorides and resistivity.

6.6 Reuse of On-site Recycled Materials

Asphalt concrete/aggregate base grindings may be generated during removal of any existing pavements. If it is desired to reuse the grindings for new site pavement structural support, we recommend the asphalt concrete be pulverized and mixed with the underlying aggregate base to meet Caltrans Class 2 Aggregate Base requirements. If laboratory testing of the recycled material indicates that it meets Caltrans Class 2 specifications, it may be used as Class 2 Aggregate Base beneath pavements and sidewalks. Recycled material containing asphalt concrete grindings should not be used below building areas. Laboratory testing may be performed on initial grindings generated to evaluate the material further and refine the pavement recommendations.

6.7 Compaction

All fill, as well as scarified surface soils in those areas to receive fill, should be uniformly compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D1557, latest edition, at a moisture content near the laboratory optimum. Fill should be placed in lifts no greater than 8 inches in uncompacted thickness. Each successive lift should be firm and relatively non-yielding under the weight of construction equipment.

In pavement areas, the upper 6 inches of subgrade and full depth of aggregate base should be compacted to at least 95 percent relative compaction (ASTM D1557, latest edition). Aggregate base and all import soils should be compacted at a moisture content near the laboratory optimum moisture content.

6.8 Wet Soils and Wet Weather Conditions

Earthwork such as subgrade preparation, fill placement and trench backfill may be difficult for soil containing high moisture content or during wet weather. The contractor should be aware that soils at the bottom of the excavation may be contain high moisture content. If the soil is significantly above its optimum moisture content, it will become soft, yielding, and difficult to compact. If saturated soils are encountered, aerating or blending with drier soils to achieve a workable moisture content may be required. We recommend that earthwork be performed during periods of suitable weather conditions, such as the “summer” construction season.

There are several alternatives to facilitate subgrade preparation, fill placement and trench backfill if the soil is wet or earthwork is performed during the wet winter season.

- Scarify and air dry until the fill materials have a suitable moisture content for compaction,
- Over-excavate the fill and replace with suitable on-site or import materials with an appropriate moisture content,
- Install a layer of geo-synthetic (geotextile or geogrid) to reduce surface yielding and bridge over soft fill,
- Chemically treat the higher moisture content soils with quicklime (CaO), kiln-dust, or cement to reduce the moisture content and increase the strength of the fill.

The implementation of these methods should be reviewed on a case-by-case basis so that a cost effective approach may be used for the specific conditions at the time of construction.

6.9 Trench Backfill

Bedding and pipe embedment materials to be used around underground utility pipes should be well graded sand or gravel conforming to the pipe manufacturer’s recommendations and should be placed and compacted in accordance with project specifications, local requirements of the governing jurisdiction. General fill to be used above pipe embedment materials should be placed and compacted in accordance with local requirements or the recommendations contained in this section, whichever is more stringent.

On-site soils may be used as general fill above pipe embedment materials provided they meet the requirements of the “Material for Fill” section of this report. General fill should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D1557, latest edition) by mechanical means only. Water jetting of trench backfill should not be allowed. The upper 6 inches of general fill in all pavement areas subject to wheel loads should be compacted to at least 95 percent relative compaction.

Utility trenches located adjacent to footings should not extend below an imaginary 1:1 (horizontal:vertical) plane projected downward from the footing bearing surface to the bottom edge of the trench. Where utility trenches will cross beneath footing bearing planes, the footing concrete should be deepened to encase the pipe or the utility trench should be backfilled with sand/cement slurry or lean concrete within the foundation-bearing plane.

Where relatively higher permeability sand or gravel backfill is used in trenches through lower permeability soils, we recommend that a cut-off plug of compacted clayey soil or a 2-sack cement/sand slurry be placed where such trenches enter the building and pavement areas. This would reduce the likelihood of water entering the trenches from the landscaped areas and seeping through the trench backfill into the building and pavement areas, and coming into contact with very highly expansive subgrade soils.

6.10 Temporary Slopes and Trench Excavations

The contractor should be responsible for all temporary slopes and trenches excavated at the site and design of any required temporary shoring. Shoring, bracing, and benching should be performed by the contractor in accordance with the strictest governing safety standards. On a preliminary basis, site soils can be classified as Type B based on soil classification by OSHA. A TRC representative should be retained to verify soil conditions in the field at the time of the excavation.

6.11 Temporary Shoring Support System

As previously discussed, excavations on the order of approximately 15 and 25 feet (as measured from existing exterior grades) are planned to construct the one-level and two-level below-grade parking garages, respectively. The excavations could potentially be temporarily supported by several methods including tiebacks, soil nailing, braced shoring, temporary slopes if space is adequate or other methods. Where shoring is required, restrained shoring will most likely be necessary to limit deflections and disruption to nearby improvements. It has been our experience that cantilever shoring might be feasible for temporary shoring to a height of about 10 to 13 feet where allowable deflections are limited. The choice of shoring method should be left to the contractor’s judgment since economic considerations and/or the individual contractor’s construction experience may determine which method is more economical and/or appropriate. However, other factors such as the location of nearby utilities and encroachment on adjacent properties may influence the choice of support.

The temporary shoring should be designed for additional surcharges due to adjacent loads such as from construction vehicles and street traffic. To prevent excessive surcharging of the walls, we recommend that heavy loads such as construction equipment and stockpiles of materials be kept at least 15 feet from the top of the excavations. If this is not possible, the shoring must be designed to resist the additional anticipated lateral loads. Shoring systems should be designed with sufficient rigidity to prevent detrimental lateral displacements. Minimum geotechnical parameters for design of a temporary shoring system are given in Table 4.

Table 4. Temporary Shoring System Design Parameter

Design Parameter	Design Value (psf)
Minimum Lateral Wall Surcharge[1]	120 psf
Earth Pressure – Cantilever Wall	40 pcf
Earth Pressure – Restrained Wall[2]	
From ground surface to H/4 (ft)	Increase from 0 to 25H psf
Below H/4 (ft)	Uniform pressure of 25H psf
Passive Pressure[3]	300 pcf up to 2,000 psf max

Note: 1 For the upper 5 feet (minimum for incidental loading)
2 Where H equals height of excavation
3 Can assume to act over 2 times the diameter of soldier piles, neglecting the upper foot

To limit potential movements of the shoring system, the shoring designer and contractor should consider several design and construction issues. For the movements of shoring to be reduced, the designer will have to provide for a uniform and timely mobilization of the soil pressures. Tiebacks or internal bracing should be loaded to the design loads prior to excavation of the adjacent soil so that load induced strains in the retaining system will not result in the system moving toward the excavation. In addition, a relatively stiff shoring system should be designed to limit deflections under loading. In general, we recommend designing a shoring system to deflect less than 1-inch.

In addition, ground subsidence and deflections can be caused by other factors such as voids created behind the shoring system by over-excavation, soil sloughing, erosion of sand or silt layers due to perched water, etc. All voids behind the shoring system should be filled as soon as feasible by grouting to minimize potential problems during installation of the shoring system.

Since we drilled our borings with hollow-stem auger drilling equipment, we are not able to evaluate the potential for caving of on-site soils, which may become a factor during soldier pile and/or tieback installation. The contractor is responsible for evaluating excavation difficulties prior to construction. Pilot holes using proposed production drilling equipment may be prudent, to evaluate possible excavation difficulties such as caving soils, cobbles, boulders and/or other excavation difficulties.

In conjunction with the shoring installation, a monitoring program should be set up and carried out by the contractor to determine the effects of the construction on the adjacent buildings, street and other improvements such as sidewalks and utilities. At a minimum, we recommend horizontal and vertical surveying of reference points on the shoring and on the adjacent street, buildings and other improvements in addition to an initial crack survey. We also recommend that all supported and/or sensitive utilities be located and monitored by the contractor. Reference points should be set up and read prior to the start of construction activities. Points should also be set on the shoring as soon as initial installations are made. Alternatively, inclinometers could be installed by the contractor at critical locations for a more detailed monitoring of shoring deflections. Surveys should be made at least once a week and more frequently during critical construction activities, or if significant deflections are noted. TRC can provide inclinometer materials and we have the equipment and software to read and analyze the data quickly.

This report is intended for use by the design team. The contractor should perform additional subsurface exploration and/or geotechnical studies as they deem necessary for the chosen shoring system. The contractor is also responsible for site safety and the means and methods of construction, including temporary shoring. Temporary shoring must be designed by a licensed California Civil or Structural Engineer. Prior to construction, we recommend that the contractor forward his plan for the support system to the structural engineer and geotechnical engineer for preconstruction review.

6.12 Surface Drainage

Positive surface water drainage gradients, at least 2 percent in landscaping and 0.5 percent in pavement areas, should be provided to direct surface water away from foundations and slabs towards suitable discharge facilities. Ponding of surface water should not be allowed on or adjacent to structures, slabs-on-grade, or pavements. Roof runoff should be directed away from foundation and slabs-on-grade. Downspouts may discharge onto splash-blocks provided the area is covered with concrete slabs or asphalt concrete pavements.

6.13 Landscaping Considerations

We recommend restricting the amount of surface water infiltrating these soils near structures and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering, especially within 3 feet of structures, slabs-on-grade, or pavements,
- Using low flow rate sprinkler heads, or preferably, drip irrigation systems
- Regulating the amount of water distributed to lawn or planter areas by installing timers on the sprinkler system,
- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements,
- Preventing water from draining toward or ponding near building foundations, slabs-on-grade, or pavements, and
- Avoiding open planting areas within 3 feet of the building perimeters.

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

6.14 Construction Observation

A representative from our company should observe the geotechnical aspects of the grading and earthwork for general conformance with our recommendations including site preparation, selection of fill materials, and the placement and compaction of fill. To facilitate your construction schedule we request sufficient notification (48 hours) for site visits. The project plans and specifications should incorporate all recommendations contained in the text of this report.

7.0 FOUNDATIONS

Based on our investigation, the proposed structures may be supported on shallow foundations as discussed below.

7.1 2013 CBC Site Coefficients and Site Seismic Coefficients

Chapter 16 of the 2013 California Building Code (CBC) outlines the procedure for seismic design of structures. Based on our explorations, the site is generally underlain by stiff to hard clays and medium dense to very dense sands and gravels, which corresponds to a soil profile type D. Based on the above information and local seismic sources, the site may be characterized for design using the information in Table 5 below.

Table 5. 2013 CBC Site Class and Site Seismic Coefficients

Latitude: 37.3328 N **Longitude: 122.0128 W**	**CBC Reference**	**Factor/ Coefficient**	**Value**
Soil Profile Type	Section 1613.3.2	Site Class	D
Mapped Spectral Response Acceleration for MCE at 0.2 second Period	Figure 1613.3.1(1)	S_s	1.55
Mapped Spectral Response Acceleration for MCE at 1 Second Period	Figure 1613.3.1(2)	S_1	0.63
Site Coefficient	Table 1613.3.3(1)	F_a	1.0
Site Coefficient	Table 1613.3.3(2)	F_v	1.5
Adjusted MCE Spectral Response Parameter	Equation 16-37	S_{MS}	1.55
Adjusted MCE Spectral Response Parameter	Equation 16-38	S_{M1}	0.94
Design Spectral Response Acceleration Parameter	Equation 16-39	S_{DS}	1.03
Design Spectral Response Acceleration Parameter	Equation 16-40	S_{D1}	0.63

7.2 Footings

The proposed apartment structures may be supported on conventional continuous and/or isolated spread footings bearing on natural, undisturbed soil or compacted fill. All footings should have a minimum width of 18 inches, and the bottom of footings should extend at least 24 inches below lowest adjacent finished grade. Lowest adjacent finished grade may be taken as the bottom of interior slab-on-grade or the finished exterior grade, excluding landscape topsoil, whichever is lower.

Footings constructed in accordance with the above recommendations would be capable of supporting maximum allowable bearing pressures of 3,000 pounds per square foot (psf) for dead loads, 4,500 psf for combined dead and live loads, and 6,000 psf for all loads including wind or seismic. These allowable bearing pressures are based upon factors of safety of 3.0, 2.0 and 1.5 for dead, dead plus live, and seismic loads, respectively.

These maximum allowable bearing pressures are net values; the weight of the footings may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

All continuous footings should be reinforced with top and bottom steel to provide structural continuity and to help span local irregularities. It is essential that we observe all footing excavations before reinforcing steel is placed.

7.2.1 Footing Settlement

Structural loads were not available for our review at the time of our investigation. Therefore, we assumed typical interior column dead plus live loads on the order of 800 kips. Based on these assumed loads and the maximum allowable bearing pressures recommended above, we estimate that total footing settlement should be approximately 1-inch with post-construction differential movement between adjacent columns of approximately ½-inch. Seismically induced settlements in the dry sands on the order of ½-inch should also be considered in design. We should be retained to review the final foundation plans and structural loads to verify the above settlement estimates.

7.2.2 Lateral Loads

Lateral loads may be resisted by friction between the bottom of footings and the supporting subgrade. A maximum allowable frictional resistance of 0.30 may be used for design. In addition, lateral resistance may be provided by passive pressures acting against footings poured neat against competent soil. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot (pcf) be used in design. The upper 12 inches of soil should be neglected when calculating lateral passive resistance unless covered by concrete slabs or pavements.

7.3 Reinforced Mat Foundations

As an alternative to footings, the proposed structures may be supported on a conventionally reinforced mat foundation. Based on the subsurface conditions, the mat may be designed for an average allowable bearing pressure of 1,500 pounds per square foot (psf) for dead plus live loads with maximum localized allowable bearing pressures of 3,000 psf at column or wall loads. Allowable bearing pressures may be increased by one-third for all loads including wind or seismic. These allowable bearing pressures are net values; the weight of the mat can be neglected for design purposes.

The mat should be reinforced with top and bottom steel, as appropriate, to provide structural continuity and to permit spanning of local irregularities. These recommendations may be revised depending on the particular design method selected by the structural engineer. It is essential that we observe the subgrade of the mat foundation prior to placement of reinforcing steel.

7.3.1 Mat Foundation Settlement

Our calculations for the apartment structures with a reinforced mat foundation designed for an average allowable bearing pressure of 1,500 psf for dead plus sustained live loads indicate static settlement of about ½-inch with differential settlement of approximately ¼-inch in 50 horizontal feet for a mat bearing approximately 25 feet below the existing grade. As discussed above, seismically induced settlements in the dry sands on the order of ½-inch should also be considered in design.

We assume that the ramps into the below-grade garage will be within the footprint of the reinforced mat foundation. In addition we assume that the handicap access will be structurally supported. However, there may be differential settlement between the structurally supported ramps and walkways and adjacent flatwork. We recommend structurally supporting flatwork adjacent to the building for a span of at least 5 feet laterally from the building or a hinge slab or other method should be used to accommodate portions of the structures that will be supported on different materials.

7.3.2 Modulus of Subgrade Reaction

For structural design of the mat, we recommend using a subgrade modulus that models the soil response under building loads. In developing the appropriate modulus of subgrade reaction (referred to as the “subgrade modulus”), we considered the varying soil conditions and stress distribution for the planned building layout. Based on the bearing pressure and settlements given above, for the proposed apartment structures we recommend a modulus of subgrade reaction of 40 pounds per cubic inch (pci).

We would be pleased to provide supplemental consultation in refining the soil subgrade modulus value, if desired. In order to proceed with further analysis, we would need the output from the first iteration of the SAFE analysis or other finite element analysis of the mat soil structure interaction.

7.3.3 Lateral Loads

Lateral loads may be resisted by friction between the bottom of mats and the supporting subgrade. A maximum allowable frictional resistance of 0.3 may be used for design. In addition, lateral resistance may be provided by passive pressures acting against deepened mat edges poured neat against competent soil. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot (pcf) be used in design.

7.4 Moisture Protection Considerations

Based on the planned excavation depths, moisture vapor through slabs-on-grade or matsshould be anticipated. If moisture vapor will be detrimental to the project, the structure should either be waterproofed or the recommendations below should be incorporated into project design.

Since the long-term performance of concrete slabs-on-grade and mats depends to a large degree on good design, workmanship, and materials, the following general guidelines are presented for consideration by the developer, design team, and contractor. The purpose of these guidelines is to aid in producing a concrete slab of sufficient quality to allow successful installation of floor coverings and reduce the potential for floor covering failures due to moisture-related problems associated with the slab-on-grade construction. These guidelines may be supplemented, as necessary, based on the specific project requirements.

- A minimum 15-mil thick vapor barrier meeting minimum ASTM E 1745, Class A requirements should be placed directly below the mat slab foundation. The vapor barrier should extend to the edge of the slab and should be sealed at all seams and penetrations.
- The concrete water/cement ratio should not exceed 0.45. Midrange plasticizers could be used to facilitate concrete placement and workability.
- Water should not be added after initial batching, unless the slump of the concrete is less than specified, and the resulting water/cement ratio will not exceed 0.45.
- Polishing the concrete surface with metal trowels should not be permitted.
- All concrete surfaces to receive any type of floor covering should be moist-cured for a minimum of 7 days. Moist curing methods may include frequent sprinkling, or using coverings such as burlap, cotton mats, or carpet. The covering should be placed as soon as the concrete surface is firm enough to resist surface damage. The covering should be kept continuously wet and not allowed to dry out during the required curing period.
- Water vapor emission levels and pH should be determined before floor installation as required by the manufacturer of the floor covering materials. Measurements and calculations should be made according to ASTM F1869-98 and F710-98 protocol.

The guidelines presented above are based on information obtained from various technical sources, including the American Concrete Institute (ACI), and are intended to present information that can be used to reduce potential long-term impacts from slab moisture infiltration. It should be noted that the application of these guidelines does not affect the geotechnical aspects of the foundation performance.

7.5 Garage Floor Slabs

The above and below grade parking garage slabs should be at least 5 inches thick, have a compressive strength of at least 3,000 pounds per square inch (psi), and supported on at least 6 inches of Class 2 aggregate base compacted to at least 95 percent relative compaction. Adequate slab reinforcement should be provided to satisfy the anticipated use and loading requirements.

If desired to limit moisture rise through garage slabs, the guidelines presented in the “Moisture Protection Considerations” Section should be considered.

7.6 Waterproofing

We recommend that a waterproofing specialist design the waterproofing system, including the under-mat area and all below-grade walls. A rat slab could be poured over the sugrade to protect the water-proofing as reinforcing steel is placed.

7.7 Differential Settlement for Utility Tie-ins

The utilities entering the structure could experience differential settlement specifically at the tie-in locations. We recommend emergency shut-off valves and flexible utility and piping connections that could accommodate at least two inches of vertical and horizontal movement.

8.0 BASEMENT WALLS

8.1 Lateral Earth Pressures

Any proposed retaining walls should be designed to resist lateral earth pressures from adjoining natural materials, backfill, and surcharge loads. Provided that adequate drainage is provided as recommended below, we recommend that unrestrained walls be designed to resist an equivalent fluid pressure of 45 pcf plus a uniform pressure of 8H pounds per square foot, where H is the distance in feet between the bottom of the footing and the top of the retained soil. Unrestrained walls should also be designed to resist an additional uniform pressure equivalent to one-third of any surcharge loads applied at the surface.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent build-up of hydrostatic pressure from surface water infiltration and/or a rise in the ground water level. If adequate drainage is not provided, we recommend an equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp proofing of the walls should be included in areas where wall moisture and efflorescence would be undesirable.

8.2 Seismic Lateral Earth Pressures

We understand the basement walls may be designed for seismic lateral loading. For our analysis, we have assumed that the walls will have flat, non-sloping backfill. We used the Mononobe-Okabe approach to approximate the increased earth pressures induced by earthquakes. As discussed in Section 3.2 of our report, a peak ground acceleration of 0.58g is expected at the site. We performed calculations using this ground acceleration, and estimated an additional seismic increment of $20H^2$ for fixed walls. This seismic increment is a resultant applied to the wall in addition to the static lateral earth pressures given in Section 8.1. For fixed walls the additional seismic load would be applied as a uniform pressure with the resultant applied at mid-height.

8.3 Drainage

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

Miradrain, Geotech Drainage Panels, or Enkadrain drainage matting may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. The drainage panel should be connected to the perforated pipe at the base of the wall, or to some other closed or through-wall system. Miradrain panels should terminate 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

8.4 Backfill

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

8.5 Foundation

Reinforced concrete retaining walls not located on or adjacent to slopes may be supported on continuous spread footings or mats designed in accordance with the recommendations presented in the “Foundations” section of this report. Lateral load resistance for the walls may be developed in accordance with the recommendations presented in the “Lateral Loads” section.

9.0 PAVEMENTS

9.1 Asphalt Concrete

During our previous geotechnical investigation, we obtained a representative bulk sample of the surface soil from the parking area and performed an R-value test to provide data for pavement design for at-grade pavements. The results of the test are included in Appendix B and indicate an R-value of 57. Because surface soils vary across the site, we judged an R-value of 30 to be applicable for design based on a subgrade consisting of untreated native soils. Using estimated traffic indices for various pavement-loading requirements and untreated native soils, we developed the following recommended pavement sections based on Procedure 608 of the Caltrans Highway Design Manual, presented in Table 6.

Table 6. Recommended Asphalt Concrete Pavement Design Alternatives
Pavement Components
Design R–Value = 30

General Traffic Condition	Design Traffic Index	Asphalt Concrete (Inches)	Aggregate Baserock* (Inches)	Total Thickness (Inches)
Automobile Parking	4.0	2.5	5.0	7.5
	4.5	2.5	6.0	8.5
Automobile Parking Channel	5.0	3.0	6.0	9.0
	5.5	3.0	7.0	10.0
Truck Access & Parking Areas	6.0	3.5	8.0	11.5
	6.5	4.0	10.0	14.0

*Caltrans Class 2 aggregate base; minimum R-value equal to 78.

The traffic indices used in our pavement design are considered reasonable values for the proposed development and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. The traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel load analysis or a traffic study.

Because the full thickness of asphalt concrete is frequently not placed prior to construction traffic being allowed to use the streets (or parking lots), rutting and pavement failures can occur prior to project completion. To reduce this occurrence, we recommend that either the full design pavement section be placed prior to use by construction traffic, or a higher Traffic Index (TI) be specified where construction traffic will use the pavement.

9.2 Exterior Portland Cement Concrete (PCC) Pavements

Recommendations for exterior PCC pavements are presented below in Table 7. Since the expected Average Daily Truck Traffic (ADTT) is not known at this time, we have provided alternatives for minimum pavement thickness. An allowable ADTT should be chosen that is greater than expected for the development.

Table 7. Recommended Minimum PCC Pavement Thickness

Allowable ADTT	Minimum PCC Pavement Thickness (inches)
0.8	5
13	5½
130	6

Our design is based on an R-value of 30 and a 28-day unconfined compressive strength for concrete of at least 3,500 pounds per square inch. In addition, our design assumes that pavements are restrained laterally by a concrete shoulder or curb and that all PCC pavements are underlain by at least 6 inches of Class 2 aggregate base. We recommend that adequate construction and control joints be used in design of the PCC pavements to control the cracking inherent in this construction.

9.3 Pavement Cutoff

Surface water infiltration beneath pavements could significantly reduce the pavement design life. While the amount of reduction in pavement life is difficult to quantify, in our opinion, the normal design life of 20 years may be reduced to less than 10 years. Therefore, long-term maintenance greater than normal may be required.

To limit the need for additional long-term maintenance, it would be beneficial to protect at-grade pavements from landscape water infiltration by means of a concrete cut-off wall, deepened curbs, redwood header, “Deep-Root Moisture Barrier,” or equivalent. However, if reduced pavement life and greater than normal pavement maintenance are acceptable, the cutoff barrier may be eliminated. If desired to install pavement cutoff barriers, they should be considered where pavement areas lay downslope of any landscape areas that are to be sprinkled or irrigated, and should extend to a depth of at least 4 inches below the base rock layer.

9.4 Asphalt Concrete, Aggregate Base and Subgrade

Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of Caltrans Standard Specifications, latest edition, except that ASTM Test Designation D1557 should be used to determine the relative compaction of the aggregate base. Pavement subgrade should be prepared and compacted as described in the “Earthwork” section of this report.

9.5 Flatwork and Sidewalks

We recommend that exterior slabs-on-grade, such as flatwork and sidewalks be at least 4 inches thick and be underlain by at least 4 inches of Class 2 aggregate base compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D1557, latest edition. If sidewalks are subject to wheel loads, they should be designed in accordance with the “Exterior Portland Cement Concrete Pavements” section of this report.

We recommend that exterior slabs be isolated from adjacent foundations and that adequate construction and control joints be used in design of the concrete slabs to control cracking inherent in concrete construction.

10.0 STORMWATER CONTROL

Two field infiltration rate tests were performed along the proposed fire lane access road with the hollow stem auger at a depth of three feet below the ground surface. Based on the test result and Table 7-1 of Part 630 of the United States Department of Agriculture National Engineering Handbook (USDA 2009), we recommend that stormwater control measures be designed based on hydrologic soil group A for depth to high ground water table of greater than 100 centimeters.

11.0 LIMITATIONS

This report has been prepared for the sole use of the Irvine Company, specifically for design of the proposed The Hamptons Apartments development in Cupertino, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was written. No other warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our investigation, which includes data from widely separated discrete locations, visual observations from our site reconnaissance, and review of other geotechnical data provided to us, along with local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between the borings do not deviate substantially from those encountered or extrapolated from the information collected during our investigation. We are not responsible for the data presented by others.

We should be retained to review the geotechnical aspects of the final plans and specifications for conformance with our recommendations. The recommendations provided in this report are based on the assumption that we 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, TRC cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of TRC's report by others. Furthermore, TRC will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services and/or at the time another consultant is retained for follow up service to this report.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of the property will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

12.0 REFERENCES

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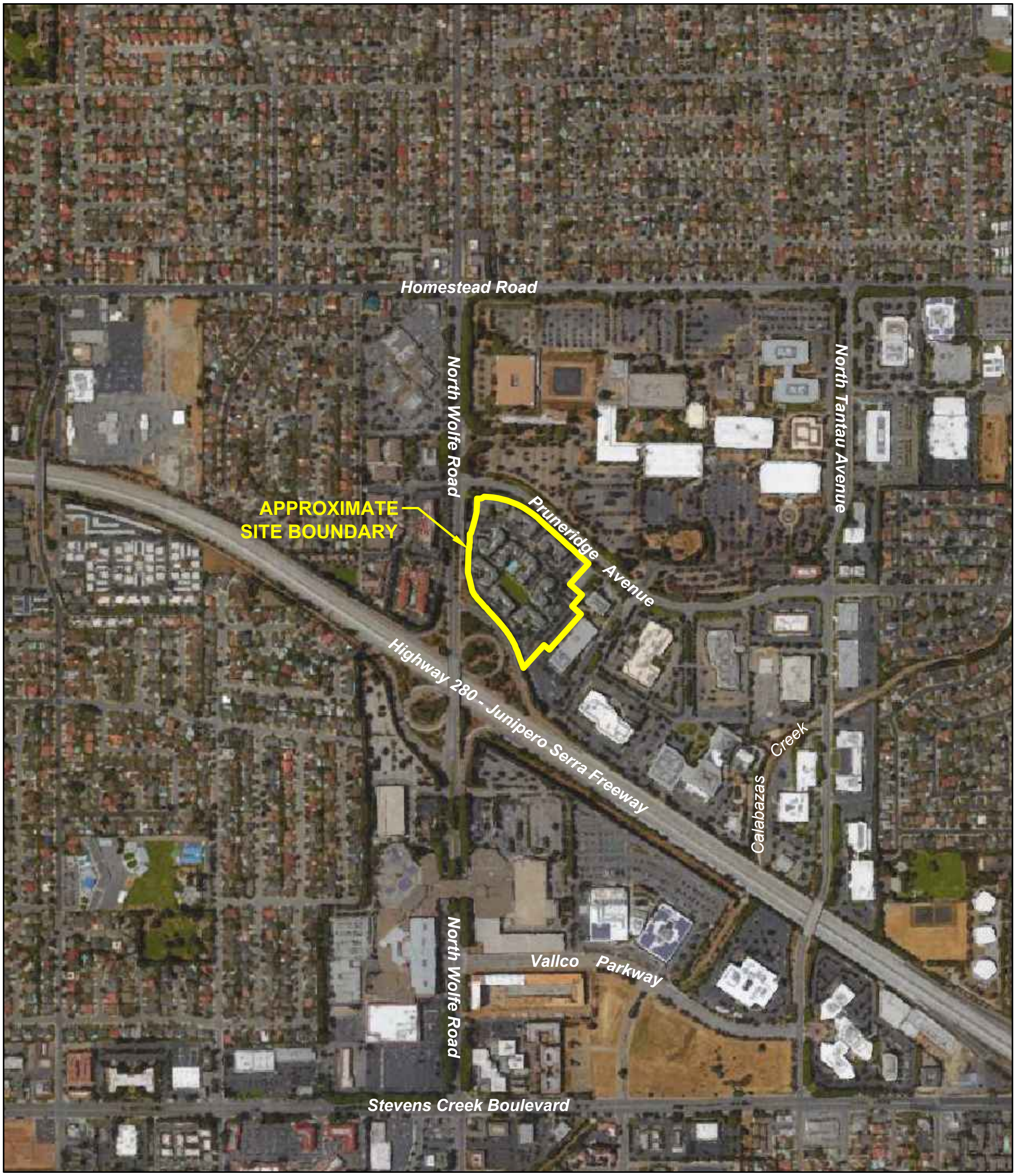
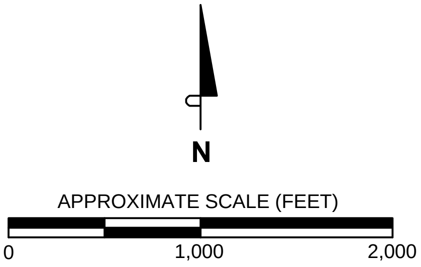


AERIAL PHOTO SOURCE: Google Earth, June 2015.



VICINITY MAP

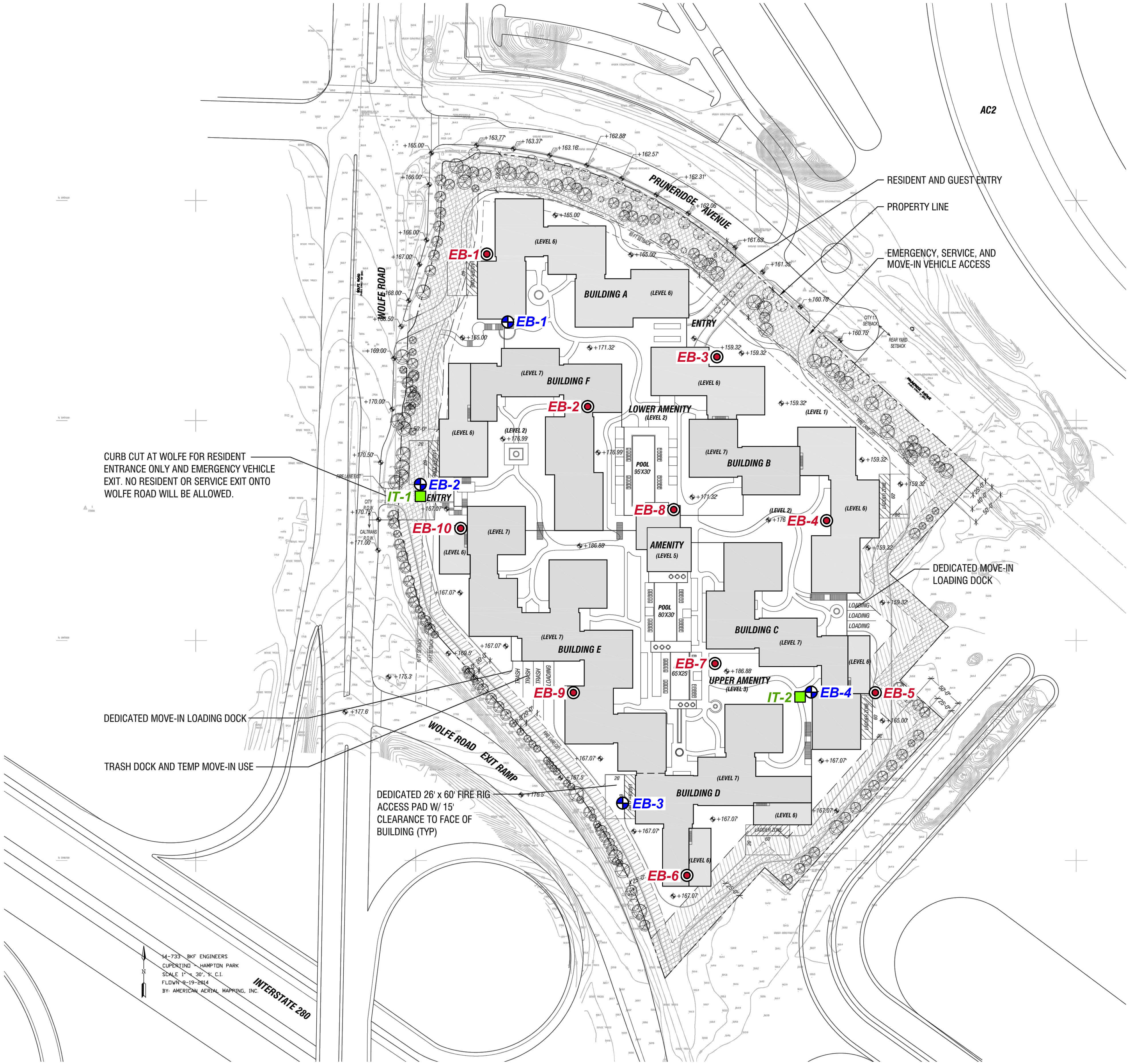
The Hamptons Apartments
19500 Pruneridge Avenue
Cupertino, California

TRC

238182

FIGURE 1

FILE NAME: N:\PROJECTS\CAD\Hamptons Apts_Cupertino\Fig1_Vicinity Map.dwg | Layout Tab: 8x11



LEGEND

Approximate locations of :

EB-10 ◉	Exploratory boring (Lowney Associates, 1997)
EB-4 ⊕	Exploratory boring (TRC, 2015)
IT-2 🟩	Infiltration test

N

SCALE (FEET)

0 150 300

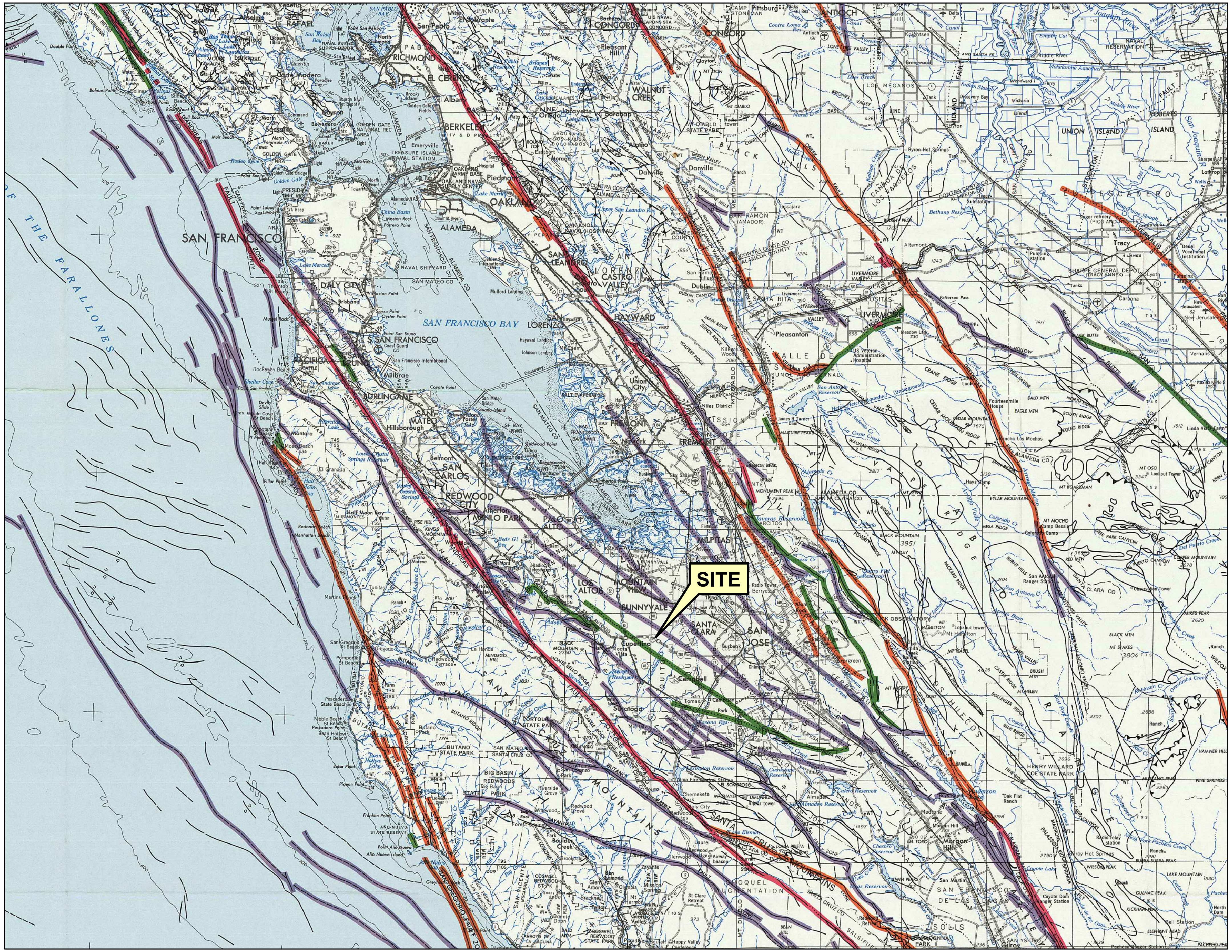
SOURCE: Site Plan provided by Irvine Company Apartment Communities, May 2015.

SITE PLAN

The Hamptons Apartments
19500 Pruneridge Avenue
Cupertino, California

TRC 238182 **FIGURE 2**

FILE NAME: N:\PROJECTS\CAD\Hamptons Apts_Cupertino\Fig2_Site Plan.dwg | Layout Tab: 11x17



SCALE: 1:500,000

APPROXIMATE GRAPHICAL SCALE (MILES)

0 10 20

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					

NOTES:

Base map is a composite of part the San Francisco 1:250,000 scale map (reference code 37 122-A1-TF-250-00, 1980) and the San Jose 1:250,000 scale map (reference code 37 120-A1-TF-250-00, 1969). For cartographic details, refer to these maps. Bathymetric information is not intended for navigational purposes.

Transverse Mercator Projection 10,000-meter Universal Transverse Mercator grid, zone 10.

Minor corrections and additions to culture by California Division of Mines and Geology 1987.

From: Bortugno & others (1991)

Some faults highlighted in purple are not considered active (Holocene Movement) by the State of California.

REGIONAL FAULT MAP

The Hamptons Apartments
19500 Pruneridge Avenue
Cupertino, California

TRC

238182

FIGURE 3

FILE NAME: N:\PROJECTS\CAD\Hamptons Apts_Cupertino\Fig3_Regional Fault Map.dwg | Layout Tab: 11x17

APPENDIX A
FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using conventional, truck-mounted, hollow-stem auger drilling equipment. Four 8-inch diameter exploratory borings were drilled on June 23, 2015 to a maximum depth of 40 feet. The approximate locations of the exploratory borings are shown on Figure 2. The soils encountered in the borings were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). The logs of the borings, as well as a key to the classification of the soil, are included as part of this appendix.

The locations of borings were approximately determined by pacing from existing site boundaries. Elevations of the boring were not determined. The locations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. Penetration resistance blow counts were obtained by dropping a 140-pound hammer 30 inches. Modified California 3.0-inch outside diameter (O.D.) samples and Standard Penetration Test (SPT) 2-inch O.D. samples were obtained by driving the samplers 18 inches and recording the number of hammer blows for each 6 inches of penetration. Unless otherwise indicated, the blows per foot recorded on the boring logs represent the accumulated number of blows required to drive the samplers the last two 6-inch increments. When using the SPT sampler, the sum of the last two 6-inch increments is the uncorrected SPT measured blow count. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

The 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.

* * * * * * * * * * * *

PRIMARY DIVISIONS			SOIL TYPE		SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (Less than 5% Fines)	GW	●	Well graded gravels, gravel—sand mixtures, little or no fines
			GP	○	Poorly graded gravels or gravel—sand mixtures, little or no fines
		GRAVEL WITH FINES	GM	◌	Silty gravels, gravel—sand—silt mixtures, plastic fines
			GC	▨	Clayey gravels, gravel—sand—clay mixtures, plastic fines
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (Less than 5% Fines)	SW	⁙	Well graded sands, gravelly sands, little or no fines
			SP	⁖	Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES	SM	⁞	Silty sands, sand—silt—mixtures, non—plastic fines
			SC	▨	Clayey sands, sand—clay mixtures, plastic fines
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50 %		ML	▥	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	▨	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	☰	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50 %		MH	▥	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	▨	Inorganic clays of high plasticity, fat clays
			OH	≋	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT	⁂	Peat and other highly organic soils

DEFINITION OF TERMS

U.S. STANDARD SIEVE SIZE — CLEAR SQUARE SIEVE OPENINGS

200 — 40 — 10 — 4 — 3/4" — 3" — 12"

SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

0.08 — 0.4 — 2 — 5 — 19 — 76mm

GRAIN SIZES

☒ TERZAGHI SPLIT SPOON STANDARD PENETRATION

⧗ MODIFIED CALIFORNIA

▯ ROCK CORE

◧ PITCHER TUBE

○ NO RECOVERY

SAMPLERS

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0–4
LOOSE	4–10
MEDIUM DENSE	10–30
DENSE	30–50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH+	BLOWS/FOOT*
VERY SOFT	0–1/4	0–2
SOFT	1/4–1/2	2–4
MEDIUM STIFF	1/2–1	4–8
STIFF	1–2	8–16
VERY STIFF	2–4	16–32
HARD	OVER 4	OVER 32

CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2–inch O.D. (1–3/8 inch I.D.) split spoon (ASTM D–1586).
+Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D–1586), pocket penetrometer, torvane, or visual observation.

KEY TO EXPLORATORY BORING LOGS

Unified Soil Classification System (ASTM D–2487)

FIGURE A–1

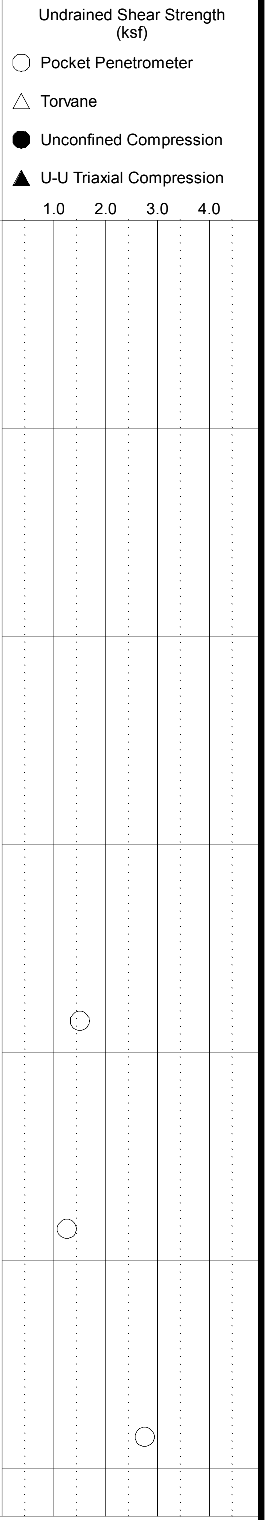
EXPLORATORY BORING: EB-1

Sheet 1 of 2

DRILL RIG: TRUCK MOBILE B-56	PROJECT NO: 238182
BORING TYPE: 8-INCH HOLLOW STEM AUGER	PROJECT: THE HAMPTONS APARTMENTS
LOGGED BY: AC	LOCATION: CUPERTINO, CA
START DATE: 6-23-15 FINISH DATE: 6-23-15	COMPLETION DEPTH: 40.0 FT.

This log is a part of a report by TRC, 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)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS SURFACE ELEVATION:	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	0	■	**3" of AC over 5" of AB**	AC/AB					
		▨	**CLAYEY SAND (SC)** dense, moist, brown to dark brown, fine to coarse sand, trace fine gravel (sub-angular)	SC	51	⧗	9		
		▨	medium dense		33	⧗	10		
	5	▨	coarse gravel (sub-angular)		45	⧗	7		
		░	**POORLY GRADED SAND (SP)** dense, moist, brown, fine to coarse sand, trace fine gravel (sub-angular/rounded)	SP	49	⧗	5		
	10	░							
	15	░	very dense		75	⧖	3		
		▨	**SANDY LEAN CLAY (CL)** stiff, moist, brown, low plasticity, fine sand	CL	23	⧖	17		
	20	▨							
	25	▨			26	⧖	18		
		▨	**LEAN CLAY (CL)** very stiff, moist, brown, low plasticity	CL	23	⧖	21		
	30		*Continued Next Page*						



GROUND WATER OBSERVATIONS:

NO FREE GROUND WATER ENCOUNTERED

EXPLORATORY BORING: EB-1 Cont'd

Sheet 2 of 2

DRILL RIG: TRUCK MOBILE B-56	PROJECT NO: 238182
BORING TYPE: 8-INCH HOLLOW STEM AUGER	PROJECT: THE HAMPTONS APARTMENTS
LOGGED BY: AC	LOCATION: CUPERTINO, CA
START DATE: 6-23-15 FINISH DATE: 6-23-15	COMPLETION DEPTH: 40.0 FT.

This log is a part of a report by TRC, 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)
- ○ Pocket Penetrometer
- △ Torvane
- ● Unconfined Compression
- ▲ U-U Triaxial Compression

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf) 1.0 2.0 3.0 4.0
	30	▨	**LEAN CLAY (CL)** very stiff, moist, brown, low plasticity	CL						
			SILTY SAND (SM) medium dense, moist, brown, fine to coarse sand, trace fine gravel (sub-angular/rounded)	SM						
	35	▨	**LEAN CLAY (CL)** very stiff, moist, brown, low plasticity, trace fine sand	CL	27	⧗	23	101		○
			CLAYEY SAND (SC) medium dense, moist, brown, fine sand	SC						
	40	▨	**LEAN CLAY (CL)** very stiff, moist, brown, low plasticity	CL	33	⧗	26	97		○
			Bottom of boring at 40 feet							
	45									
	50									
	55									
	60									

GROUND WATER OBSERVATIONS:

NO FREE GROUND WATER ENCOUNTERED

LA_CORP.GDT 7/10/15 MV*

EXPLORATORY BORING: EB-2

Sheet 1 of 1

DRILL RIG: TRUCK MOBILE B-56
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: AC
START DATE: 6-23-15 FINISH DATE: 6-23-15

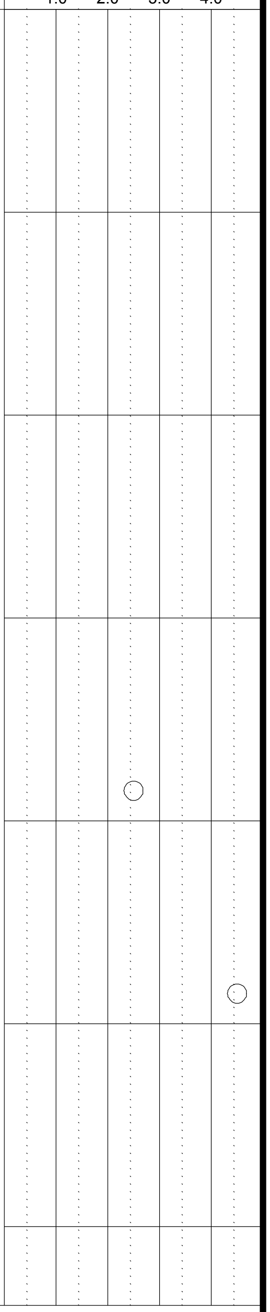
PROJECT NO: 238182
PROJECT: THE HAMPTONS APARTMENTS
LOCATION: CUPERTINO, CA
COMPLETION DEPTH: 30.0 FT.

This log is a part of a report by TRC, 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)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS SURFACE ELEVATION:	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	0		**2.5" of AC over 5.5" of AB**	AC/AB					
			CLAYEY SAND (SC) medium dense, moist, brown, fine to coarse sand, trace fine gravel (sub-angular)	SC	25	☒	7		
	5		coarse gravel		19	☒	5		
			dense		32	☒	5		
	10		**POORLY GRADED SAND WITH CLAY (SP-SC)** dense, moist, brown, fine to coarse sand, trace fine gravel (sub-angular/rounded)	SP-SC	31	☒	5		
	15		**POORLY GRADED GRAVEL WITH CLAY (GP-GC)** dense, moist, brown, some fine to coarse sand, fine to coarse gravel (sub-angular/rounded)	GP-GC	34	☒	5		
			SANDY LEAN CLAY (CL) very stiff, moist, brown, low plasticity, fine sand	CL					
	20		**CLAYEY SAND (SC)** medium dense, moist, brown, fine sand	SC	46	■	7		
	25		**SANDY LEAN CLAY (CL)** hard, moist, brown, low plasticity, fine sand, trace coarse sand	CL	26	☒	14		
			POORLY GRADED SAND (SP) dense, moist, brown, fine sand, trace fine to coarse gravel (sub-angular/rounded)	SP	34	☒	3		
	30		Bottom of boring at 30 feet						

Undrained Shear Strength (ksf)

○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression



GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

LA_CORP.GDT 7/10/15 MV*

TRC

EXPLORATORY BORING: EB-3

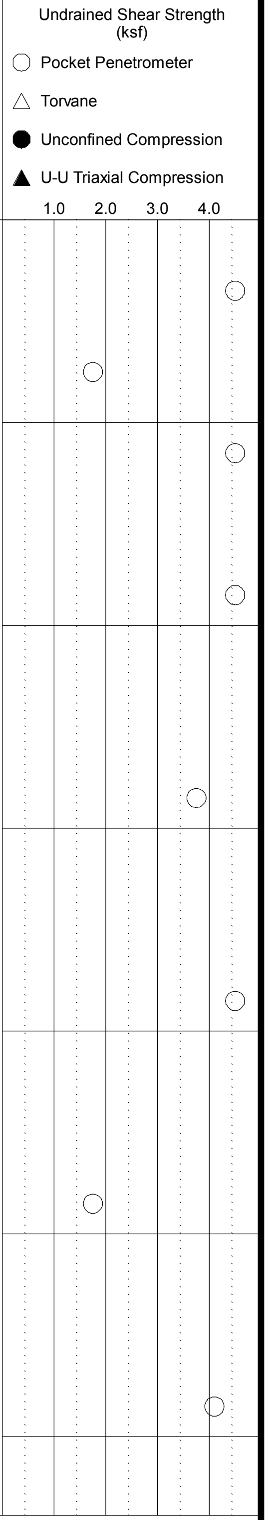
Sheet 1 of 1

DRILL RIG: TRUCK MOBILE B-56
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: AC
START DATE: 6-23-15 FINISH DATE: 6-23-15

PROJECT NO: 238182
PROJECT: THE HAMPTONS APARTMENTS
LOCATION: CUPERTINO, CA
COMPLETION DEPTH: 30.0 FT.

This log is a part of a report by TRC, 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)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS SURFACE ELEVATION:	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	0	■	**2.5" of AC over 5.5" of AB**	AC/AB					
		▨	**LEAN CLAY (CL)** hard, moist, dark brown, moderate plasticity	CL	30	⧖	15		
		▨	stiff, light brown, trace fine sand		21	⧖	10		
	5	▨	hard, no sand, dark brown		31	⧖	16		
	10	▨	**SANDY LEAN CLAY (CL)** hard, moist, brown, low plasticity, fine sand	CL	36	⧗	12	103	
	15	▨	**LEAN CLAY (CL)** very stiff, moist, brown, low to moderate plasticity, trace fine sand	CL	28	⧗	21	105	
		⁂	**POORLY GRADED GRAVEL (GP)** medium dense, moist, brown, trace fine to coarse sand, fine to coarse gravel (sub-angular/rounded)	GP					
	20	▨	**LEAN CLAY (CL)** hard, moist, brown, low plasticity, trace fine sand	CL	36	⧗	18	108	
	25	▨	**LEAN CLAY WITH SAND (CL)** stiff, moist, brown, low plasticity, fine sand	CL	25	⧗	24	97	
		▨	**LEAN CLAY (CL)** hard, moist, brown, moderate plasticity	CL	19	⧖	25		
	30		Bottom of boring at 30 feet						



GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

LA_CORP.GDT 7/10/15 MV*

TRC

EXPLORATORY BORING: EB-4

Sheet 1 of 2

DRILL RIG: TRUCK MOBILE B-56	PROJECT NO: 238182
BORING TYPE: 8-INCH HOLLOW STEM AUGER	PROJECT: THE HAMPTONS APARTMENTS
LOGGED BY: AC	LOCATION: CUPERTINO, CA
START DATE: 6-23-15 FINISH DATE: 6-23-15	COMPLETION DEPTH: 40.0 FT.

This log is a part of a report by TRC, 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.

SURFACE ELEVATION:

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	0	■	**3" of AC over 5" of AB**	AC/AB					
		▨	**CLAYEY SAND (SC)** dense, moist, brown, fine to coarse sand, trace fine gravel (sub-angular/rounded)	SC	30	☒	11		
					34	☒	8		
	5	▨	**SANDY LEAN CLAY (CL)** hard, moist, brown, low plasticity, fine to coarse sand, trace fine gravel (sub-angular/rounded)	CL	17	☒			
		▨	**CLAYEY SAND (SC)** medium dense, moist, brown, fine to coarse sand, trace fine gravel (sub-angular/rounded)		27	☒	8		
	10			SC					
			dense		34	☒	9		
	15								
		▨	**LEAN CLAY (CL)** very stiff, moist, brown, low plasticity, trace fine sand	CL	18	☒	23		
	20								
		▨	**SANDY LEAN CLAY (CL)** very stiff, moist, light brown, low plasticity, fine sand	CL	26	☒	13		
	25								
		▨	**LEAN CLAY (CL)** very stiff, moist, brown, low plasticity	CL	38	■	20	103	
	30								

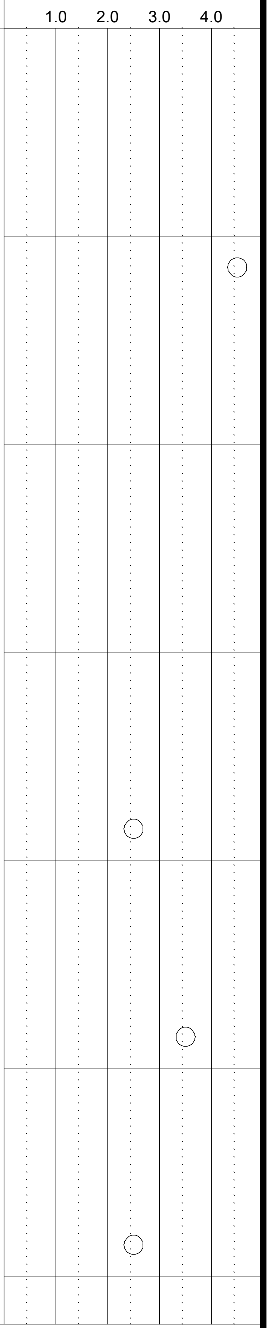
Undrained Shear Strength (ksf)

○ Pocket Penetrometer

△ Torvane

● Unconfined Compression

▲ U-U Triaxial Compression



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GROUND WATER OBSERVATIONS:

NO FREE GROUND WATER ENCOUNTERED

EXPLORATORY BORING: EB-4 Cont'd

Sheet 2 of 2

DRILL RIG: TRUCK MOBILE B-56
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: AC
START DATE: 6-23-15 FINISH DATE: 6-23-15

PROJECT NO: 238182
PROJECT: THE HAMPTONS APARTMENTS
LOCATION: CUPERTINO, CA
COMPLETION DEPTH: 40.0 FT.

This log is a part of a report by TRC, 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)
- ○ Pocket Penetrometer
- △ Torvane
- ● Unconfined Compression
- ▲ U-U Triaxial Compression

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf) 1.0 2.0 3.0 4.0
	30	▨	**LEAN CLAY (CL)** very stiff, moist, brown, low plasticity							
		▨			30	⧗	29	95		○
	35	▨		CL						
		▨			18	⧖	24			○
	40		Bottom of boring at 40 feet							
	45									
	50									
	55									
	60									

GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

LA_CORP.GDT 7/10/15 MV*

APPENDIX B
LABORATORY PROGRAM

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

Moisture Content: The natural water content was measured (ASTM D2216) on 35 samples of the materials recovered from the boring. These water contents are recorded on the boring log at the appropriate sample depths.

Dry Densities: In place dry density tests (ASTM D2937) were performed on 8 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring log at the appropriate sample depths.

* * * * * * * * * * * *

Corrosivity Tests Summary

CTL # 028-2418 **Date:** 6/30/2015 **Tested By:** PJ **Checked:** PJ

Client: TRC **Project:** The Hamptons **Proj. No:** 238182

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
EB-1	2A	3.5	-	-	8,824	6	6	0.0006	7.7	-	-	-	9.1	Dark Olive Brown Clayey SAND w/ Gravel
EB-3	4A	9.0	-	-	1,528	5	61	0.0061	7.6	-	-	-	20.3	Light Olive Brown CLAY
EB-3	5A	14.0	-	-	1,528	80	86	0.0086	7.9	-	-	-	22.8	Light Yellowish Brown SILT (slightly plastic)
EB-4	3A	6.0	-	-	3,455	26	70	0.0070	7.7	-	-	-	9.1	Dark Yellowish Brown Clayey SAND w/ Gravel

APPENDIX C

PREVIOUS BORINGS AT THE SITE AND LABORATORY PROGRAM

PRIMARY DIVISIONS				SOIL TYPE	LEGEND	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)		GW	▨	Well graded gravels, gravel-sand mixtures, little or no fines.
				GP	▨	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVEL WITH FINES		GM	▨	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
				GC	▨	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)		SW	▨	Well graded sands, gravelly sands, little or no fines.
				SP	▨	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES		SM	▨	Silty sands, sand-silt mixtures, non-plastic fines.
				SC	▨	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50%			ML	▨	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
				CL	▨	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
				OL	▨	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50%			MH	▨	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
				CH	▨	Inorganic clays of high plasticity, fat clays.
				OH	▨	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS				Pt	▨	Peat and other highly organic soils.

DEFINITION OF TERMS

	U.S. STANDARD SERIES SIEVE				CLEAR SQUARE SIEVE OPENINGS		
	200	40	10	4	3/4"	3"	12"
SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

GRAIN SIZES

◫ TERZAGHI
SPLIT SPOON
STANDARD PENETRATION

◿ MODIFIED CALIFORNIA

⊠ SHELBY TUBE
PITCHER TUBE

SAMPLERS

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH ‡	BLOWS/FOOT*
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
MEDIUM STIFF	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

CONSISTENCY

* Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).

‡ Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

KEY TO EXPLORATORY BORING LOGS
Unified Soil Classification System (ASTM D - 2487)

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE A - 1

DRILL RIG: Mobile B-40 SURFACE ELEVATION: --- LOGGED BY: BAF

BORING TYPE: 8-inch hollow stem DEPTH TO GROUND WATER: N/E DATE DRILLED: 12/3/96

FINED RESSIVE STRENGTH * (KSF)	SHEAR STRENGTH BY TORVANE (KSF)	DRY DENSITY (PCF)	WATER CONTENT (%)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	DEPTH (FEET)	LEGEND	SOIL TYPE	MATERIAL DESCRIPTION AND REMARKS
								SM	SILTY SAND (SM) dense, moist, brown, with gravel, fine to medium sand, subangular gravel to 1/4 inch, trace clayey fines
			6		41				
			4		63			SW	increased gravel, no clay
						5			GRAVELLY SAND (SW) very dense, moist, dark brown, well graded sand, subangular gravel to 1/2 inch, occassional cobbles to 2 inchs, trace fine sands
			4		67				decreasing fine sands
			3		47				
						10			
								SM	SILTY SAND (SM) medium dense, moist, brown, appreciable fine and occasional coarse sand
			14		19				
						15			Bottom of Boring = 15 feet
						20			
						25			
						30			
						35			
						40			

Note: The stratification lines represent the approximate boundary between the soil types. The transition may be gradual.

*Pocket Penetrometer Strength

EXPLORATORY BORING - EB-1

PRUNERIDGE AVENUE APARTMENTS
Cupertino, California

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

EB-1
1186-1A

DRILL RIG: Mobile B-40 SURFACE ELEVATION: -- LOGGED BY: BAF

BORING TYPE: 8-inch hollow stem DEPTH TO GROUND WATER: N/E DATE DRILLED: 12/3/96

NFINED COMPRESSIVE STRENGTH * (KSF)	SHEAR STRENGTH BY TORVANE (KSF)	DRY DENSITY (PCF)	WATER CONTENT (%)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	DEPTH (FEET)	LEGEND	SOIL TYPE	MATERIAL DESCRIPTION AND REMARKS
			7		9			SC	CLAYEY SAND (SC) loose, moist, dark brown, with gravel, well graded sand, subangular gravel to 1/2 inch, minor clay
			6		13			SM	SILTY SAND (SM) medium dense, slightly moist, brown, with gravel, well graded sand, subangular to subrounded gravel to 3/4 inch
						5			
			4		15				gravel to 1 inch
			6		32				decreased silt
						10			
			4		46			SW	GRAVELLY SAND (SW) dense, moist, brown, well graded sand, subangular gravel to 1 1/2 inch, trace silt
						15			
								SC	CLAYEY SAND (SC) dense, moist, dark brown, with gravel, minor coarse sand, subrounded gravel to 1/4 inch
						20			Bottom of Boring = 20 feet
									Note: The stratification lines represent the approximate boundary between the soil types. The transition may be gradual. *Pocket Penetrometer Strength
						25			
						30			
						35			
						40			

EXPLORATORY BORING - EB-2

PRUNERIDGE AVENUE APARTMENTS
Cupertino, California

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EB-2
1186-1A

DRILL RIG: Mobile B-40 SURFACE ELEVATION: -- LOGGED BY: BAF

BORING TYPE: 8-inch hollow stem DEPTH TO GROUND WATER: N/E DATE DRILLED: 12/3/96

ONFINED COMPRESSIVE STRENGTH * (KSF)	SHEAR STRENGTH BY TORVANE (KSF)	DRY DENSITY (PCF)	WATER CONTENT (%)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	DEPTH (FEET)	LEGEND	SOIL TYPE	MATERIAL DESCRIPTION AND REMARKS
			9		5			SC	CLAYEY SAND (SC) loose, moist, dark brown, with gravel, well graded sand, subangular to subrounded gravel to 1/4 inch
			7		6			SM	color changes to brown
						5			SILTY SAND (SM) loose, moist, brown, well graded sand, subangular to subrounded gravel to 3/4 inch, trace clay
			9		9				
			9		18				medium dense, sand gravel to 1 inch, no clayey fines
						10			
			5		21				
						15			
									dense
					58				
						20			
									trace clayey fines
			5		31				
						25			Bottom of Boring = 25 feet
						30			
						35			
						40			

Note: The stratification lines represent the approximate boundary between the soil types. The transition may be gradual.

*Pocket Penetrometer Strength

EXPLORATORY BORING – EB-3

PRUNERIDGE AVENUE APARTMENTS
Cupertino, California

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DRILL RIG: Mobile B-40 SURFACE ELEVATION: -- LOGGED BY: BAF

BORING TYPE: 8-inch hollow stem DEPTH TO GROUND WATER: N/E DATE DRILLED: 12/3/96

INED COMPRESSIVE STRENGTH * (KSF)	SHEAR STRENGTH BY TORVANE (KSF)	DRY DENSITY (PCF)	WATER CONTENT (%)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	DEPTH (FEET)	LEGEND	SOIL TYPE	MATERIAL DESCRIPTION AND REMARKS
			7		14			SC	CLAYEY SAND (SC) medium dense, moist, dark brown, well graded sand, subrounded gravel to 1 inch
			5		11			SM	SILTY SAND (SM) medium dense, moist, dark brown, with gravel, predominately fine to medium sand, trace subangular gravel to 1/2 inch
			4 4		18 43	5 10		SW	GRAVELLY SAND (SW) medium dense, slightly moist, brown, well graded sand, subangular to subrounded gravel to 3/4 inch, minor silt
			10		22	15		SM	SILTY SAND (SM) medium dense, brown, fine sands, appreciable silty fines
			4		40			SP	SAND (SP) dense, slightly moist, brown, poorly graded fine sand
9.0+			20 23		45	20		CL	SILTY CLAY (CL) very stiff, moist, brown, low plasticity, occasional fine rootlet
						25 30 35 40			Bottom of Boring = 21 1/2 feet Note: The stratification lines represent the approximate boundary between the soil types. The transition may be gradual. *Pocket Penetrometer Strength

EXPLORATORY BORING - EB-4

PRUNERIDGE AVENUE APARTMENTS
Cupertino, California

EB-4
1186-1A

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DRILL RIG: Mobile B-40 SURFACE ELEVATION: -- LOGGED BY: BAF

BORING TYPE: 8-inch hollow stem DEPTH TO GROUND WATER: N/E DATE DRILLED: 12/3/96

[illegible]FINED COMPRESSIVE STRENGTH * (KSF)	SHEAR STRENGTH BY TORVANE (KSF)	DRY DENSITY (PCF)	WATER CONTENT (%)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	DEPTH (FEET)	LEGEND	SOIL TYPE	MATERIAL DESCRIPTION AND REMARKS
								CL	3 inches asphaltic concrete
4.5			17		8				SILTY CLAY (CL) very stiff, moist, dark brown, low to moderate plasticity, trace fine sand
8.0		115	12		11			CL	
						5		SC	SANDY CLAY (CL) very stiff, moist, dark brown, low plasticity, fine to medium with trace coarse sand, occasional gravel to 1/2 inch
			9		15				
			7		21				CLAYEY SAND (SC) medium dense, moist, brown/dark brown mottled, with gravel, fine to medium sands, trace subangular gravel to 1 inch
						10			Bottom of Boring = 10 feet
						15			
						20			
						25			
						30			
						35			
						40			

Note: The stratification lines represent the approximate boundary between the soil types. The transition may be gradual.

*Pocket Penetrometer Strength

EXPLORATORY BORING - EB-5

PRUNERIDGE AVENUE APARTMENTS
Cupertino, California

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EB-5
1186-1A

DRILL RIG: Mobile B-40
SURFACE ELEVATION: --
LOGGED BY: BAF
BORING TYPE: 8-inch hollow stem
DEPTH TO GROUND WATER: N/E
DATE DRILLED: 12/3/96

NFINED COMPRESSIVE STRENGTH * (KSF)	SHEAR STRENGTH BY TORVANE (KSF)	DRY DENSITY (PCF)	WATER CONTENT (%)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	DEPTH (FEET)	LEGEND	SOIL TYPE	MATERIAL DESCRIPTION AND REMARKS
7.0		94	21	▨	19		▨	CL	1 inch asphaltic concrete SILTY CLAY (CL) very stiff, moist, dark gray brown, low to moderate plasticity, trace coarse sand, occasional subrounded gravel to 1/2 inch
		110	17	▨	9		▨	CL	SANDY CLAY (CL) stiff, moist, dark brown, low to moderate plasticity, abundant fine sand
						5			
		112	18	▨	34				
			18	▨	18		░	SC	CLAYEY SAND (SC) medium dense, moist, brown, fine to medium sand
						10			Bottom of Boring = 10 feet
						15			
						20			
						25			
						30			
						35			
						40			

Note: The stratification lines represent the approximate boundary between the soil types. The transition may be gradual.

*Pocket Penetrometer Strength

EXPLORATORY BORING – EB-6

PRUNERIDGE AVENUE APARTMENTS
Cupertino, California

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EB-6
1186-1A

DRILL RIG: Mobile B-40
SURFACE ELEVATION: --
LOGGED BY: BAF
BORING TYPE: 8-inch hollow stem
DEPTH TO GROUND WATER: N/E
DATE DRILLED: 12/3/96

INED ESSIVE STRENGTH * (KSF)	SHEAR STRENGTH BY TORVANE (KSF)	DRY DENSITY (PCF)	WATER CONTENT (%)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	DEPTH (FEET)	LEGEND	SOIL TYPE	MATERIAL DESCRIPTION AND REMARKS
			8		15			SC	CLAYEY SAND (SC) medium dense, moist, dark brown/black, well graded sand, trace gravel to 1/4 inch
			5		12			SC	CLAYEY SAND (SC) medium dense, moist, dark brown, well graded sand, subangular gravel to 1 inch
						5		SW	GRAVELLY SAND (SW) medium dense, trace clay, gravel to 1/2 inch
			6		25				
			4		27				
						10			
			3		13				trace clay/silt
						15			
									dense
			5		49				trace silt, increased gravel
						20			
			5		22				predominantly medium sand
						25			
			28		16				
						30		CL	SILTY CLAY (CL) very stiff, moist, brown, low to moderate plasticity, trace fine sand
7.0	3.0	82	27		35				
									Bottom of Boring = 31 1/2 feet
						35			
						40			

Note: The stratification lines represent the approximate boundary between the soil types. The transition may be gradual.

*Pocket Penetrometer Strength

EXPLORATORY BORING - EB-7

PRUNERIDGE AVENUE APARTMENTS
Cupertino, California

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EB-7
1186-1A

DRILL RIG: Mobile B-40
SURFACE ELEVATION: --
LOGGED BY: BAF

BORING TYPE: 8-inch hollow stem
DEPTH TO GROUND WATER: N/E
DATE DRILLED: 12/3/96

NFINED COMPRESSIVE STRENGTH * (KSF)	SHEAR STRENGTH BY TORVANE (KSF)	DRY DENSITY (PCF)	WATER CONTENT (%)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	DEPTH (FEET)	LEGEND	SOIL TYPE	MATERIAL DESCRIPTION AND REMARKS
			8		4			SC	CLAYEY SAND (SC) loose, moist, dark brown, with gravel, well graded sand, subrounded gravel to 1 inch
			5		18	5		SM	SILTY SAND (SM) medium dense, moist, dark brown, with gravel, well graded sand, subangular to subrounded gravel to 1 inch, trace clay
			4		27				
			5		16	10			trace gravel to 1/2 inch
			6		33	15			incresed clay, increased moisture
3.0			28		8	20		CL	SILTY CLAY (CL) stiff, moist, brown, low to moderate plasticity, occasional medium sand
			16		29			SC	CLAYEY SAND (SC) medium dense to dense, moist, brown, with silt, fine sand, trace clayey fines
			19		34	25			
4.0	1.6	110	20		20	30		CL	SANDY CLAY (CL) very stiff, moist, brown, low to moderate plasticity, fine sand
3.0	2.0	102	25		23	35			interbedded sandy silt
						40			Bottom of Boring = 35 feet Note: The stratification lines represent the approximate boundary between the soil types. The transition may be gradual. *Pocket Penetrometer Strength

EXPLORATORY BORING - EB-8

PRUNERIDGE AVENUE APARTMENTS
Cupertino, California

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DRILL RIG: Mobile B-40
SURFACE ELEVATION: --
LOGGED BY: BAF
BORING TYPE: 8-inch hollow stem
DEPTH TO GROUND WATER: N/E
DATE DRILLED: 12/3/96

FINED RESSIVE STRENGTH * (KSF)	SHEAR STRENGTH BY TORVANE (KSF)	DRY DENSITY (PCF)	WATER CONTENT (%)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	DEPTH (FEET)	LEGEND	SOIL TYPE	MATERIAL DESCRIPTION AND REMARKS
								SM	SILTY SAND (SM) loose/medium dense, moist, dark brown, with gravel, fine to medium sand, gravel to 1/4 inch
			8		8				
			7		5				
						5			
									increasing gravels
			7		13				
			10		23				
						10			
								SM	SILTY SAND (SM) dense to very dense, light brown, fine sand, trace clay
			26		34				
						15			
									increased sand, fine to medium
			13		71				
								SW	GRAVELLY SAND (SW) dense, moist, dark brown, well graded sand, subangular to subrounded gravel to 2 inches
			5		53				
						20			Bottom of Boring = 20 feet
						25			
						30			
						35			
						40			

Note: The stratification lines represent the approximate boundary between the soil types. The transition may be gradual.

*Pocket Penetrometer Strength

EXPLORATORY BORING - EB-9

PRUNERIDGE AVENUE APARTMENTS
Cupertino, California

EB-9
1186-1A

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DRILL RIG: Mobile B-40 SURFACE ELEVATION: -- LOGGED BY: BAF

BORING TYPE: 8-inch hollow stem DEPTH TO GROUND WATER: N/E DATE DRILLED: 12/3/96

FINED COMPRESSIVE STRENGTH * (KSF)	SHEAR STRENGTH BY TORVANE (KSF)	DRY DENSITY (PCF)	WATER CONTENT (%)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	DEPTH (FEET)	LEGEND	SOIL TYPE	MATERIAL DESCRIPTION AND REMARKS
								SC	CLAYEY SAND (SC) loose, moist, dark brown, with gravel, well graded sand, subangular to subrounded gravel to 3/4 inch
			10		3				
			10		7				
						5		SM	SILTY SAND (SM) medium dense, moist, brown, well graded sand, subangular to subrounded gravel to 3/4 inch, trace clay/silt
			8		16				
			7		28				gravel to 1 inch
						10			
			5		19				predominantly medium sand, gravel to 1 inch
						15			Bottom of Boring = 15 feet Note: The stratification lines represent the approximate boundary between the soil types. The transition may be gradual. *Pocket Penetrometer Strength
						20			
						25			
						30			
						35			
						40			

EXPLORATORY BORING – EB-10

PRUNERIDGE AVENUE APARTMENTS
Cupertino, California

EB-10
1186-1A

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APPENDIX B

LABORATORY INVESTIGATION

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D-2216) on all 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 D-2937) were performed on eight samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depth.

R-Value: A R-value resistance test (California Test Method No. 301) was performed on a representative sample of the surface soils at the site to provide data for the pavement design. The test indicated an R-value of 57 at an exudation pressure of 300 pounds per square inch. The results of the test are presented on Table B-1.

TABLE B-1. **Results of R-Value Tests**

Sample	Description of Material	Water Content (%)	Dry Density (pcf)	Exudation Pressure (psi)	"R" value	Expansion Pressure (psf)
B-1	Clayed Sand with gravel	9.9	128.9	213	33	0
		8.2	132.7	800	81	108
		9.1	131.3	398	67	9

* * * * * * * * *